PAGES

MISSING

The Canadian Engineer

A weekly paper for Canadian civil engineers and contractors

QUEBEC BRIDGE DISASTER

HISTORY OF FAMOUS STRUCTURE—SOME DETAILS OF LIFTING APPARATUS —RECORD OF INCIDENTS LEADING UP TO COLLAPSE OF SUSPENDED SPAN.

AS early as 1852 a project for a bridge over the St. Lawrence River at Quebec was considered, and again in 1884 a design was prepared and submitted to the Quebec Board of Trade for a bridge at about the present site, but nothing actually was done until about 1900, when the Quebec Bridge and Railway Company located a site near Cap Rouge and took definite steps towards the erection of such a structure.

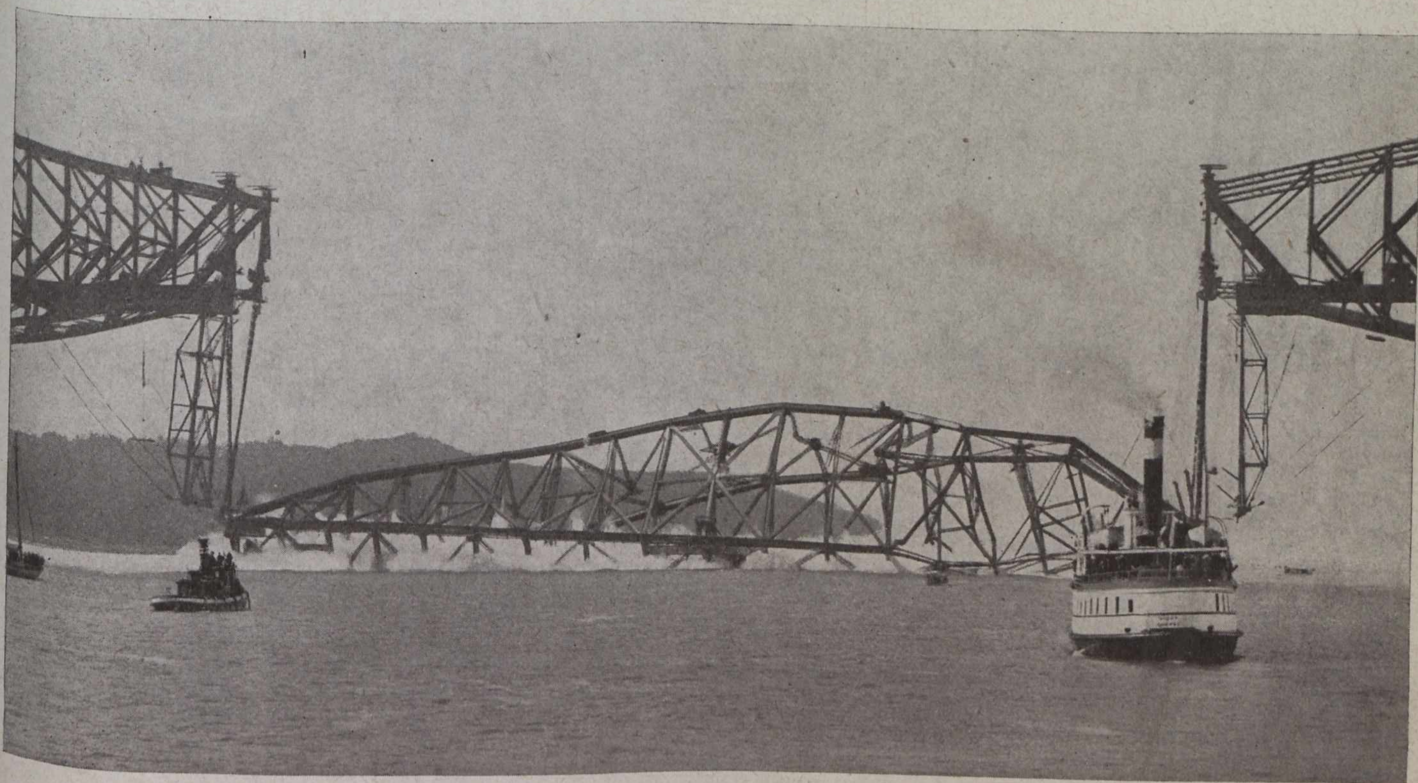


Fig. 1.—Taken Just as Centre Span is Falling into River After Failure of Casting on South Lifting Girder.

As the photograph shows, the hoisting connection on one of the southern extremities of the centre span became detached. That corner of the span dropped twisting the whole structure out of shape.- Copyrighted in Canada, Great Britain and the United States by Chesterfield and McLaren.

Main Dimensions of the First Quebec Bridge.

Type of bridge, cantilever.

Total length of bridge between abutments, 3,220 feet.

Consisted of: Two deck truss approach spans, each 210 feet long; two anchor arms, each 500 feet; two cantilever arms, each 562½ feet long; one suspended span, 675 feet long.

Central span, centre to centre of main piers, 1,800 feet; the longest in the world.

Type of trusses, pin-connected.

Width, centre to centre of trusses, 67 feet.

Depth of trusses varied from 97 feet at the portals to 315 feet over main piers.

Clear headway over high tide, 150 feet, for a width of 1,200 feet.

Height of peaks of main posts above the river, 400 feet.

Capacity, two railway and two electric railway tracks, two roadways and two footwalks, all on same level.

Total weight of steel in bridge, 38,500 tons.

Weight of heaviest single pieces handled, 100 tons.

Longest single section shipped to bridge site, 105 ft.

Eyebars, the largest yet used, with a maximum of 56 on one pin.

Diameters of pins, from 9 to 24 inches, and up to 10 feet in length.

Type of traveller used for erecting anchor and cantilever arm trusses, gountry, running outside of trusses, on tracks at about floor level, and spanning highest point of bridge.

Weight of gountry traveller, fully rigged, with all accessories, 1,000 tons.

Steel wire cable on traveller, seven miles of seven-eighth.

Manilla rope on traveller, 13 miles of 1-inch, 1½-inch, 1¾-inch, and 2-inch.

Grade of one per cent. on each end, connected at centre by vertical curve, 1,125 feet long.

Main piers were built of concrete, faced with massive rock-faced granite, were sunk with pneumatic caissons 150 by 49 feet, and 25 feet high. The tops of these piers measured 133 by 30 feet, and contained 35,000 cubic yards of masonry.

Anchor piers, built of concrete, faced with granite, were 30 by 111 feet at the base, 56 feet high from bottom of anchorage metal, measured 24 feet by 105 feet at the coping, and contained 14,400 cubic yards of masonry.

This location is at the narrowest point on the St. Lawrence River between Montreal and Quebec, the width at mean water level being about 2,000 feet. The water at this point has a maximum depth of about 200 feet and a current at ebb tide of about 7 miles per hour. The Bridge and Railway Company awarded contracts in 1900 for a bridge of the cantilever type, having a main span of 1,800 feet. Work was started and proceeded until the year 1907, when about half the superstructure then collapsed.

The Present Bridge.—After the accident which destroyed the first bridge a commission was appointed to examine the fallen structure and to make a report thereon. The gentlemen comprising this body were Henry Holgate, Prof. J. G. G. Kerry, of McGill, and the late Dr. Galbraith, of Toronto University.

Following this report, the government decided to reconstruct the bridge, and in 1908 appointed a board of three engineers for that purpose. The commission named was composed of H. E. Vautelet, chairman, of Montreal; Ralph Modjeski, Chicago, and Maurice Fitzmaurice, chief engineer of the County Council of London, England. Each of these was a very strong man, both in his personality and in his profession. After studies lasting about a year, Mr. Fitzmaurice resigned.

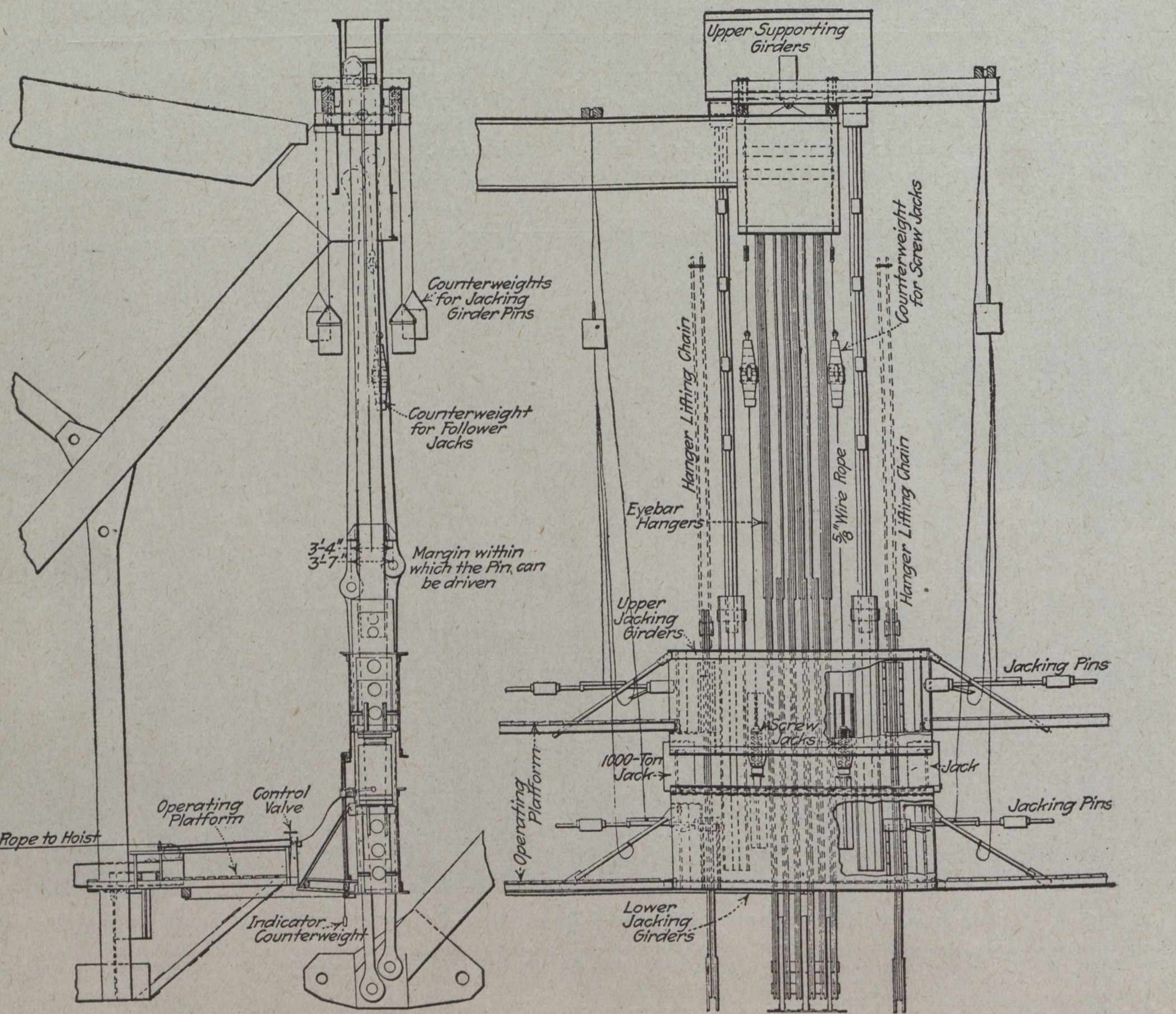


Fig. 2.—Jacking Equipment Used for Hoisting Span into Position, Showing Counterweights for Jacking Pins, etc.

This board made very exhaustive studies of various designs, including suspension and cantilever bridges, and finally decided, for good and sufficient reasons, that the cantilever type of bridge was the most satisfactory and economical kind of structure for such a crossing. It also decided that the bridge should be much wider and designed for heavier loading than the former bridge, that the same length of main span should be retained and that it should be built at the same site.

Tenders were received from one company in Germany, one in England, two in the United States and from the St. Lawrence Bridge Company. In addition to

Messrs. Vautelet, Modjeski and McDonald, Messrs. Hodge, of New York, and M. J. Butler, then manager of the Dominion Steel Works, were called in for consultation as to what tender ought to be accepted and what design approved. As a result, four out of five advised that the design of the St. Lawrence Bridge Company, on which the bridge has been constructed, should be accepted.

The contract was finally let to the St. Lawrence Bridge Company for the superstructure on April 4th, 1911; that for the substructure having been let to M. P. Davis on January 10th, 1910.

The chairman of the board resigned and was succeeded by Mr. C. N. Monsarrat, as chairman and chief engineer. Mr. McDonald having only agreed to act until such time as the contract was signed, also wished to be relieved of his duties and he was succeeded by C. C. Schneider, of New York, since deceased. Then began the real construction work.

Several changes have been made in the personnel of this board during the progress of the work, and at present it consists of C. N. Monsarrat (chairman and chief engineer), Ralph Modjeski and H. P. Borden, who took Mr. Schneider's place on the board, and the work has progressed under these men for some time.

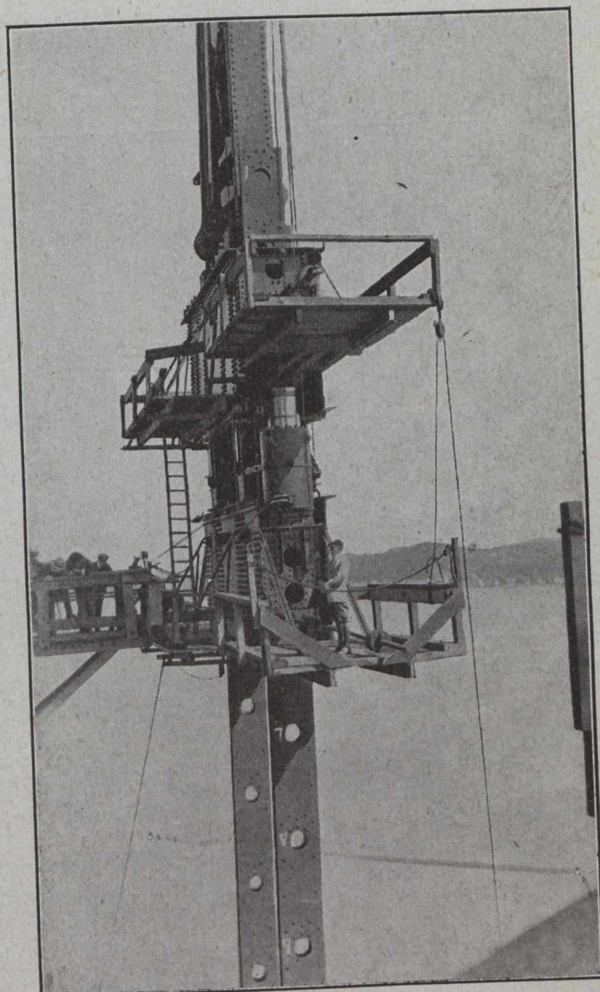


Fig. 3.—Showing Powerful Jacks in Position; Also the Lifting Chain in which Pins were Inserted as Centre Span was Elevated.

