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The Canadian Engineer

A weekly paper for Canadian civil engineers and contractors

Canadian Engineering Has Triumphed at Quebec

Great Central Span of the Quebec Bridge Successfully Floated, Then Hoisted Through Thirty-Four Two-Foot Lifts in Two Days—With New Mooring Tackle Arrangement Ordinary Storms Are Not Feared Now, and There Seems To Be No Reason Why Plan Should Not Succeed

Quebec, P.Q., 11 p.m., September 18th.—Canadian engineering has triumphed at Quebec. The great suspended span of the Quebec Bridge, weighing 5,080 tons as lifted, has been successfully floated to the bridge site, connected to the lifting chains, and hoisted 68½ feet. It now rests for the night, safely moored, within 82 feet of its final level.

The work is progressing smoothly, no accident of any serious account having occurred and only one minor accident which did not in any way affect or delay the lifting.

Not counting the first lift, when the weight of the span came onto the hanger chains only when the hydraulic jacks were within 6 inches of their top positions, there were required seventy-five 2-ft. lifts to hoist the span into its final position. Of these, twelve lifts were accomplished yesterday and twenty-two to-day.

The average time of yesterday's lifts was 16 mins 34 secs.; of to-day's lifts, 13 mins. 35 secs.; of both days, 14 mins. 27 secs.

Last Saturday morning, September 15th, was the date finally determined upon for the floating of the span, provided that the meteorological observatory's reports were favorable. Telephone communication was obtained with the observatory at Toronto at 10 a.m. Friday, and R. F. Stupart, the director of the observatory, reported as follows:—

Fig. 1.—Connecting the South End Hanger Chains to Stub Links of Lifting Girder. Engaging the Leafs Before Driving Lifting Pin

"Centre of low pressure area over North Dakota travelling southeast. Another centre of low pressure off the South Carolina coast. Centre of high pressure over Gulf of St. Lawrence."

The barometer reading at the bridge site was 30.45.

Report from R. F. Stupart at 10 p.m. Friday:—

"We do not expect any more wind than at present during the next fifteen hours and after that it may increase very considerably. Still from northeast. Tropical storm over Atlantic off Carolina coast may move due north."

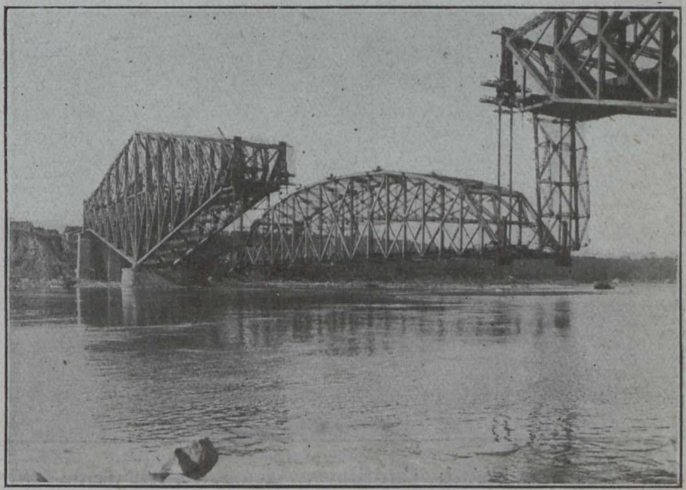


Fig. 2.—The Quebec Bridge at 5.30 p.m., September 17th, 1917

Barometer reading at bridge site, unchanged at 30.45.

As the velocity of the wind at the bridge was over 20 miles per hour, and as it was feared that if the tropical storm were to move due north its effects might be felt at Quebec earlier than desirable, announcement was made about 11 p.m. that the floating would be "postponed on account of uncertain weather conditions."

The judgment of the engineers was justified, even though the threatened tropical storm did not materialize. Saturday proved to be a dark, gloomy day with frequent gusts of high wind and occasional choppy wave action early in the morning.