Launching of Centre Span.—It had been decided to float the span on the morning of September 11th, provided suitable weather conditions were predicted and existed. On September 10th at 11 a.m. the Weather Bureau at Toronto telephoned that there was a centre of low pressure over the Western provinces, Saskatchewan in particular, the barometer reading there being 29.4 inches; and that a centre of high pressure existed over the provinces of Ontario and Quebec, the barometer reading in these localities being 30.4 inches. The forecast for wind was a fresh breeze from the northeast of about 20 miles an hour velocity. On the evening of the 10th at 11 p.m. the report came from Toronto of the existing meteorological conditions and the wind velocity forecast for the morning of September 11th. The centre of low pressure had moved from Saskatchewan to Brandon, Manitoba, in twelve hours, the barometer reading being 29.18 inches. The centre of high pressure was still over Ontario and Quebec, the barometer reading being 30.46 inches, and the forecast for the wind was moderate easterly winds with a velocity of from 12 to 14 miles per hour. The electric storm detector at the bridge site showed no indications of any coming disturbances, and the night was clear and cold, with practically no wind. It was therefore decided to float the span.

The scows had all drained by 11 p.m. September 10th and the tide was still falling. The valves in the bottom of the scows were therefore closed between 11.45 and 12.45 o'clock midnight. The tide gradually rose and at 3.30 a.m. the span was floating free of its end supports, the whole structure being carried by the scows. The calculated draft of the scows while carrying the load was 8 ft. 2 ins. Measurements showed that the actual draft was 8 ft. 3 ins.

The morning broke cold and clear except for a slight mist which floated over the surface of the river obstructing the view to a small extent, but it was seen that this was gradually rising and dispersing. Two tugs of about 500 h.p. capacity were attached to the stream side of the span, one at each end. The span was first moved out of its berth by the pull from the hoisting engines, situated on the floor of the span at each end, the reaction from the pull of the engines being taken by the end supporting bents. The two tugs took up the slack in their towing cables of about 3 ins. in diameter, at 4.38 a.m., and the span gradually moved out at the rate of about 10 ft. a minute. By 4.50 a.m. the span had moved 88 ft. and was clear of the supporting bents. Four minutes later the shore lines were cast off except the one leading downstream which held the span against the four-mile upstream tidal current, and as the span moved out it swung about the

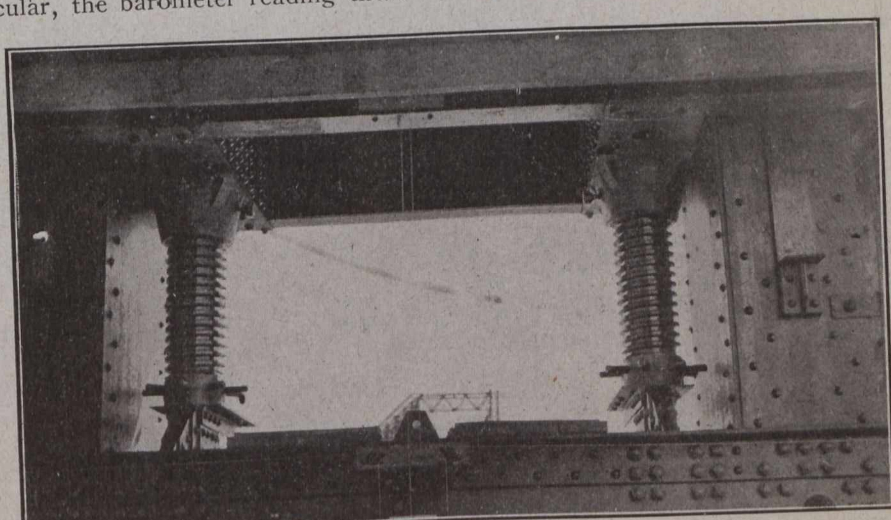


Fig. 4.—Hand-operated Screw Jacks.

The upward stroke of the hydraulic jacks was followed by these jacks to provide against dropping the load through any accident to the hydraulic connections.

anchorage point of this line as a centre. As the immense structure floated out there was not the slightest indication of any swaying or rocking motion, and it was practically as steady as if it had been resting on a solid foundation.

At 5.05 a.m. two more tugs took hold of the barges on the stream side of the span, one at each end. These tugs were of approximately 500 h.p. capacity. At 5.13 the span had pivoted about the anchorage point of the mooring line until it was at an angle of approximately 45° to the line of flow of the current. The mooring line was then cast off, and at this time a fifth tug of 1,000 h.p. capacity with a 4½-in hawser took hold of the middle of the span on the stream side. Five tugs were now holding the span against the pull of the tidal current, the tide being now at full flood and running up-stream with a velocity of 4 to 5 miles per hour. Two more tugs of 500 h.p. capacity prepared to take hold of the scows on the opposite or upstream side of the span. Communication with each end of the span and the centre was kept up by telephone and the orders to the tugs were given by megaphone. At 5.22 a.m. the span was practically normal to the line of flow of the current and began to float upstream under the restraint of the five tugs on the downstream side. At 5.23 the sun rose over the eastern hills and the mist was gradually dispersing before a slight east breeze of about two to three miles per hour velocity. By 5.30 the span was moving upstream at a rate of speed of about two miles per hour, the velocity of the current being about four miles per hour.

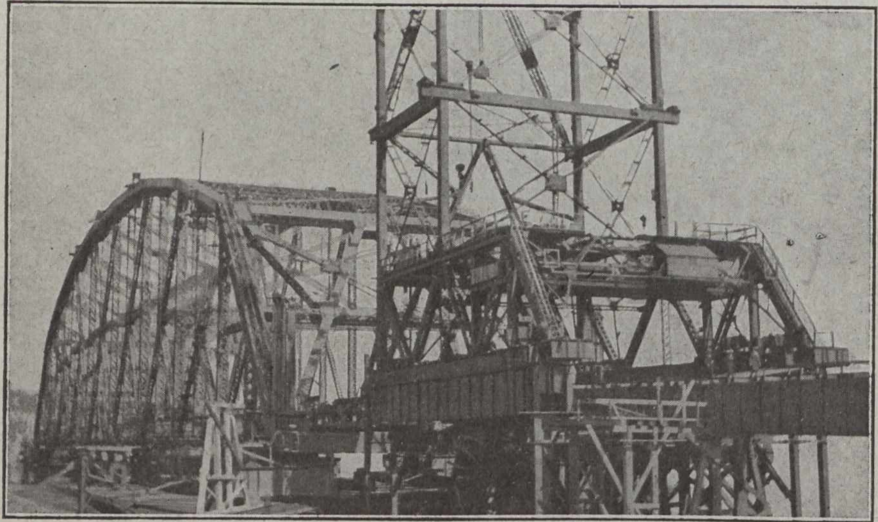


Fig. 5.—Showing the Centre Truss Lying at Sillery Cove Before Launching.

Ranges placed at measured distances apart along the shore recorded the advance and rate of progress of the span on its journey to the bridge site. The first range was passed at 5.33 a.m. and the second at 5.44 a.m. The distance between these ranges was 1,700 ft. and the rate of speed of the span had been 1¼ miles per hour. The distance between the second and third ranges was 2,460 ft. and the third range was passed at 5.53, the average speed of the span between these two ranges being three miles per hour. The distance to the fourth range was 4,200 ft. and it was passed at 6.05 a.m. at a speed of

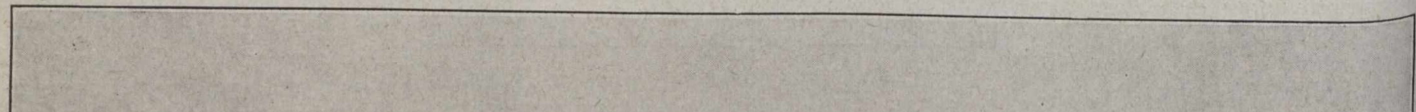


Fig. 6.—Showing Suspended Span, Mounted on Scows, Being Towed to Bridge.

about 4 miles per hour. A float thrown out in front of the span showed the speed of the current to be between 4 and 5 miles per hour and the span followed this float very closely until the seventh range was passed. At the seventh range the speed of the span was checked and the span brought practically to a standstill for a moment in order to show that the tugs had perfect control of the floating structure. It took approximately 3 minutes to stop the span. The span was then about three-quarters of a mile from the main bridge site and from then on was allowed to move slowly forward at a speed of about two miles per hour, and as it approached the space between the two cantilever arms it was lined up parallel to the main bridge by ranges on the shore and normal to the bridge by centering targets suspended by wires at the middle of the opening between the cantilever arms.

Fig. 7.—Showing Span in Place After North Mooring Arm Had Been Connected and Just Before Scows Floated Away.

At 6.50 a.m. the span arrived at the bridge site and the mooring lines were connected up to the cast steel snubbing posts located at each of the four corners of the suspended span. These 1¼-in. steel mooring ropes, eight in number, four at each end of the span, were calculated to take a pull of 75,000 lbs. each and passed through sheaves at the lower corners of the mooring trusses and from there up to a nine-part ¾-in. wire rope tackle which led back to the drums of the derrick hoists situated on the bridge floor at the ends of the cantilever arms. The span was pulled directly under its final position in the bridge by means of these 1¼-in. ropes and the derrick hoists. The hanger lifting chains which were to raise the span were then lowered and connected through the slotted holes at the lower ends to the pins at the top of the short hanger links connecting to the supporting girders under the end corners of the span. This connection was made at 7.40 a.m., when the current was practically at zero; that is, the tide had turned and the current was about to change from a westward to an easterly flow.

The mooring frames were made up of two steel trusses braced together, the bracing being designed to take a transverse pull from each end of the suspended span of 300,000 lbs. They were pin-connected to the cantilever arm floorbeams so that by means of the nine-part ⅞-in. wire rope tackle leading from the lower corners of the trusses to the connection to the floor between panel points CF5 and CF6 of the cantilever arm and from there to the main hoists situated on the bridge floor, they could be raised so as not to obstruct the channel unnecessarily.

The hanger chains at each corner of the span were made up of four slabs to each chain, each slab being built up of two 30-in. by 1⅛-in. carbon steel plates. The slabs were manufactured and shipped in lengths of about 30 ft. centre to centre of end connecting pins. They were controlled after being suspended from the jacking girders located at the elevation of the bottom chords of the cantilever arms, by means of a two-part tackle connecting to the cantilever arm trusses at panel point CL2.

Details of Lifting Apparatus.—The hoisting apparatus at each corner of the span is illustrated in Fig. 2. A pair of supporting girders 6 ft. 11½ ins. deep and 25 ft. 0 in. long and braced together by bearing and pin-connecting diaphragms and cover plates, were placed under each corner of the span. The plate-lifting chains were pin-connected to these girders and also riveted to the same girders and passed up through a set of upper and lower jacking girders to which they were alternately pin-connected as the jacks between the upper and lower jacking girders were operated. These jacking girders were each made up of two plate girders 9 ft. 0 in. deep and 22 ft. 6 ins. long, connected together by cross bearing diaphragms and cover plates. The upper jacking girders were the movable girders and slid up and down in the stiff built guides which were riveted into the lower jacking girders, passed up through the upper jacking girders and connected to the stiff hangers which led on up to the upper supporting girders. These upper supporting girders were placed on top of the CUO joints of the cantilever arms, and were of similar construction to the lower supporting girders. The load of the span was transferred to the upper and lower supporting girders, by means of cast steel rocker bearings, designed so as to allow the span to sway in any direction under the influence of external forces from current and wind which might have acted while the span was being hoisted.

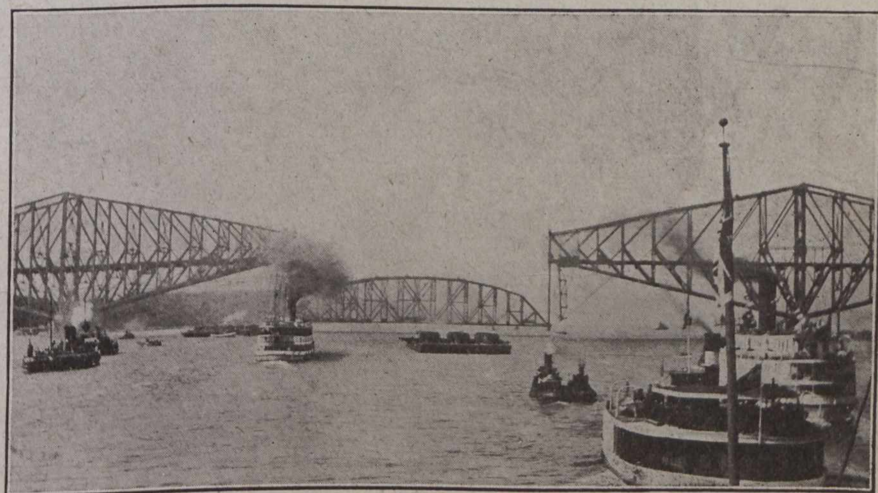


Fig. 8.—Taken Just After the Barges had Left and the Load is Being Carried by Cantilever Arms. Note One of the Specially Built Barges Floating Down Stream.

The total load carried by the hanger chains while lifting the span was 5,147 tons. The supporting girders, hanger chains, jacks and jacking girders and all their connections were designed throughout to carry this lifted load plus 20 per cent. impact.

The work of hoisting was done by eight 1,000-ton hydraulic jacks, placed two at each corner of the span, as shown in Fig. 3. These jacks were operated under a pressure of 4,500 lbs. per square inch, the water being supplied to them by a pair of direct-acting double-plunger pumps operated by compressed air and located on the centre line of the bridge floor at the ends of the cantilever arms. Valves placed in the feed pipe lines in front of the pumps controlled the water supply sent to each corner of the span and by this means and with the aid of an indicator, which showed any difference in elevation between the two sides of the span, the span was kept approximately level. Another set of valves with a similar indicator attachment was located in front of each set of jacking girders and controlled the water supplied to each separate jack and by means of which the jacking girders were kept horizontal.