A conference was held Saturday afternoon to decide whether to attempt to float the span early Sunday morning, even should favorable weather reports be received that night. It was decided to make no attempt under any circumstances to start the work on Sunday, although work would have been continued on Sunday had it been started on Saturday.

The priest of the Roman Catholic Church at Sillery had blessed the span some days previously, and had given his permission to the workmen to continue on Sunday any necessary work. Despite this, however, it was realized that many of the men might have decided objections to a Sunday start, and at the conference it developed that many of the officials had similar objections. And as all agreed that an extra day's practice with the jacks would be desirable, it was unanimously voted to postpone the start until early Monday morning. The men were paid for the fortnight on Saturday afternoon, but were required to report early Sunday for further drill.

At 10 a.m. Sunday the following report was received from R. F. Stupart:—

"During next 24 hours winds will be light. Disturbance on Atlantic coast has almost disappeared and

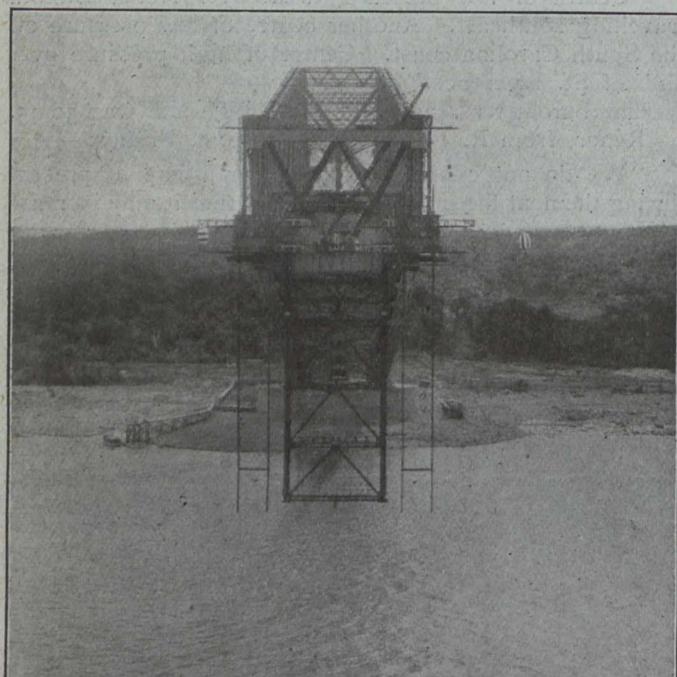


Fig. 3.—The South Cantilever Arm, Taken from the North CLO. Note Balloons Used for Centering Floating Span in Gap

high pressure area dominates weather of nearly whole continent."

Barometer reading at bridge site, unchanged at 30.45.

Report from R. F. Stupart at 10 p.m. Sunday:—

"Light winds and fair weather to-morrow."

Barometer reading at bridge, 30.4.

It was thereupon fully decided to make the big effort Monday morning, and the valves in the scows were closed at 11 p.m. Sunday.

The sun rose strong and there was fine light for all the work on Monday morning. The day was fairly warm, the water very calm, the wind only about 10 miles per hour; in fact, all conditions were ideal and a vast improvement over what would have been encountered Saturday morning, and even considerably better than Sunday, which had been a fine day.

The superintendent of erection, Mr. Fortune, was in charge of all operations until the span floated clear of the falsework at Sillery Cove. The span floated at 5.15 a.m. The bearing shoes were centered and leveled by means of ratchet turnbuckles, the work being completed at 5.40. Five minutes later the span began to draw out. The tug "M. E. Hackett" took hold of the south end, the towing hawser being attached to snubbing blocks on the towing bitts on the outside scow. The tug "Belle of Quebec" took hold of the north end in similar manner.

Fig. 4.—Swinging the Suspended Span Normal to the Current After Floating

The freeboard of the scows when floating was 3 ft. 2 ins. outside, 3 ft. 3 ins. inside.

The span was clear of the outside end bents at 5.54 and one minute later Geo. Davie, of the Davie Shipbuilding Co., of Levis, took command of the span and all the tugs. Mr. Davie is a towing expert. He had charge of the towing of the span up the river last year also, and had again volunteered his services, having shown great interest in the work throughout.

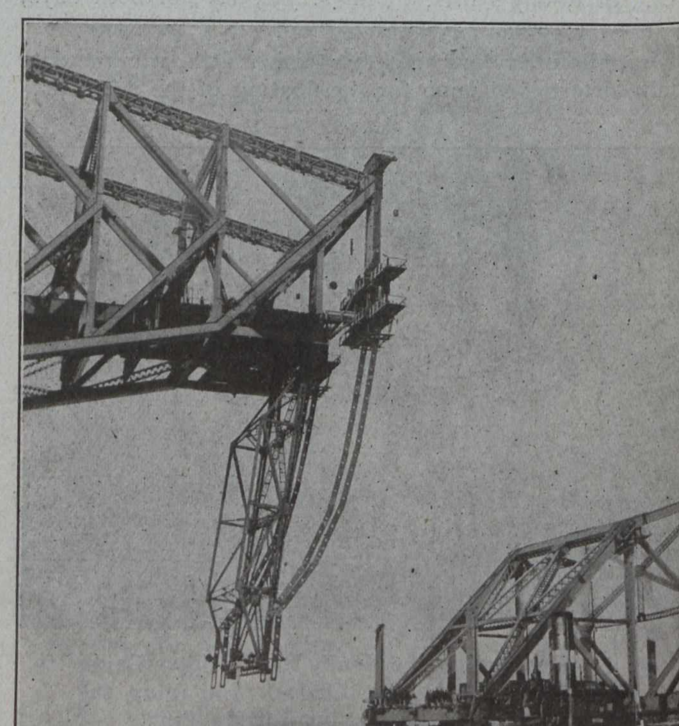


Fig. 5.—Looking West, Showing One End of Suspended Span Just Before Lowering the South Mooring Frame

The span rode very steadily. There was a west wind of five miles per hour and a westward current of four miles. The mooring tackle was cut clear of the end bents when Mr. Davie took charge, and the span was at first drawn out by means of tackle running from a steam hoist situated at each end of the span to the end bents Lo and L18. The larger tugs, "John Pratt," "Virginia," "F. Dupre," "Mathilda," and "Spray," all of the Sincennes-McNaughton Line, of Montreal, were standing by.

The span cut loose from its downstream stone crib anchored at 6.06 and was then absolutely free and normal to the current, which was running westward at four miles per hour.

The tug "Spray" replaced the "Belle of Quebec" at 6.11. The "Virginia" took hold of the middle of the lower chord of the span at 6.12. The "Mathilda" replaced the "M. E. Hackett" at 6.12. The "M. E. Hackett" took hold of the span at the south end, upstream side, at 6.15, the "Belle" simultaneously taking hold at the north end, upstream side. The "F. Dupre" and the "John Pratt" also took hold of the north end, downstream side, as the current was stronger nearer the shore. The span was then normal to the current and directly opposite the erection site at Sillery, and began to float upstream with the current and the aid of the two small tugs which were pulling westward, the five larger tugs on the easterly side of the span pointing downstream and ready to act as a check on the span's progress at any moment.