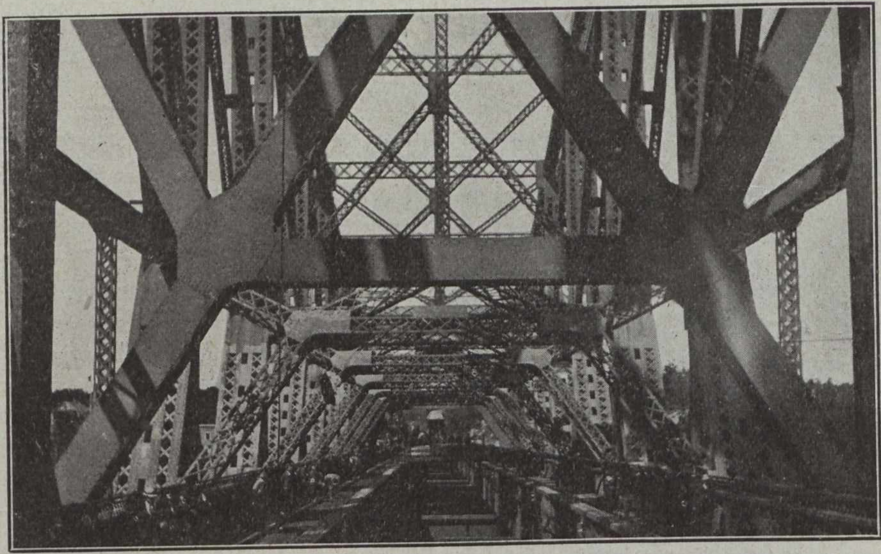
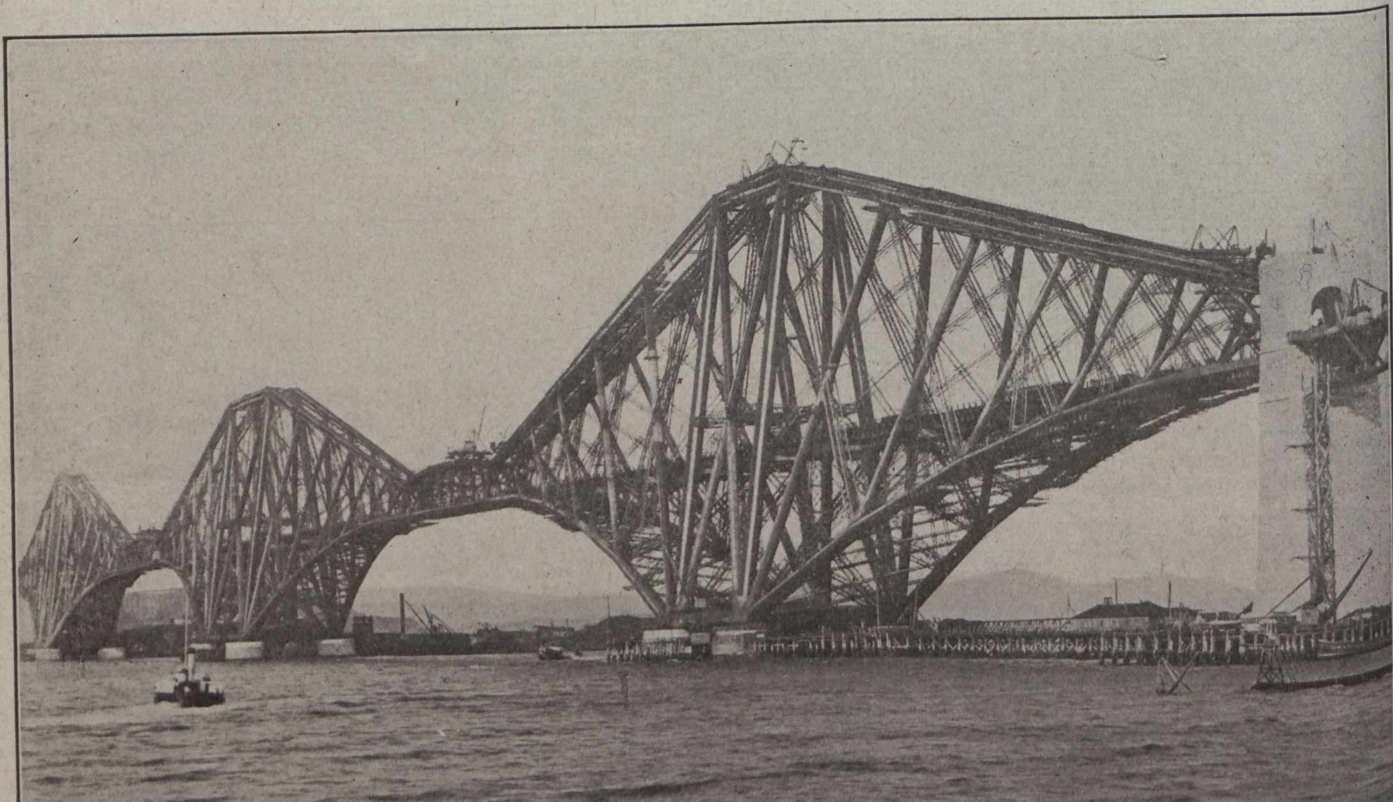


Fig. 9.—View Taken Looking Along the Floor of North Cantilever Arm.

At 8.30 a.m. the tide had dropped sufficiently to make the pins bear at the ends of the slotted holes in the hanger links and the links themselves had straightened out. At 9.00 a.m. the tide had fallen about a foot and a half further, and the first jacking operation was commenced. Each operation of the jacks lifted the span 2 ft. During the lifting or upward stroke the 12-in. pins engaged the hanger chains through the diaphragms in the upper jacking girders. At the finish of the stroke the pins were entered in the diaphragms of the lower jacking girders to engage the hanger chains. The upper pins were then removed, the jacks and upper girders lowered, the upper pins again entered, the lower pins removed, and the jacks



FORTH BRIDGE NEARING COMPLETION.

As so many comparisons have been made between the Quebec and the Forth Bridges the accompanying picture will be of interest. Notice the short suspended spans in the Forth Bridge made possible by the use of Inchgarvie, a small island in the middle of the river, which formed a natural pier. The Forth Bridge is 5,349 feet long between approaches, cost about $17,000,000, contains 54,000 tons of metal and about 250,000,000 tons of masonry.

again operated. The first operation of the jacks was completed in fifteen minutes; the second and third operations in about 13 minutes each. At the end of the third operation the span had lifted clear of the scows, the scows had drifted away and were taken care of by the waiting

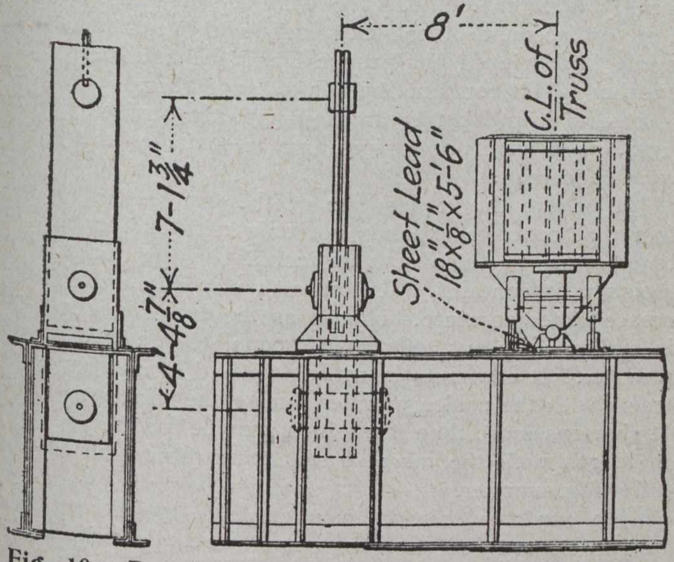


Fig. 10.—Detail of Lifting Girders, Suspected Casting and Ends of Lifting Links.

tugs, leaving the span supported entirely by the hoisting apparatus on the ends of the cantilever arms. The fourth stroke was completed, after which an intermission of about an hour was allowed in order that the working men could get their breakfast, and upon their return the jacks were again operated. The sixth stroke had been completed and while the upper jacking girders were in the act of being lowered the cast steel bearing on the lower lifting girder supporting the span at the southwest corner failed. The span slipped off the support at this corner, the sway and lateral bracing then collapsed, precipitating the span into the water. As it fell it turned over to the west and plunged to the bottom of the river, where it lies a mass of twisted steel, 200 feet below the river surface. As the load of the span was suddenly released from the ends of the cantilever arms these arms whipped back, vibrated and swayed to such an extent that a number of the workmen, engineers, and visiting members of the engineering profession, assembled to view the hoisting of the span, were thrown off their feet. These vibrations lasted for some seconds, then gradually subsided, leaving the anchor and cantilever arms in the same condition as they were before they had taken the weight of the span. Check elevations have since been made and it has been established, after examination, that these arms are apparently uninjured in any manner after having been subjected to a more severe test than would have been possible under ordinary service conditions.

Inspection of Work by Engineers.—On the Saturday previous a party of engineers, many of them experienced bridge engineers, who had gone over the centre span, the cantilever arms, inspected the arrangements that had been made for elevating, made careful note of the safeguards which had been made to eliminate the faintest chance of accident, were all, almost without exception, fully convinced that all that mortal man could do had been done to prevent any slip. It was, therefore, to these men a great shock when the collapse came.

Quebec Bridge in Tabloid

Total length	3,239 feet
Length of centre span	640 feet
Height of bridge floor	150 feet
Top of cantilever arm	348 feet
Length of north cantilever arm	515 feet
Distance between centre of piers	1,800 feet
Length of south cantilever arm	515 feet
Total weight of bridge	66,000 tons
Weight of centre span	5,200 tons
Height of centre span	110 feet
Width of centre span	88 feet
Total yardage in masonry on job	105,000 cu. yds.
Approximate cost,	$14,000,000.00

Contract for substructure let January, 1910.
Contract for superstructure let April, 1911.

The span fell after it had been raised twelve feet after leaving the scows and the well-laid, carefully thought-out plans of the leading bridge engineers of the country, were suddenly thwarted and the massive structure, weighing 5,200 tons, was at the bottom of the River St. Lawrence.

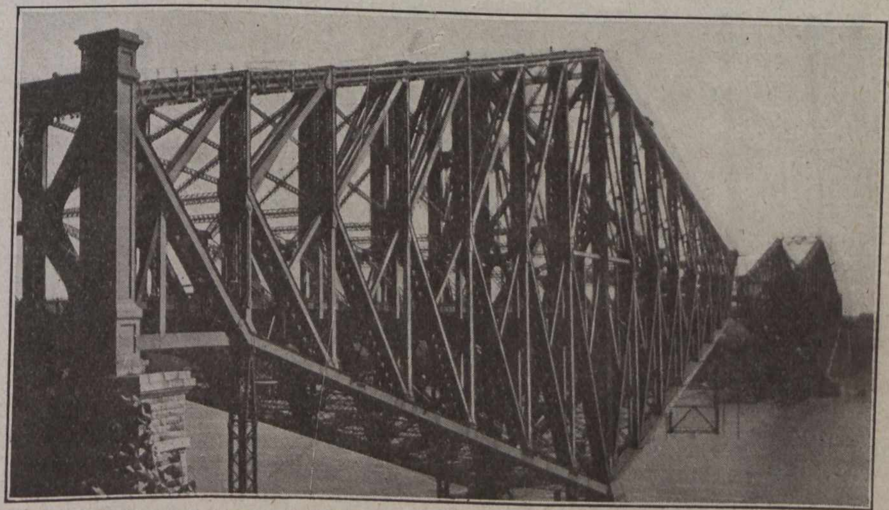


Fig. 11.—This Picture Shows Clearly the K Truss Construction.

All the steam railways in New Zealand are owned and operated by the Government. There are about 3,000 miles of road in operation, and new lines are under construction.

Pitman, Cesar and Company, engineers and manufacturers, 25 Victoria St., London, S.W., England, have dissolved partnership and the firm has been reorganized as Percy Pitman and Co., under the management of Mr. Percy Pitman.

TORONTO-HAMILTON HIGHWAY.

THE following extracts are from a report made to the Toronto and Hamilton Highway Commission by H. S. Van Scoyoc, chief engineer of the commission. This report was made largely as a reply to criticism of the road by the works department of Toronto, and contains many detailed figures that are omitted in the following:—

The probable defects in a concrete highway can be discussed in three groups:—

First—(and most important) Defects in the surface itself. They are serious, as they show the use of unsatisfactory materials or improper methods of mixing, placing or curing.

Second—Cracks. They are no reflection upon the materials or workmanship but are due chiefly to uncertain foundations causing unequal settlement or to insufficient drainage, resulting in damage from frost action. Some cracks have occurred in all concrete roadways so far constructed. They can be repaired at a reasonable cost, but of course the greater the number the greater the cost for maintenance.

Third—Joints. With any type of joint so far made use of, some expenditure for maintenance is to be expected.

In going over the work this grouping has been followed: Joints have been considered as in a satisfactory condition, as requiring chipping, and as requiring chiseling through the slab.

Slabs have been considered as uncracked, as slightly cracked, and as cracked to the extent of requiring to be filled.

Only one surface defect was noted.

Summary of Work Done in 1915.

SLABS.

	Number.	Per cent. of total.
Uncracked	1,751	84.3
Slightly cracked	243	11.8
Need filling	82	3.9
Total laid	2,076	100.0

JOINTS.

	Number.	Per cent. of total.
Satisfactory	1,944	93.9
Need chipping	85	4.1
Need chiseling	42	2.0
Total made	2,071	100.0

Summary of Work After October 15th, 1915.

SLABS.

	Number.	Per cent. of total after Oct. 15, 1915.	Per cent. of total in year 1915.
Uncracked	526	80.4	30.0
Slightly cracked	95	14.5	39.1
Need filling	33	5.1	42.2
Total after Oct. 15	654	100.0	31.5

JOINTS.