The "Hackett" and the "Belle" are about 500 h.p., while the other five tugs are about 1,000 h.p.; the towing hawsers used were 4½ ins. The tide was now at full

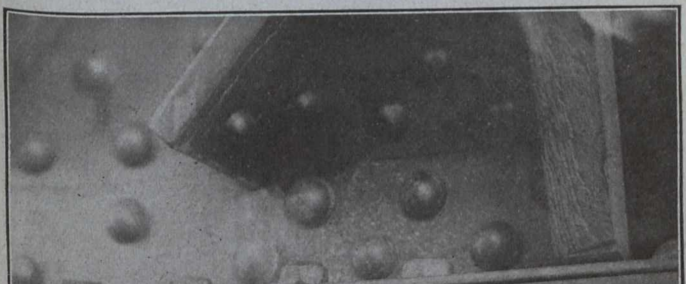


Fig. 6.—Side View of One of the End Bearings Under XLO Joint. Taken while Span Rested at Sillery Last Friday. The Bottom Shoe of the Bearing is the Part Immediately Above the Rope

flood and the tidal current was running upstream with a velocity of from four to five miles per hour. By means of ranges which had been previously placed at measured distances apart along the shore, the rate of progress of the span was recorded. The ranges were poles carrying conspicuous targets, each range consisting of two such poles, one placed at the shore line and the other at a little distance from the shore in a line perpendicular to the course taken by the span. When the observer on the span saw the distant pole exactly behind the one on the shore, the time at that moment was taken as the time of passing that particular range.

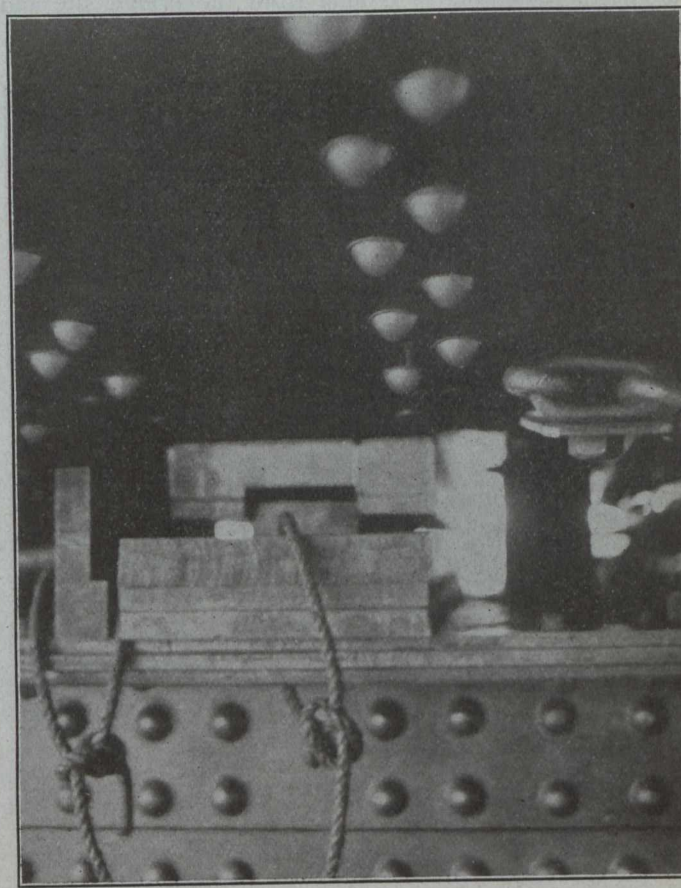


Fig. 7.—Close View of One of the Nickel-Steel Key Bearings

Taken while span rested on the bronze plate bearings at Sillery. The four key-bearings were used only during the lifting of the span, but they then bore its entire weight. The change in these bearings constituted the most important change in design from last year's program. The rocker type of bearing used last year failed, resulting in the loss of the first span. At the left, the rope is looped through the forged steel plate which was later bolted across the end of the bearing to assist in stopping the upper shoe from slipping off the key in case of accident. The right loop of the rope is through one of the bolt holes provided for this purpose.

The first range was passed at 6.23 a.m. The distance from the first range to the second was 1,700 ft., and as the second was passed at 6.29, the speed was about 3.2 miles per hour.

The distance to the third range was 2,560 ft., and it was passed at 6.36, so that the speed between the third and fourth ranges was about 4.1 miles per hour.