	Number.	Per cent. of total after Oct. 15, 1915.	Per cent. of total in year 1915.
Satisfactory	548	83.9	28.2
Need chipping	65	10.1	76.4
Need chiseling	41	6.0	97.6
Total after Oct. 15	654	100.0	31.5

We are willing to admit that joint conditions similar to those found on limited sections of the roadway are undesirable and that they should be prevented. The summary shows quite conclusively that our trouble developed in those sections laid after October 15th. It is open to serious question if any work should be done in this locality after this date. We were tempted last season both on account of the extremely wet weather earlier which had delayed our progress and the unusually mild weather which continued well into November. We were successful in preventing any injury to the surface by the frost. We were not so fortunate in so far as the joints were concerned.

I believe the explanation of the trouble lies in the fact that concrete expands not only with an increase in temperature but also with an increase in the quantity of moisture it contains. It contracts not only with a decrease in temperature, but also as the excess water used in mixing dries out. Ordinarily concrete is laid in warm weather. It contracts in setting up and in addition contracts with the cooler weather during the winter months. It shortens more than enough to provide for the increase in length with the increase in temperature during the following summer.

In our own case the concrete laid late did not dry out on account of the low temperature. In addition a very wet fall was succeeded by an extremely wet, cold spring. The warm weather came suddenly and the concrete expanded before there had been the usual contraction. Even with these conditions we believe that there would have been no noticeable trouble if the joints had been vertical, as the forces on the opposite sides of the joint should have balanced each other.

All of the joints that have been chiseled through are more or less inclined. In practically every case this inclination is in the direction in which the mixer was working. The only exceptions that I have been able to find up to the present have been on the steeper grades, where the tendency of the concrete is to flow down-hill and in the joints made at the end of a day's work where lack of care would make the tendency just opposite to that on the other joints.

It is quite possible that wider joints would have decreased the trouble, but it is worth mentioning in this connection that to have increased the thickness of joint to one-half inch would have entailed an additional expenditure of more than $5,000 for the materials alone. It is to be anticipated that the maintenance on a wide joint will be greater than that on a narrower one. In our own case a very small part of the $5,000 will put the joints in such shape as to cause no serious inconvenience to travel.

Cracks.—I feel that they need no extended explanations. They have been tarred on the approximately seven miles from Oakville bridge to the Blue Dragon at a cost of less than $25 for materials and $175 for labor. This includes any tarring that was done on the joints as well. Four or five more barrels of tar will do the remaining work.

Surface Defects.—If there were signs of this trouble there would be real cause for worry. I believe, however, that both for evenness of surface and quality of concrete our work compares favorably with any work of this kind anywhere.

Six per cent. of the line of a railroad being built in Switzerland will be over bridges and 13.5 per cent. through tunnels.

Recent official statistics place the available water power of Spain at about 5,000,000 horse-power, of which only about 300,000 is being utilized.

TIME TESTS OF CONCRETE.*

By Franklin R. McMillan, C.E.,
Asst. Professor Structural Engineering, Univ. of Minnesota.

(Concluded from last issue.)

Test of a 10-ft. x 10-ft. One-Way Slab.—The data concerning this slab is more complete than any that has yet been obtained. Observations on the steel and concrete were begun when the slab was less than 24 hours old and have been continued to date, embracing a period of two years. Special care was taken in casting the slab and in giving it ample time to cure before loading, as it was desired to know what effect the time and shrinkage would have under the more favorable conditions. As an indication of the quality of the concrete the following tests of auxiliary specimens are quoted:

Compression Tests 8-in. x 16-in. Cylinder.

Age.	Ultimate Strength.
238 days	2,530 pounds per square inch
414 days	2,690 pounds per square inch
415 days	2,950 pounds per square inch

Transverse Tests 4-in. x 5-in. Plain Beams; Age, 414 Days.—Modulus of rupture, average of three tests, 730 pounds per square inch. The modulus of elasticity from compression and transverse tests were about the same, the average of all being 3,330,000.

The concrete was machine mixed in the proportions 1:2:4, using a standard grade of Portland cement, a good grade of washed sand and a crushed limestone ¾ in. to ¼ in. in size. The slab when one day old was covered with sand which was kept thoroughly wet for 30 days. The auxiliary specimens were cured in the same manner. The forms were removed when the slab was 135 days old and no live load was applied until 309 days after casting. The slab is 4⅞ in. thick and reinforced with ⅜ in. round rods 5½ in. on centres, hooked at both ends. Transversely it is reinforced with 5/16 in. rods 12 in. on centres. The centres of the longitudinal rods are 4.35 in. below the top surface of the slab and the transverse rods ½ in. above these. The knife edge supports are placed 10 feet apart and consist of a ½ in. round rod between two plates bearing on the underside of the slab and the top of a concrete pier.

On the usual assumptions the calculations for this slab show stresses as follows: For the live load of 100 pounds per square foot the steel and concrete stresses are 16,000 and 475 respectively, and for the dead load they are 55/100 of these, or 8,800 and 260 pounds per square inch. With the ordinary limiting steel stress, 16,000 pounds per square inch, the slab will support a total load of 100 or a live load of 45 pounds per square foot.

The data from these tests are shown on Plates 7 and 8. On Plate 7 are given the results of measurements of shrinkage and maximum longitudinal steel and concrete deformations. For the period during which the forms were in place only two curves are shown. These represent the shrinkage as measured at numerous points on the surface of the concrete and as measured on the longitudinal steel. During the first 30 days when the top was covered with wet sand the concrete measurements show a swelling, while those on the steel show a shrinkage. From the day when the wetting ceased the shrinkage in the concrete developed very rapidly, exceeding in 20 days that already shown by the steel and being some 30 per cent. in excess on the date the forms were removed.

*Read before the Engineers' Club of St. Louis.

Beginning with the day the forms were removed a separate curve is drawn for shrinkage. This shows a continuation of the shrinkage in the concrete as determined from gauge lines located so as to be free from any effect of the loading. This has the same characteristics as the other shrinkage curves presented and shows a maximum of nearly .0006 inch. per inch, or 0.70 inch per 100 feet. Although this represents the total from the first day, it is somewhat less than values found in most of the other tests.

The steel curve is continued by measurements on the same bars, but from the time of application of the dead load shows the combined effect of bending and shrinkage. The drop of about .00005 inch per inch corresponding to a steel stress of 1,500 pounds, shown on the day the forms were removed, represents the reduction, from the initial compression of 7,500 pounds per square inch, due to the

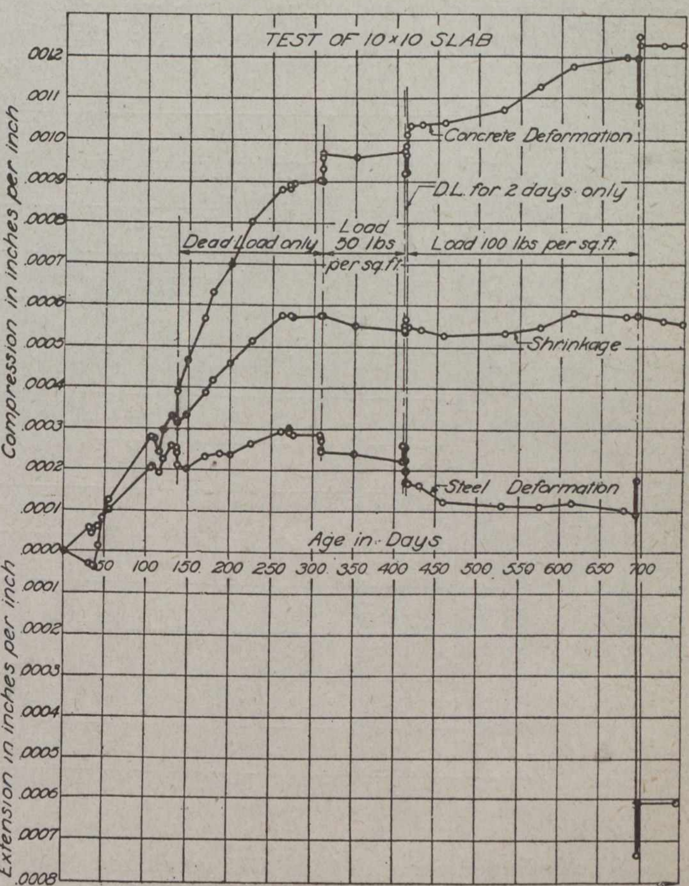


Plate 7.

dead load. It will be observed that this is overcome by the additional shrinkage which continues until a maximum of 9,300 pounds per square inch is reached at the age of about nine months. This maximum is followed by a further reduction of the compression from the application of a live load of 50 pounds per square foot. At the end of the next 100 days, during which there was but slight change in the steel stress, the live load was removed, and after two days was reapplied and increased to 100 pounds per square foot. The recovery of the compression with the removal and its reduction with the reapplication of the load are readily seen on the curve. From here on until the large change near the 700th day, which will be explained further on, there is a gradual reduction of the compression, leaving, however, at the end of two years, a compression of 3,000 pounds; this in spite of the fact that for more than a year the slab had been carrying a total load 55 per cent. greater than it was designed to carry. This means, of course, that the concrete had not broken in tension, and was carrying in addition to the full bending effect whatever stress resulted from the shrinkage. Calculated as a homogeneous beam 4⅞ in. deep, the total stress in the concrete top and bottom is 490 pounds per square inch, considerably less than 730, the strength quoted above for the transverse specimens.

The upper curve, marked concrete deformation, represents the combined effect of bending, shrinkage and time yield in the upper surface at the centre of the span. The effect of dead load, and the live load on and off, can be followed as on the curve of steel deformation. The remarkable time yield under dead load only should be noted. This is represented by the divergence in the curves for shrinkage and total deformation; also the total concrete deformation of .0012 inch per inch, which is six times the sum of the deformations due to the several load increments. These values are very significant when the strength of the concrete and the age before any load was applied are considered, together with the fact that the concrete stress is only about 500 pounds per square inch.

In the calculated stresses n has been taken at 9 corresponding to the value of the modulus of elasticity determined from the auxiliary specimens. The deformations are the results of measurements on the two-inch gauge lines which were crossed by the observed cracks. These were somewhat higher than found for the eight-inch gauge lines crossing the same cracks, as is usually the case due to the effect of the concrete remaining uncracked.

Calculated and Measured Steel Stresses.

Load	Calculated for n=9	Measured Deformations	Def. Times 30,000,000
D. L. to 10,000 concentrated	24,075	.000913	27,390
D. L. to 7,200 concentrated	17,330	.000790	23,700

It will be observed that the measured stress exceeds the calculated by a considerable margin, a fact that is contrary to the usual experience in testing beams. The measurements from short gauge lines to a certain extent account for the higher relative value of measured to calculated stress, but cannot explain how the ratio exceeds unity. This must be explained by the initial compression and the effect of the increase of dead load stress when the

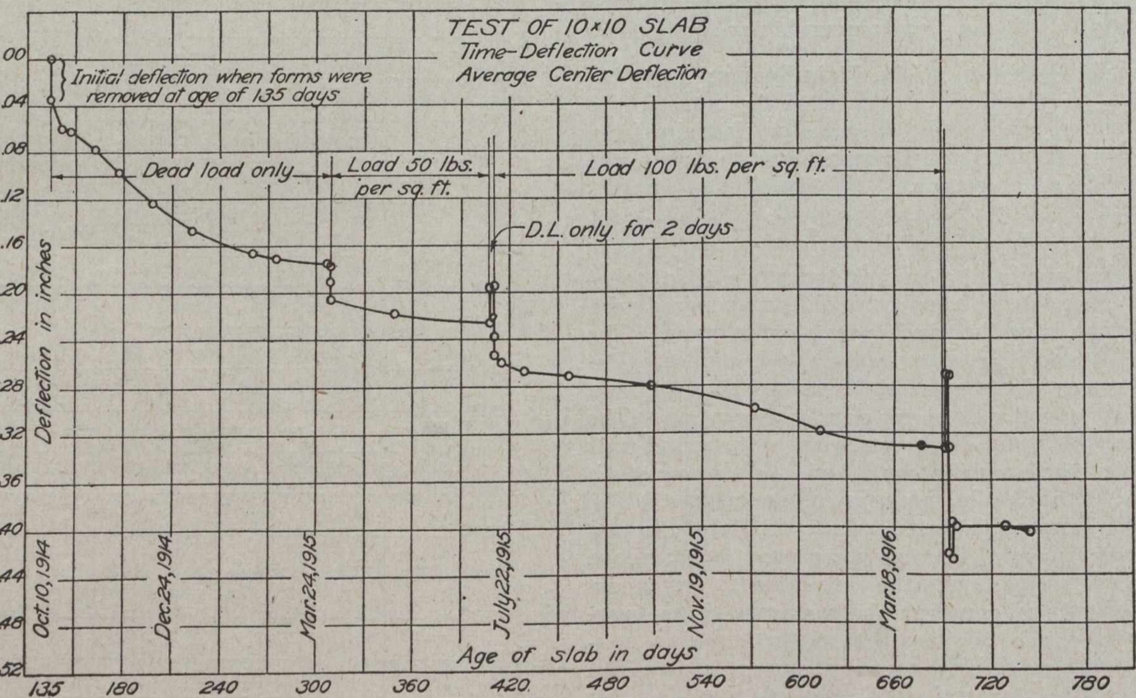


Plate 8.

beam changes from a homogeneous beam with concrete acting in tension to one in which the steel is largely carrying the tension, at least at sections where cracks occur. It is thus seen how the initial stresses due to shrinkage may affect considerably the interpretation of test data. Had this slab been loaded to the point of cracking on the 270th day when the compressive stress was over 9,000 pounds the effect might have been considerably greater. It is the writer's opinion that the stress in tension measured from the original zero should show, and will eventually, the full dead and live load amounts, at least within the limits of exactness with which it may be expected that measured stress will agree with theoretical calculations.

In the simple tests which follow the effect of initial compression on the stress in a beam at first load is brought out very nicely by comparison with a beam in which no compression existed.

The deflection curve in Plate 8 is very interesting in showing the continued sag in this slab with the concrete still unbroken on the tension side.

The large deformations and deflections shown near the 700th day were caused by applying the same total load, 10,000 pounds, concentrated in two rows 45 inches from either support. This was done after removing all live load and allowing the slab to stand one day unloaded. The live load bending moment under this concentration was exactly 50 per cent. greater than that due to the 100 pounds per square foot. After standing two days this load was reduced to a total of 7,200 pounds, leaving a bending moment 8 per cent. in excess of that from the 100 pounds uniform load.

The purpose of this overloading was to crack the concrete in tension and observe the effect of the initial compression in the steel. The following table shows the calculated and measured steel stresses for the maximum live load applied, and for the load of 7,200 pounds concentrated in two rows 45 inches from either support, under which the slab has stood since.

Test of Two Beams Cured Differently.—These two beams made from 1:2:4 limestone concrete were cured for the first two weeks alike under sacks kept thoroughly wet, after which number 124 were left open to the ordinary air of the laboratory and number 125 continued under the wet sacks until tested. They were cast August 5th, and tested December 29 and 30, at the ages of 146 and 147 days. The beams were 4 in. x 5 in. in cross-section and reinforced with one 3/8 in. round rod without hooks, placed 4½ in. from the compression face. Measurements were taken from the day after casting of the shrinkage at both the concrete surface and on the steel.

One detail followed in the making of these beams should be noted because of its influence on the manner of failure. They were cast with the steel held near the top of the beam instead of near the bottom as usual. In setting it was noted that the concrete shrunk away from the rod, which was held rigidly in place, leaving a small crack along the centre for the full length. This crack later became invisible. The low bond strength developed at failure is probably due to the settling of the concrete away from the rod.

On Plate 9 is given the results of shrinkage measurements up until the day of test. From these it will be seen that the steel of both beams deformed together during the first few days and showed a considerable shortening in spite of the fact that the beams were still covered, and the concrete swelling slightly. This may be due to a cooling off from the heat generated during the early hours by the setting. From this point on the steel in the beam cured

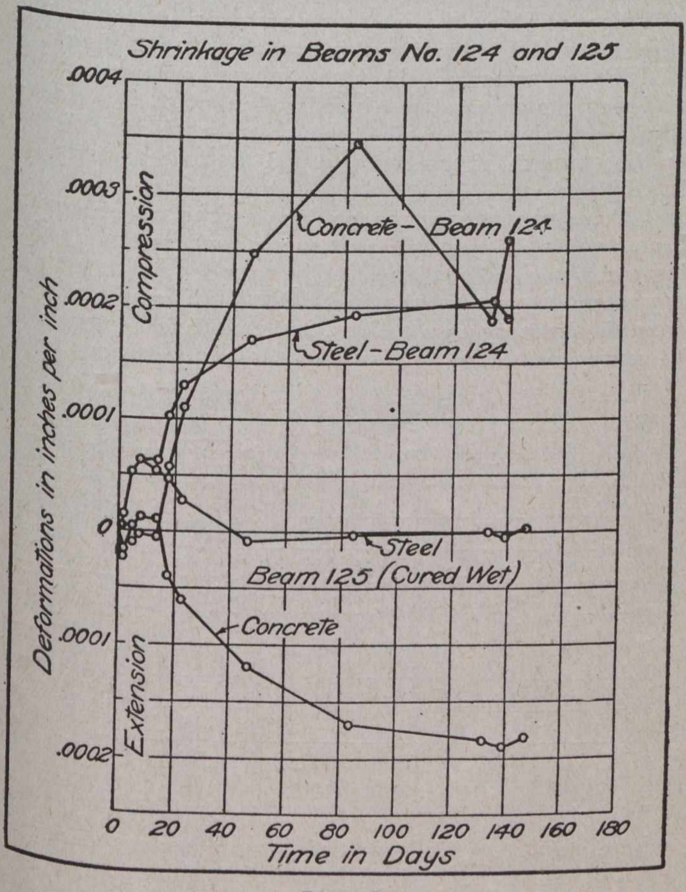


Plate 9.

open to air shows an increasing compression until the last few days before testing. This slight falling off, together with the curious drop in the curve of concrete deformation of beam 124 can only be accounted for as one of those freak incidents that are frequently encountered in this class of work. There are undoubtedly very good reasons for such results but the recorded data and conditions offer no explanation.

The continued swelling in the concrete of beam 125 is followed for a time by the steel but from the 50th day on the steel shows no change. It is impossible to say whether the steel is under no stress because it remains about at the length it had on the second day when zero measurements were made, or whether it has an initial tension due to the swelling beginning from the 8th day; that is, whether it had no stress or an initial tension of 1,900 pounds per square inch. And likewise whether the steel of beam 124 was under 6,000 or 4,100 pounds per

Test of Beams No. 124 and 125.

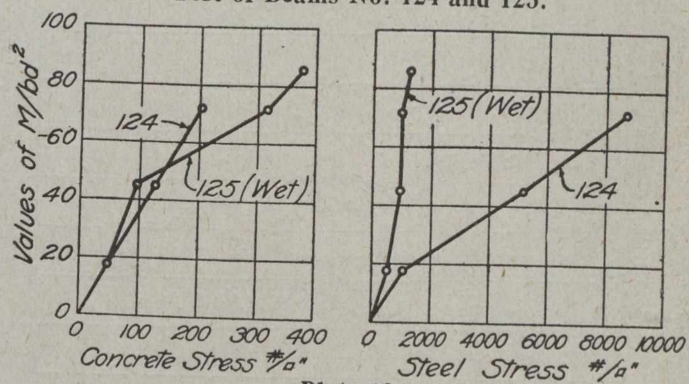


Plate 10.

square inch compression. Either interpretation, however, leaves the steel of beam 124 with a compression of 6,000 pounds relative to that of 125.

These beams were tested on a 43½-inch span loaded at the third points with the results shown in Plate 10. The concrete stresses shown on this drawing were obtained by multiplying the measured deformations by the modulus of elasticity determined from the tests of two 8-in. x 16-in. cylinders cured identically with each beam. These values were 3,120,000 for the dry and 3,870,000 for the wet cylinders. The modulus of the steel was taken at 30,000,000.

It will be noted from Plate 10 that the concrete stresses do not differ greatly but that the steel stress in the dry beam is about eight times that in the wet for nearly all loads. No cracks appeared until the bars had slipped at the ends, in each case. The bars slipped when applying the increments of load next above the last values shown on these curves.

The large difference in steel stress shown by this test is an indication of what might occur in many tests. Reference to this phase of the shrinkage effect has been emphasized here because the writer feels that in many important instances the deduction from test data have been at fault because of this particular feature. Further tests covering this subject are in preparation or under way.

Measurements on Columns in Service.—These results are introduced as they confirm the conclusions to be drawn from all of these tests that very high steel stresses may occur in reinforced columns. The curves of Plate 11 show results of measurements taken on two separate buildings, each curve representing the average of several measurements on a single column.

In the first building the columns were five months old and carrying four floors and the roof when the observations were begun. Only a small amount of dead load, such as partitions, interior finish and equipment has been added since. This is a university building and like others of its class receives a very small proportion of the live load for which it was designed. The results of these observations are given on Plate 11 in the curves marked columns 1 and 2. While these measurements were made at the surface of the column there is no reason to believe that measurements taken on the vertical rods would have shown other results. The steel stress represented by these deformations is 18,000 pounds per square inch, which is very noteworthy when it is considered that practically no dead or live load is included and that at least some of the shrinkage must have taken place during the five months before the gauge lines were established.

The observations on the second building are given in the curves for columns 9, 19 and 20. These were begun when the columns were about two months old and were carrying only a portion of the full dead load, the results, therefore, probably show a considerable portion of the shrinkage as well as some of the dead load stress. For column 20 observations on both the concrete at the surface and on the vertical steel are shown; these, it will be noted, are in practical agreement. The steel stresses from the measurements on these three columns are from 21,000 to 27,000 pounds per square inch with evidence that there will be still some increase. These are basement columns and as the building is still unfinished, have not received a thorough drying out.

Time-Deformation Tests of Reinforced Concrete Columns in Service.

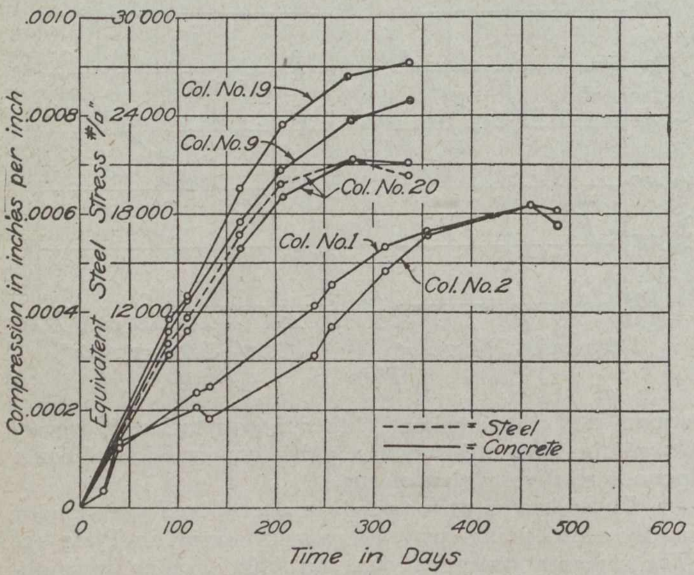


Plate 11.

Test of 5½-in. x 30-in. x 12-ft. Beam.—The results obtained on this beam for a period of nearly two years were rather completely treated in Bulletin No. 3, previously referred to. The deflection curve brought up to date is presented here for the reason that it shows results over the longest period of any of the laboratory tests, and also because the load has been taken off and replaced a large number of times with apparently no effect on the deformations or deflections.

This beam is 5½ in. deep x 30 in. wide and supported on wooden horses over a 12-ft. span. It is reinforced with ⅜-in. round rods, 3 in. on centres, 4⅝ in. below the top. For three years it has been loaded with 1,500 pounds concentrated at the third points. The calculated stresses from the dead and live load are the same and together equal 16,000 in the steel and 650 in the concrete. The concrete was hand mixed, but of good grade, showing an ultimate strength in compression of 2,500 pounds per square inch at one month.

On Plate 12 three curves are shown representing the measured deflections at the centre of the span, the movements of the supports and the difference between these two or the net deflection of the beam. At a few points near the end of the first year the curve of total deflection shows certain vertical lines. These represent the recovery when the load is released. At other points not shown the recovery upon release was of practically the same amount.

On the two occasions noted near the end of the curves the live load was released and replaced a large number of times; as many as 26,600 times during the week of April 20 to 26. As far as can be determined in the few weeks since this was done there is no change in either the deflection or deformations.

Too great importance must not be attached to these results as the stresses represented by this load change are only 325 pounds in the concrete and 8,000 in the steel, and yet such a large number of repetitions of the load after it had been continuously in place for three years cannot help but add some to our confidence in the construction.

What has seemed to the writer as a possible source of danger from these time changes is the possibility that the large deflections may weaken the bond between steel and concrete. The results from these repetitions, while not conclusive, are at least favorable as far as they extend.

Summary.—To summarize briefly the results of these tests the following items should be mentioned: Concrete will shrink an amount equivalent to about 1 inch per 100 feet when exposed to an ordinarily dry atmosphere. Concrete kept continuously wet will show a slight swelling, but once it is exposed to drying conditions shrinking will take place at about the same rate and amount that would have occurred if allowed to dry out soon after casting.

The continuous yield with time in concrete under load seems to go on almost indefinitely though at a gradually diminishing rate. The amount and rate are affected by the strength of the concrete and the load to which it is subjected.

These two phenomena produce a progressive sag in beams and slabs that will continue as long as the shrinking and time yield continues. The sagging makes useless any attempt to predict the final deflection by computations from the ordinary assumptions, and is sufficient in amount to cause unsightly cracking in walls and partitions, and

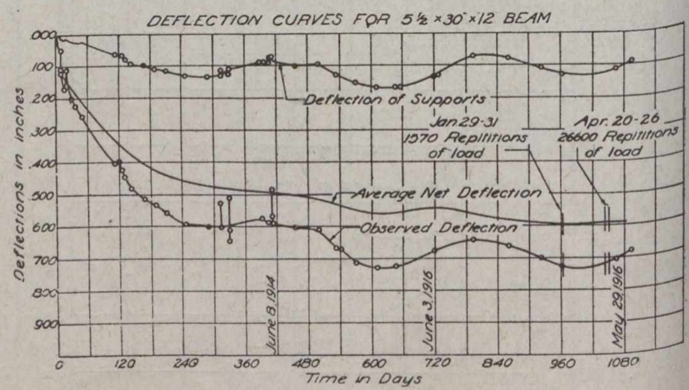


Plate 12.

might, under certain conditions, cause serious derangement of the setting and alignment of machinery.

The initial compression set up in embedded steel due to shrinkage in the concrete may affect materially the interpretation of stresses from test data though it may not affect greatly the final stress condition.

The combined effect of shrinkage and time yield produces steel stresses in reinforced columns far in excess of those contemplated in design. Values of from 18,000 to 24,000 are the minimum that may be expected and it is not too much to say that values above 30,000 may frequently occur.

SURGES IN AN OPEN CANAL.

By Messrs. Karl R. Kennison and Irving P. Church.

Karl R. Kennison, Assoc.M.Am.Soc.C.E. (by letter)—The author's determination of the height of the surge in an open canal, following a sudden interruption of the flow, appears to be based on sound theory. It is particularly interesting to the writer on account of the intimate relationship between this surge and the hydraulic jump, the discussions of which are published with the writer's paper "The Hydraulic Jump, In Open-Channel Flow at High Velocity." The author's treatment of the hydraulic theories involved is complete, and requires little to be said in addition. The same conclusions, however, may be reached in a different way, at the same time bringing out some interesting characteristics of the hydraulic jump which were suggested by reading this paper; also, a formula for the canal surge is submitted herewith which is simpler than that deduced by the author.

There are points of difference between the ordinary hydraulic jump or standing wave and the author's receding wave which at first seem inconsistent, but which are really in agreement. It has already been shown that in an open channel, carrying a certain quantity of water under a certain head, there are only two surface levels at which the water can flow steadily. If the velocity is less than $\sqrt{g \times \text{depth}}$, it is flowing at the upper alternative stage, and, if a dam of the proper height is interposed, it will drop to the lower alternative stage. If the velocity is greater than $\sqrt{g \times \text{depth}}$, it is already at the lower stage, and may jump to the upper stage by meeting either an obstruction, which, if of the right height and smoothness, may avoid all but incidental eddy losses, or a change in channel conditions sufficient to cause the normal jump with its eddy losses, as ordinarily observed. The question may arise: How can the level in a low-velocity canal, which is already at the upper alternative stage, jump any higher on the sudden closing of a gate, even higher than the level of quiet water before its acceleration into the canal entrance, as the author states?

The explanation is that the conclusions previously drawn, with reference to the hydraulic jump, assumed that the jump was in every case stationary, not moving up or down stream. Now, if this standing wave travels along the channel, we may, since velocity is only relative, correct all the velocities by an amount equal to the velocity of the wave, and then the conclusions regarding the hydraulic jump apply correctly to all such moving waves. For example, a suddenly interrupted canal flow, though flowing apparently at the upper low-velocity stage, is approaching the (receding) wave at so high a relative velocity that it is relatively at the lower stage and capable of jumping higher. In fact, when we consider the standing wave or jump as movable along the stream, instead of stationary, there are, instead of two, an indefinite number of possible water levels. It can even be shown that absolutely still water in an open channel can theoretically be made to drop to any lower level or to rise to any higher level by the passage of a standing wave.

This relationship between the hydraulic jump and the surge in an open canal is already clear to one who has followed the author's admirable mathematical analysis. At the risk of some uninteresting repetition, an attempt is made to say the same thing in a different way, and also to show graphically some peculiarities of the hydraulic jump and its relation to the canal surge. The writer has found that elementary diagrams like these are often helpful in getting a clear idea of the subject.

In Fig. 1, two smooth obstructions or dams are assumed to be kept a uniform distance apart and moved along the bottom of a rectangular flume containing still water, with the result that the water level drops, as shown, and rises again to still water, neglecting, of course, friction and incidental eddies. The dimensions are chosen so that direct comparison may be made, if desired, with Figs. 3 to 8 in the writer's paper, "The Hydraulic Jump, In Open-Channel Flow at High Velocity." Higher dams moved at lower velocity would cause a drop lower than shown, and lower dams at higher velocity a drop not as low. The same height of dams, moved much more slowly than shown, would cause only a local depression over each dam. They could not be moved faster without raising the level of the still water ahead, until their velocity is increased to that shown in Fig. 2. Then the water would

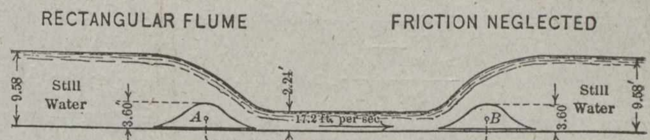


Fig. 1.—Dams A and B moved through still water.

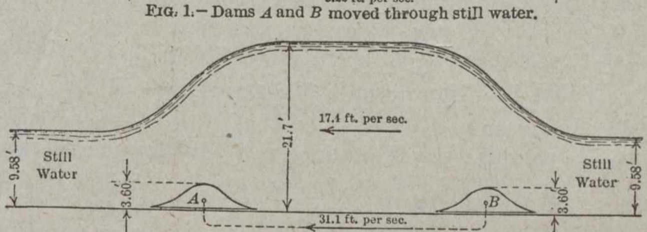


Fig. 2.—Same as Fig. 1, except that A and B are moved about six times as fast.

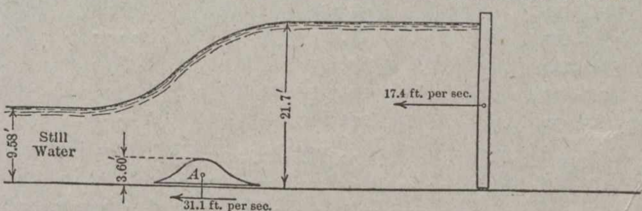


Fig. 3.—Same as Fig. 2, except that B is replaced by gate moving about half as fast.

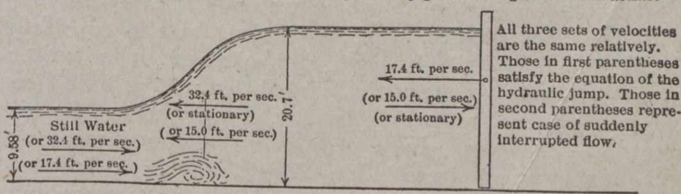


Fig. 4.—Same as Fig. 3, except that A is removed so that jump, instead of occuring without loss of head, contains the normal jump losses, the surface is not lifted so high, and the standing wave travels faster.

rise theoretically as shown and drop again to still water, neglecting friction and incidental eddies, which, of course, would actually be considerable at this velocity. In Fig. 3, one of the dams is replaced by a gate; and, in Fig. 4, the other dam is removed, resulting in a case exactly similar to the suddenly interrupted canal flow described by the author.

All velocity being relative, the absolute velocities in Fig. 4 are also expressed relative to the velocity of the standing wave, illustrating the normal hydraulic jump, and also relative to the gate, illustrating the canal surge. These four figures are not necessary to show this relation, but they may prove interesting in a study of the hydraulic principles involved.

It is apparent, therefore, that to find a general expression (neglecting friction) for the resulting depth, D, in a channel of rectangular cross-section in which the water, flowing with depth, d, and velocity, v, is suddenly checked, it is merely necessary to take Professor Unwin's formula for the hydraulic jump,* which is in excellent agreement with experiment and is apparently based on sound theory, and substitutes for the velocity before the jump its value in terms of the difference in velocities before and after the jump. The result checks exactly with the equation deduced independently by the author. Since this is a cubic equation, it cannot be solved easily, except by trial. The following simple equations will probably be found more convenient. They are not mathematical equivalents of the Unwin quadratic equation for the hydraulic jump and the author's cubic equation for the canal surge, but, as shown by Table 1, for all reasonable uses, the error is well within the precision attainable in hydraulic computations of this nature. d is the depth, in feet, and v is the velocity, in feet per second, in a rectangular (frictionless) flume. D is the depth after the jump.

For the hydraulic jump or standing wave:

$$D = \frac{v\sqrt{d}}{4} - 0.45\,d \quad \ldots\ldots\ldots\ldots\ldots (1)$$

For the receding wave caused by sudden interruption of flow:

$$D = \frac{v\sqrt{d}}{5} + 0.99\,d \quad \ldots\ldots\ldots\ldots\ldots (2)$$

Table I.—Approximate Values of $D \div d$.

By the Unwin or Johnson Formulas.	By Equation (1).	By Equation (2).
1.20	1.18	1.21
1.40	1.39	1.41
1.60	1.60	1.60
2.00	2.00	1.97
4.00	4.03	....
6.00	6.05	....
10.00	10.07	....
20.00	20.10	....

Irving P. Church, Assoc.M.Am.Soc.C.E. (by letter).—As regards the very interesting problem involved in this paper, on the surge produced in a nearly level, open canal of rectangular section, in which water is initially flowing with uniform velocity, when a vertical gate is suddenly dropped and completely closes the channel, the writer recalls no prior treatment in English, except as found on the last page of an article by Mr. Ford Kurtz. In his treatment, however, in applying the method involving "change of momentum" Mr. Kurtz inadvertently used the mass of the flow per second of the water approaching the advancing surge and as yet unaffected by it, instead of the mass suffering impact per second, in forming his expression for the rate of change of momentum; that is (in his notation), he wrote $\frac{\gamma b h_1 v_1}{g}$, instead of $\frac{\gamma b h_1}{g} \times \frac{l_1}{t}$. Had he used the latter expression, he would have arrived at a result identical with Mr. Johnson's Equation (1).

Turning to French sources: Flamant gives the following demonstration (here modified for a channel of rectangular section and constant width, b, and with some of the notation of the present paper). See Fig. 5.

Let AB be a rigid vertical plate, or gate, entirely closing the end of a straight horizontal channel of rectangular section and constant width, b, which contains water at rest, extending indefinitely to the right, and of depth, d. The plate is at right angles to the sides of the channel. Let the plate now assume instantaneously a velocity of U feet per second toward the right, this velocity being then maintained at constant value. The water near the plate acquires (in small instalments) the same velocity, U, and is heaped up in front of it with a flat top and constant depth, D, the front edge, $C''C$, of this wave (wave-front) moving toward the right with some constant velocity, V. Let $D - d$, or the height of the wave, be denoted by h. If γ is the weight of a cubic unit of water, the total horizontal pressure (above atmospheric) between the plate and the raised water is $\frac{\gamma b D^2}{2}$; and, similarly, that between the vertical face, CE, of the water just under the wave front and the (as yet) stationary water on its right is $\frac{\gamma b d^2}{2}$.

Let us now trace the motion for the first second of time. At the end of this second the plate is at $A'B'$, having moved a distance $\overline{AA'} = U$, the wave front is at C, at a distance $\overline{AC} = V$, from AB, and the parallelopiped of water $A''C''CEB'A'A''$ has a velocity, U (the water on the right being still at rest); while at the beginning of this second, the plate being at AB, this same mass of water formed the parallelopiped, $ACEBA$, and had a velocity of zero. The change of momentum brought about in this mass during the first second, therefore, is $\frac{\gamma V b d}{g}(U - 0)$, which is also the rate of change of momentum, since the time concerned is a unit. Hence, equating the sum of the horizontal components of the external forces

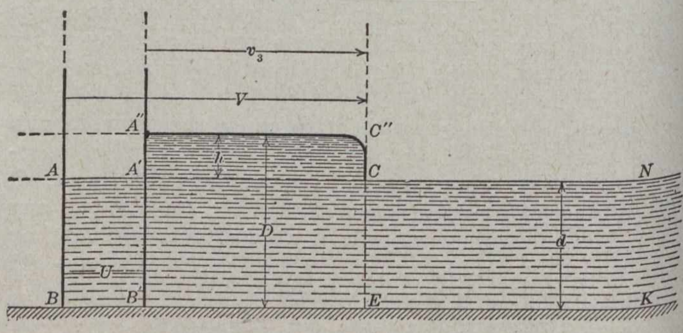


FIG. 5.

to the rate of change of momentum in a horizontal direction (neglecting friction on the bed), we have

$$\frac{\gamma b D^2}{2} - \frac{\gamma b d^2}{2} = \frac{\gamma V b d}{g} U \quad \ldots\ldots (2)$$

Water being incompressible, we have also: volume, $AA'B'B$ = that of the horizontal "lamina," $A''C''CA'A''$; whence

$$Ud = (V - U)\ (D - d) \quad \ldots\ldots\ldots\ldots (3)$$

or,

$$UD = V\ (D - d) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3a)$$

that is, replacing $D - d$ b Flamant's symbol, h (height of wave), we may also write

$$UD = (V - Uh \quad \ldots\ldots\ldots\ldots\ldots\ldots (3b)$$

and

$$UD = Vh \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3c)$$

Eliminating U by means of Equations (2) and (3b), and writing h for $D - d$, we may solve for V, obtaining

$$V = \sqrt{g\left[d + \frac{3}{2}h + \frac{1}{2} \times \frac{h^2}{d}\right]} \quad \ldots\ldots\ldots (4)$$

or, approximately, since the last term in the bracket is generally quite small compared with those preceding,

$$V = \sqrt{g\left[d + \frac{3}{2}h\right]} \quad \ldots\ldots\ldots\ldots (4a)$$

*Proceedings, Am.Soc.C.E., for February, 1916, p. 292, Fig. 35; or Transactions, Am.Soc.C.E., Vol. LXXX., p. 410.

or, again, expanding $\left[1 + \frac{3}{2} \times \frac{h}{d}\right]^{\frac{1}{2}}$, and retaining only the first two terms of the converging series (that is, neglecting $\left(\frac{h}{d}\right)^2$ and higher powers as compared with the first two terms), we derive another approximate relation, *viz.*:

$$V = \left[1 + \frac{3}{4} \times \frac{h}{d}\right] \sqrt{g d} \ldots\ldots (4a)'$$

Flamant does not attempt to solve for the wave height, h, in terms of U and d, but is principally interested in the value of V, the velocity of wave propagation; but, if we substitute in Equation (2) the value of V derived from Equation (3a), there is obtained

$$\frac{D^2 - d^2}{2} = \frac{D\,d}{D - d} \times \frac{U^2}{g} \ldots\ldots\ldots\ldots (5)$$

a cubic in D, if D (and finally h, $= D - d$) is sought; U and d being given.

Leaving Flamant's demonstration, it is noted that, relatively to the moving plate, the still water on the right of CE has a velocity toward the plate (call this velocity v) equal to U; and, similarly, that the velocity (call it v_3) of the wave front $C''CE$, relatively to the moving plate, is AC, that is, $V - U$, away from the plate. It follows, therefore, that in case the water in the open channel, with depth, d, has originally a velocity of v feet per second toward the left and the plate is suddenly dropped, so as to block completely the flow toward the left, and remains fixed in that position, the values of the wave-front velocity, v_3, and of D (or of wave height, h), become determinable by simply substituting v for U, and $v_3 + v$ for V, in the preceding equations. v_3, therefore, is the velocity of the wave front, or surge, and D is the depth of the (now motionless) water between the plate and the wave front. This conception, by which the solution of the surge problem may be based on that of the plate advancing against still water, is introduced by the French engineer, Bazin, in a report (to be referred to later) on the experimental investigation of both cases.

In this way, then, we obtain for the problem of the surge wave dealt with by Mr. Johnson,

$$\frac{D^2 - d^2}{2} = \frac{D\,d}{D - d} \times \frac{v^2}{g} \ldots\ldots\ldots\ldots (6)$$

(which checks the author's Equation (1)), and

$$v_3 = \sqrt{g\left[d + \frac{3}{2}h + \frac{1}{2} \times \frac{h^2}{d}\right]} - v \ldots\ldots (7)$$

In these equations, and also in all subsequent formulas, the author's notation is used, with the addition of $h = D - d$. We may also set down for this case the approximate relations (see Equations (4a) and (4a)'):

$$v_3 = \sqrt{g\left[d + \frac{3}{2}h\right]} - v \ldots\ldots\ldots (8)$$

and

$$v_3 = \left[1 + \frac{3}{4} \times \frac{h}{d}\right] \sqrt{g d} - v \ldots\ldots (9)$$

As regards solving separately for the height of surge, h, or $D - d$, we may write $v_3 + v$ for V, and v for U, in Flamant's Equation (2), combine with Equation (9), and then solve the resulting quadratic; whence, if $v \sqrt{\frac{d}{g}}$ be denoted by k, there results

$$h = \frac{1}{4}\left[\sqrt{16 d^2 + 8 k d + 9 k^2} + 3k - 4d\right] \quad (10)$$

as a fair approximation when h is small compared with d.

We have also from Equation (3b)

$$vd = v_3 h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10a)$$

In his report* to the French Academy of Sciences on an experimental investigation of the propagation of waves in open channels, Bazin gives the data and results of Bidone (1824) in the same field as well as Darcy's (1856). Bidone's experiments were performed on a very small scale, his channel being only about 2 ft. wide and about 40 ft. long in which to create and observe the wave phenomena; whereas Darcy and Bazin made use of, not only an experimental channel about 6.5 ft. wide and more than 1,000 ft. long, with depths of water of 2 ft. and less, but also of a straight reach of a navigation canal, some 3,000 ft. long and 30 ft. wide, with depths as great as 3 ft., as well as of a smaller basin 20 ft. wide.

Such being the fairly large scale on which the Darcy experiments were made, it has always seemed remarkable to the writer that Bidone's results should be quoted so frequently in American books on hydraulics, with little or no mention of Darcy's (in this field of wave motion, standing waves, etc.).

According to this report of Bazin's, Bidone derived the following relations from his experiments with surges going up stream and caused by the abrupt closing of a transverse gate in moving water, channel rectangular, *viz.*:

$$h = \frac{2d + h}{h} \times \frac{v_2}{2g} \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

which may be written

$$h = \frac{v^2}{2g}\left[\frac{1}{2} + \sqrt{\frac{4gd}{v^2} + \frac{1}{4}}\right] \ldots\ldots (12)$$

and

$$v_3 = \frac{v\,d}{h} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

As to the formulas based by Bazin on the Darcy experiments, let us first note that in 1844 J. Scott Russell presented at the Fourteenth Meeting of the British Association for the Advancement of Science, an account of his interesting experiments on the "wave of translation" in still water. These were made on quite a small scale, in a channel of rectangular section, and led to the formula, $V = \sqrt{g(d + h)}$, for the velocity of the wave. (Compare with Flamant's Equation (4a)). This result was verified very satisfactorily by the Darcy experiments on the same phenomenon, these being made on a much larger scale. Bazin, therefore, adopted this formula, modified to suit the altered conditions, as a foundation for an expression for the velocity of wave propagation for the case now under discussion (sudden complete closing of a gate across a rectangular channel containing moving water). That is, he first writes

$$v_3 = \sqrt{g(d + h)} - v \ldots\ldots\ldots\ldots\ldots (14)$$

and then eliminates h by the aid of the relation,

$$h\,v_3 = v\,d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

see Equation (10a). The resulting cubic in v_3 can be factored, and yields one positive root, *viz.*:

$$v_3 = \sqrt{g\left(d + \frac{v^2}{4}\right)} - \frac{v}{2} \ldots\ldots\ldots\ldots (16)$$

But, experiment showing a somewhat larger value for h in terms of v_3 than as given by Equation (15), he modifies the algebraic relations on that basis and finally obtains, as fairly justified by experiment,

$$v_3 = \sqrt{\frac{v^2}{4} + g d} - \frac{2}{5}v \ldots\ldots\ldots\ldots (17)$$

and, as an average,

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*"Recherches Hydrauliques"; by Darcy and Bazin, Deuxième Partie, Paris, 1865.

$$h = \frac{1.2 v d}{v_3} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

As it may be of interest to compare the results obtained from these various formulas, let us take the data: $v = 4$ ft. per sec., and $d = 4$ ft., to find both v_3 and h (that is, $D - d$). The results are shown in Table 2.

Table 2.

Equation from which derived.	h, in feet.	v_3, in feet per sec.
Equation (6), Johnson	1.518	.....
(7) "	.....	10.53
(8)	.....	10.22
(9)	.....	10.56
(10)	1.52	.....
(12), Bidone........	1.544	.....
(13) "	.....	10.39
(17), Bazin.........	.....	9.92
(18) "	1.93	.....

Bazin calls attention to an important fact which escaped the attention of Bidone: that the height of wave at the wave-front itself is somewhat greater than that of the portion of water behind. Bazin's formula seems to provide for this.

[In *The Canadian Engineer* for June 22nd, 1916, there was published an article on the above subject by R. D. Johnson. In that article the author pointed out a rational theory upon which to base research into the rise of water in a canal, following an interruption of flow, due, for example, to a shut-down of a water power plant. It called attention to the analogy between this surge and the phenomenon known as the "hydraulic jump." This article was taken from the Proceedings of the American Society of Civil Engineers for May, 1916. The present article is the report of a discussion of Mr. Johnson's paper as it appeared in the proceedings of the same society for August, 1916.—EDITOR.]

COBALT ORE SHIPMENTS.

The following are the shipments of ore in pounds from Cobalt Station for the week ended September 8th,—

Trethewey Mine, 42,800; McKinley-Darragh-Savage Mine, 95,671; Dominion Reduction Company, 87,000; Beaver Consolidated Mine, 70,213; Nipissing Mining Company, 263,926. Total, 559,610 pounds, or 279.8 tons.

From Elk Lake—

Miller Lake O'Brien Mine, 40,000 pounds.

The total shipments since January 1st, 1916, now amount to 21,445,565 pounds, or 10,722.7 tons.

Much interest attaches to the invention by Mr. R. W. Strehlenert, a Swedish engineer, of sulphite coal from the residue of sulphite wood pulp. A company has been formed in Norway for the exploitation of the invention, with a capital of £90,000. The residue from sulphite manufacture is large, as the paper pulp only accounts for about 45 per cent. of the original weight of the wood. The balance has so far been neglected, with the exception of that portion employed in the sulphite spirit manufacture. In this only 1½ per cent. is utilized, so that more than 50 per cent. of the total weight of the wood remains unused. In the Strehlenert process the product is a coal or coal powder, of a heating value of 12,400 B.Th.U per lb. It is calculated that the production of cellulose in Sweden and Norway amounts in the aggregate to about 1,000,000 tons per annum, of which about three-fourths comes from Sweden and one-fourth from Norway. The first works are being commenced at Greaker, between Sargsborg and Frederikstad, with an initial capacity of some 6,000 tons annually, the quantity of coal which the Greaker Cellulose Works use in the year.

APPOINTMENT OF ALIEN ENGINEERS.

The secretary of the Canadian Society of Civil Engineers has forwarded a circular to all members of the society, protesting against the appointment of alien engineers, and particularly citing the appointment of Prof. Swain as consulting engineer to the Railway Board of Inquiry.

A reply form is attached to the circular, to be signed by the member and returned to the secretary, stating to what Federal Representatives, and upon what dates, the member has written regarding the matter; also stating with whom the member has influence at Ottawa; and also advising of any instances of unjustifiable engagement of outside engineers that may have reached the member's attention.

The complete text of the circular letter, which is dated at Montreal, September 7th, and which is signed by Prof. C. H. McLeod, is as follows:—

The Council of the Canadian Society of Civil Engineers desires to call your attention to a matter of vital interest to the Society, and requests your personal action therewith for the benefit of the civil engineers of Canada.

The Canadian Government, as you are aware, recently appointed a commission to advise upon certain phases of the railway situation of the country. It is understood that the government wishes to determine whether it should continue to assist private ownership as in the past by additional loans, or take over for itself the ownership and operation of certain railways, or allow them to go into receivership. The commission consists of Mr. A. H. Smith, general manager of the New York Central Railway; Sir Henry Drayton, chairman of the Railway Commission of Canada; and Sir George Paish, financier, London, England.

The above commission immediately appointed an American engineer as its advisor, and instructed him to organize a corps of engineers for valuation and advisory work. We wish to record our strong condemnation of the policy of placing in the hands of aliens the engineering work of a commission appointed by the Canadian Government to investigate Canadian railways for which the Canadian community has paid.

The inferences to be drawn from the employment of aliens in the above connection are that the Federal Government considers:

1st. That the Canadian engineers who built the railways are not competent to report upon them.

2nd. That the Canadian universities, in many cases enjoying government subsidies, are not producing competent engineers; and

3rd. That the Canadian Society of Civil Engineers, although embracing a membership of about three thousand, is not considered worthy of consultation on an important engineering question.

The above-mentioned appointment of alien engineers is not by any means the first of its kind, as many similar but possibly less flagrant cases have preceded it, and it is not improbable that the recurrence of such appointments may be due to the fact that Canadian engineers neither assert themselves nor demand recognition.

In order to impress upon the Federal Government the fact that one of its first duties is to encourage and develop the engineering profession in Canada in every possible way, the Council has selected this gross violation of a vital principle to initiate a campaign and impress the fact that Canadian engineers must receive due consideration. The Canadian railways, canals, public works and other engineering attainments are a proof that Canadian engineers stand in the front rank, and it should be quite unnecessary for them to have to appeal to their own government for recognition.

It may be argued in support of the present alien appointment that Canadian engineers are not acceptable because many have been in the employ of the railway companies. To this we would reply that, as the commission itself is to advise the government, basing itself upon the engineering data given to it, any experienced engineers are competent to collect and submit the necessary information to the commission.

It may also be argued that the government gave the commission a free hand in the appointment of an engineering staff, and, since this freedom of appointment is essential, our protest should be to the commission itself. To this we reply:

1st. That when a particularly flagrant case arises such as this where competent constructing and operating engineers are passed over in favor of alien engineers, the question of the suitability of the commission appointed by the government for the work in hand comes into question.

2nd. A protest to the commission itself would probably prove futile, and, even if successful, would not in any way impress the government, the creator of commissions, in regard to future procedure in matters of this kind.

The Council of the Canadian Society of Civil Engineers, therefore, ask you to use your influence in every way to diffuse a knowledge of this matter throughout your community, and to place before those with whom you may come in contact the facts of the case and the position of engineers in relation thereto. specifically, the following is recommended:

1st. That you write to your representative in the Dominion Parliament, whether Government or Opposition, setting forth the facts of the case in a strong way, and pointing out that this is merely an incident in a long course of similar procedures.

2nd. That you write in a similar vein to those having influence with the government in your own community or elsewhere.

3rd. That wherever similar incidents are brought to your attention you voice a protest, giving the facts to your local branch of the Canadian Society of Civil Engineers or to the secretary at Montreal in such a manner that the Council may deal therewith.

4th. That you do not delay acting in this matter as above outlined, but proceed to-day to do what you can to bring every pressure to bear in every direction for the good of the engineering community in Canada.

The Council requests that you will be good enough to fill out and return the accompanying blank to your Branch or to Headquarters if you are not a Branch member, in order that they may be fully aware of what has been done by the membership at large towards strengthening such representation as it may seem desirable to make to the Dominion Government.

SIR HENRY DRAYTON'S REPLY.

In reply to the above circular, Sir Henry Drayton gave out the following interview:—

"Professor Swain, of Harvard University, has had a varied experience in the valuation of railways, not only from the standpoint of the investor, but also from the standpoint of the government inquiry and valuation of the lines which have been followed for the past two years by the Interstate Commerce Commission in its task of making and fixing a valuation of all the railways of the United States, a work as yet not undertaken in Canada. Professor Swain's experience in this connection is unique.

"The instructions to the commission are that the investigation and report should be made at the earliest possible moment. In view of this it was essential that an engineer of the greatest experience in such matters should be employed.

"Professor Swain has, therefore, been engaged, and with him W. H. Chadbourn, who has acted in past inquiries as office assistant for Professor Swain, and who is familiar with the proper methods to be adopted and information to be obtained. He will, in this case, as in others, act as such office assistant, in so far as work in the field is concerned, and all outside work or further assistance which may be required, the commission intends and always has intended to employ Canadian engineers."

RAILWAY BOARD OF INQUIRY.

The Railway Board of Inquiry is constituted under the Inquiries Act of Canada to inquire into the railway situation in Canada. The members of the board are Alfred H. Smith, president of the New York Central Railway Lines; Sir Henry Drayton, chairman of the Dominion Railway Commission; and Sir George Paish, editor of "The Statist," of London, England.

The outline regarding the scope of the inquiry, as submitted by the Prime Minister, was as follows:

1. The general problem of transportation in Canada.

2. The status of each of the three transcontinental railway systems; that is to say, the Canadian Pacific Railway System, the Grand Trunk Railway System (including the Grand Trunk Pacific Railway and the Grand Trunk Railway and their several branches), and the Canadian Northern Railway System, having special reference to the following considerations:

(a) The territories served by each system and the service which it is capable of performing in the general scheme of transportation.

(b) Physical conditions, equipment and capacity for handling business.

(c) Methods of operation.

(d) Branch lines, feeders and connections in Canada.

(e) Connections in the United States.

(f) Steamship connections on both oceans.

(g) Capitalization, fixed charges and net earnings, having regard to (i) present conditions, and (ii) probable future development with increase of population.

3. The reorganization of any of the said railway systems, or the acquisition thereof by the State, and in the latter case, the most effective system of operation, whether in connection with the Intercolonial Railway or otherwise.

4. Generally speaking, all matters which the members of the board may consider pertinent or relevant to the general scope of the inquiry.

SOME FACTS REGARDING PROF. SWAIN.

Dr. George Fillmore Swain was president of the American Society of Civil Engineers in 1913, and is probably best known to many Canadian engineers as the author of the remarkable paper, "The Engineer and the Social Problems," delivered at the Ottawa meeting of the American Society in June, 1913. Dr. Swain's paper aroused great enthusiasm and was widely commented upon as being a masterpiece.

Owing to the interest being taken in him at the time, *The Canadian Engineer* in its June 5th, 1913, issue published the following main facts of his career:—

"George Fillmore Swain was born 2nd of March, 1857, in San Francisco. His father was a prominent citizen of that city and a leading merchant. It was while he served as president of the Chamber of Commerce that he was appointed superintendent of the branch mint during the presidency of Abraham Lincoln. Swain, Jr., received his preparation for college at a military school. When sixteen years of age he became a student at the Massachusetts Institute of Technology. His teacher in civil engineering was Professor John B. Henck.

"In 1871 Mr. Swain received the degree of Bachelor of Science. This was followed by courses of study in Berlin, Germany, where he specialized in bridges, railroads and hydraulics. He returned to the United States in 1880, and shortly after was appointed instructor in civil engineering at the Massachusetts Institute of Technology. He was soon promoted to the position of assistant professor, and a few years later, in 1881, became full professor in charge of the department of civil engineering.

"In 1909 Professor Swain was offered and accepted the Gordon McKay professorship of civil engineering in the graduate school of applied science, Harvard University. In 1893, upon the organization of the Society for Promotion of Engineering Education, he was appointed the second president of that society. His publications include Notes on Hydraulics and also on Structures, for the use of his classes. In 1887 he contributed a paper to the American Society of Civil Engineers on "The Calculation of Stresses in Bridges for Concentrated Loads," which paper has had a very marked effect upon present practice so far as structural computations and investigations are concerned.

"Professor Swain has served on a number of different commissions, among them being the Boston Transit Commission, organized for the construction of the Boston subways; the commission appointed to fix the method of eliminating grade crossings in various parts of New England, and has in very many cases been called as an expert in court cases, not only in Massachusetts but elsewhere."

QUEBEC NEW GOOD ROADS LOAN.

One of the chief questions which will be considered at the forthcoming session of the Quebec legislature will be the proposed new loan of $10,000,000 for improvement of highways.

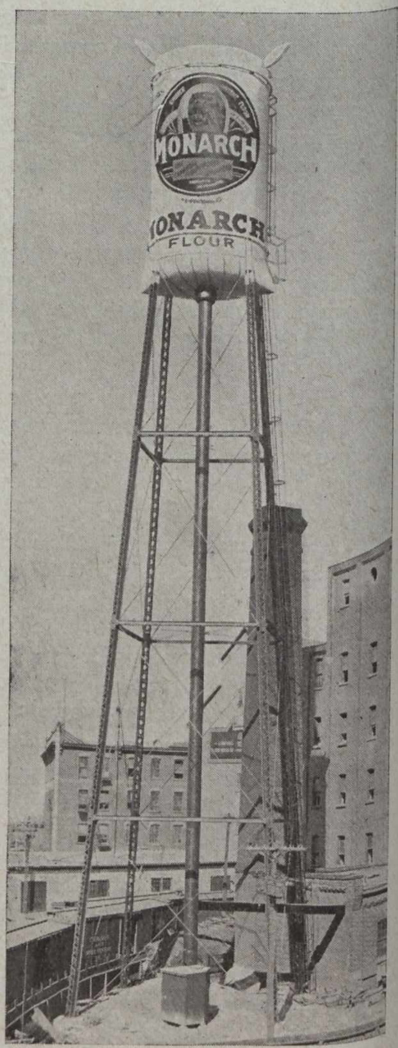
Steel production in France during the second year of the war was double that of the first year, it was shown by statistics submitted at a general meeting of the French Steel Association recently. Eighty per cent. of the pig iron and 70 per cent. of the steel prior to the war was produced in the part of France now invaded.

At the ninth annual assembly of the Royal Architectural Institute of Canada, held in Quebec on the 8th of September, the following officers were elected: J. P. Ouellet, Quebec, president; A. Frank Wickson, Toronto, and W. G. VanEgmond, Regina, vice-presidents; Alcide Chausse, Montreal, honorary secretary; J. W. H. Watts, Ottawa, honorary treasurer.

Armstrong Bros. Tool Co., Chicago, are building a 50 x 70-ft. steel and brick addition to their drop forging department. They are also erecting a reinforced concrete fireproof building, 60 x 130 ft., four floors, to be used for finished stock warehouse, shipping department and offices. These buildings, with new machinery and equipment to be installed, will largely increase the company's facilities.

UNIQUE STEEL WATER TANK.

Sprinkler tanks as advertisements showing the form of the goods advertised are becoming popular in Canada. Two have already been built in Toronto by the Canadian Chicago Bridge and Iron Works of Bridgeburg, Ont., which was the company that introduced this novel type of construction. A tank in the shape of a huge milk bottle was constructed in Toronto for the City Dairy some time ago, and was described in *The Canadian Engineer* in the issue of December 2nd, 1915. Recently another tank of this nature was built for the Campbell Flour Mills, Limited, of Toronto. When the mills were equipped with automatic sprinkler system for fire protection, the company decided to adapt their tank to an advertisement, so the tank was built in the shape of a bag of flour.



Advertising the Product that It Protects.

The prominent position and the great height of the structure renders it very effective for the advertising purpose. The structure is entirely of steel, no difficulty being experienced in forming the metal to any special shape desired. The tank is painted and lettered to represent the sack in which the company's product is marketed, just as the City Dairy's tank is painted to give the appearance of a bottle that is filled with milk having a goodly percentage of cream on top.

The Campbell tank is 40,000 Imperial gallons capacity. The tank is 152 feet above foundations, is 37 feet deep, 18 feet wide, and is oval in cross-section. The supported weight is over 200 tons.

There are 225,000 miles of railroad track in the United States. This excludes municipal traction and interurban lines.

An agreement between Sweden and Russia for linking the railway systems of the two countries by bridging the River Tornea, which forms part of the boundary between Sweden and Russia, has just been ratified, according to a Reuter Stockholm despatch.

The largest railway scale in the world has recently been completed at West Albany, N.Y. It is capable of accommodating a load of 1,650,000 pounds. It consists of six weighing instruments each having a capacity of 275,000 pounds. It is designed for weighing locomotives and other heavy rolling stock.

Editorial

QUEBEC BRIDGE.

The regrettable failure of some portion of the material connected with the erection of the suspended span of the Quebec Bridge, which caused the enormous structure to fall into the St. Lawrence River on Monday, September 11th, has caused national disappointment and especially keen must be the disappointment of the engineers, the contractors and all those who have been most closely allied to the task of completing the bridge.

This disappointment is accentuated when it is recalled that the method of procedure decided upon for the lifting of the span would seem to have been the right one, as it was after the scows had floated away and the hydraulic jacks had commenced to raise the enormous weight up to the desired level that the accident occurred.

Only 48 hours before the span fell a party of bridge engineers from all parts of Canada and the United States had been privileged to go over the structure, had examined carefully and critically the arrangements made for the placing into position of the span and had pronounced them adequate. But again the unexpected happened, and within sight of the linking up of the north and south arms, failure of a casting on the lifting girder at the southwest corner occurred and within a few seconds the span broke loose and lay at the bottom of the river.

While the toll of lives taken was not as great as was the case when the first bridge collapsed in 1907, it took twelve men, and to those left behind a full measure of sympathy is due.

According to several who were on the end of the north cantilever arm at the time the suspended span fell, it was confidently expected that the whole bridge was doomed, but the engineers had built wisely and well. It stood.

The strain which the collapse of the suspended span placed upon the anchor arms and the way in which these arms stood it is eloquent testimony that from a constructional point of view the bridge is all that could be desired. The accident was not due to a fault in construction so much as to a mistake in the method of erection.

The main arms of the bridge having been subjected to such unusual stresses it becomes very important that great care be taken in the examination of these to ascertain just what deflection has actually taken place.

The span which was lost is but one-twelfth of the total weight of the bridge and will speedily be replaced by another. When it is, sufficient care will be exercised in the design and manufacture of the equipment for the carrying and hoisting into position of the structure as to make a repetition of the accident impossible.

It has been stated that the casting that failed had carried the weight of the span for several weeks prior to its launching. While the span was in a reposed, quiet condition at Sillery Cove, the casting was free from the many stresses that must surely have been introduced when the structure was being towed down the river, and while it was being manoeuvred into position before the lifting pins were finally connected.

The courage of the construction company is remarkable. There is no thought of abandoning the enterprise and doubtless greater precaution will be taken when the work of erection of the new span is reached. A greater factor of safety in lifting appliances, and a well-determined knowledge that when built the span will stand the strain of lifting, would seem to be necessary.

TORONTO-HAMILTON HIGHWAY.

Mayor Church of Toronto motored over the Toronto-Hamilton highway this summer. His Worship was severely jolted on various raised joints. Also, he noticed cracks in the concrete. As the city had contributed $250,000 toward the cost of the work, he was naturally perturbed and properly requested Works Commissioner Harris to report on the progress and condition of the road.

Mr. Murray Stewart, the roadway engineer of Toronto, inspected the highway and submitted a report to Mr. Harris, with photographs showing a number of cracks and faulty joints.

In order to prove that the defective slabs described by Mr. Stewart form a very small portion of the work, Mr. H. S. Van Scoyoc, the chief engineer of the Toronto and Hamilton Highway Commission, forwarded a report to Mr. Gooderham which goes into considerable detail. Mr. Van Scoyoc states that out of over two thousand 35-foot slabs laid in 1915, 84.3 per cent. are uncracked, 11.8 per cent. are slightly cracked and 3.9 per cent. are cracked to an extent requiring filling. This is not a serious condition, nor is it by any means unprecedented. Every concrete road that has ever been built has shown a certain percentage of cracked slabs, and in that respect the Toronto-Hamilton highway probably compares favorably with most other concrete highways. A certain small percentage of cracks in concrete roads seem to be inevitable and is probably due to slight inequalities in the foundation and to insufficient drainage. Provided the cracking is not excessive, the openings should in themselves be readily maintained if the road has been properly constructed otherwise.

Mr. Van Scoyoc's report says that of 2,071 joints built in 1915, 93.9 per cent. are satisfactory, 4.1 per cent. need chipping and 2 per cent. need chiseling. This trouble with the joints is a more unusual matter than the cracks in the slabs, and there are undoubtedly more raised joints in the road than the engineers expected. The same difficulty, however, has developed in other concrete roads built on this continent during the year 1915.

The consensus of opinion attributes the trouble to the unusual weather conditions experienced this spring, and the bad joints seem to be confined mainly to work done late in the season.

An inspection of the Toronto-Hamilton highway shows that the work is not subject to any severe criticism provided that another spring does not bring further troubles to light. If the work that was laid this year is in as good condition two or three years from now as it is at the present time, there will be but little reason for complaint. There does appear to be careless workmanship on the joints, many of these not being truly vertical.

This undoubtedly contributed to the trouble by allowing one slab to ride on top of the other.

It is a debatable point as to whether a wider and more expensive expansion joint would have remedied the trouble.

Undoubtedly it would have provided more room for expansion. Also the use of a thicker and stiffer pre-moulded joint might have assisted the workmen in keeping the joints vertical. On the other hand, some engineers maintain that a wider joint merely forms better lubrication for the ends of the slabs and allows them to ride over each other more easily if the ends are in the slightest out of the vertical. Of course, the wider the joint, the more expensive the maintenance, other things being equal.

Some engineers are now laying concrete pavements with only tar-paper expansion joints (at approximately every 35 feet), depending upon the shrinkage to open up the joints sufficiently to allow for subsequent expansion. And on account of the trouble that has been experienced in getting the joints truly vertical and in keeping the slabs from rising, numerous highway engineers in the United States have even reached the point where they are beginning to recommend the construction of concrete pavements without any joints, feeling that the trouble which may result from transverse cracks can be more readily taken care of than the trouble which may result from raised slabs. Of course, the jointless pavement must be well reinforced. But the modern tendency in the construction of concrete pavements, anyway, is to use sufficient reinforcing of good quality.

Whatever may be the future of the Toronto-Hamilton highway, it is certainly a splendid road at the present time, and the bad joints having been chipped, chiseled and repaired, an automobile can travel with safety and comfort—so far as the road itself is concerned—at a speed of even sixty miles an hour. It is, of course, difficult to predict just how the road will withstand the abrasive action of steel wheels and horse shoes, but the surface is well finished and should not be soon affected. However, exclusive of bridges but including all other costs, the highway will have cost in the neighborhood of $30,000 a mile, so that the municipalities involved are naturally entitled to expect a good surface for some years to come, assuming that the repair work is properly done each year.

By the end of this season the entire road, with the exception of bridges, will be finished from Hamilton to Etobicoke. The remaining five miles of the road will not be built until next summer, owing to sewer construction that is going on at New Toronto along the line of the highway. By the time the completed road is open to through traffic, the sections laid this year should demonstrate whether or not the unusual weather of 1915 really was the cause of the joint troubles now being experienced.

PERSONAL.

J. C. RESTON, municipal electrician for South Vancouver, has resigned, having secured another appointment in Northern, B.C.

C. H. DANCER, formerly deputy minister of public works for Manitoba, has been appointed Winnipeg district engineer for the Federal Department of Public Works.

GEORGE A. GUESS, professor of metallurgy at the University of Toronto, has been engaged to start the copper smelter of the Vermont Copper Co., at South Trafford, Vermont.

A. E. PICKERING, manager of the Water and Light Commission of Sault Ste. Marie, Ont., has resigned in order to accept an important position with the Great Lakes Power Company.

S. BINGHAM HOOD, for many years distribution engineer for the Toronto Electric Light Company, has resigned to accept a similar position with the Northern States Power Company, Minneapolis.

DAVID W. JOHNSTON, engineer for the South Saanich municipality, B.C., informed the council that it is his intention to volunteer for service in the Canadian Expeditionary Force, and that he would like to be released from his civil duties in November.

A. HASTINGS, who for eight years has been foreman of the Terra Cotta Pressed Brick Company, of Brampton, Ont., has now severed his connections with that concern and accepted a position as superintendent of the Meaford Brick Company, Meaford, Ont.

W. S. GUEST, of the university staff in applied science, University of Toronto, is attending the special course of lectures in Illuminating Engineering at the University of Pennsylvania, including also a tour of inspection of the illumination in the principal American cities.

G. R. G. CONWAY, M.I.C.E., of Toronto, formerly chief engineer and assistant general manager, and now consulting engineer to the British Columbia Electric Railway Company, is leaving for Mexico City immediately to represent the Bondholders' Committee of the Mexican Light and Power Company, Limited, and the Mexico Tramways Company.

A. P. BROADHEAD has been appointed superintendent of the Drummondville, P.Q., section of the Southern Canada Power Company, with headquarters at Drummondville. Mr. Broadhead was formerly electrical engineer of the St. Lawrence Brick Company, Laprairie, Que., and prior to that assistant superintendent of the Montreal Light, Heat and Power Company.

GEORGE R. ARCHDEACON has been appointed general manager of the Canadian Hart Accumulator Company, the head office of which is situated at St. Johns, Que. Mr. Archdeacon is an associate member of the Institution of Electrical Engineers. He has had over fifteen years' engineering experience, and was formerly upon the staff of Messrs. Ferranti, Limited, and the Chloride Electrical Storage Company, Limited, England.

OBITUARY.

WILLIAM M. MANIGAULT, of Strathroy, Ont., a well-known drainage engineer and a county surveyor for many years, died recently.

JAMES BRADY, pioneer mining and civil engineer, and one of the best-known residents of the Columbia Valley, passed away recently at Wilmer, B.C.

JAMES IRONSIDES, 50 years of age, a member of the contracting firm of Ironsides, Rennie & Campbell, Vancouver, B.C., died recently after a brief illness.

J. J. FRANCIS, P.L.S., died at Sarnia, Ont., on September 13th. Mr. Francis went to Sarnia in 1861, and had been resident there continuously since then. He was 82 years old, and was widely known as an expert land surveyor.