From the third to the fourth range was 2,180 ft., and the fourth was passed at 6.45½, so the speed between those two ranges was about 2.6 miles per hour. The span was then about 1¼ miles from the bridge site. The effect of the 10-mile-per-hour west wind was being felt, checking the speed of the span somewhat.

The distance to the fifth range was 2,050 ft. and it was passed at 6.53. Speed, about 3.1 miles per hour.

Distance to sixth range, 1,750 ft. Time of passing sixth range was missed. Distance to seventh range, 620 ft. Distance from fifth to seventh range, 2,350 ft. Seventh range passed at 6.59¼. Speed between fifth and seventh ranges, about 4.2 miles per hour.

Distance to eighth range, 1,000 ft. Passed at 7.02¼. Speed, about 3.8 miles per hour.

Distance to ninth range, 6.20 ft. Passed at 7.05½. Speed, about 2.1 miles per hour. The span was then about three-eighths of a mile from the bridge site and was halted and lined up on the centre of the opening,

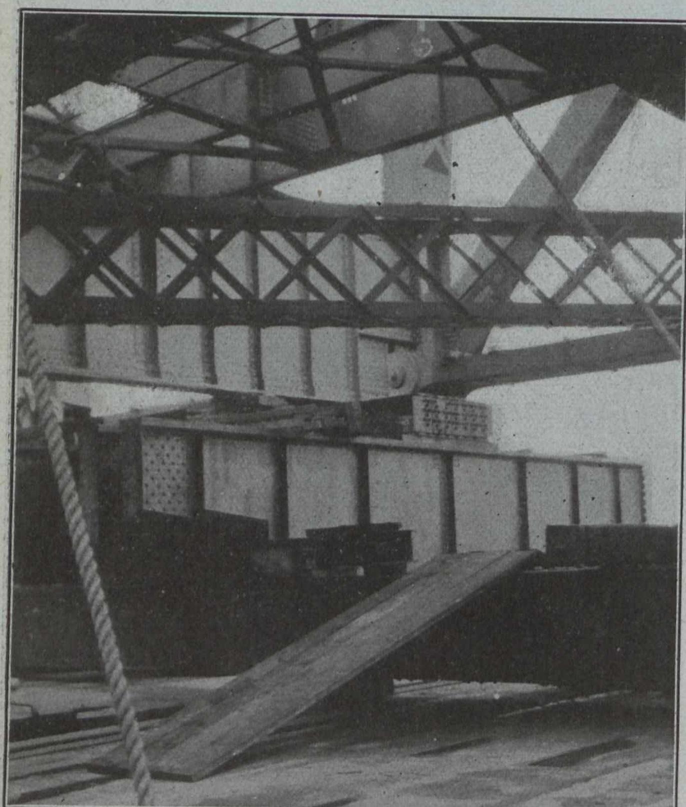


Fig. 8.—View from Floor of Scow While the Span Rested on End Bents, Showing Support for One of the Panel Points of the Bottom Chord, Stringers Under Panel Point Distributing the Load to Girders

which was marked by two balloons hung on lines reaching from the top chords of one cantilever to the top chords of the other cantilever arm. The span was also swung as parallel as possible to the centre line of the bridge.

The distance from the ninth to the tenth range was 1,840 ft., and within that distance the progress of the span was gradually checked, and the span halted at the tenth range, which was only 450 ft. from the bridge site.

A tide gauge had been placed near the north main pier and at the moment of high tide, an observer signalled to a man on the top of the north main pier, and the latter raised a large red and white disc target, signalling to Mr. Davie that the tide had reached its maximum height. When this signal was raised at 7.21, the span was allowed to leave the position where it had been halted opposite the tenth range, and the tidal current, which at the bridge site continues upstream for about an hour after high tide, was allowed to carry the span the 300 feet to the eleventh range in eight minutes, or at a speed of about 0.4 miles per hour. The current was flowing westward with a velocity of about 6 miles per hour.

The centre line of the span was then only about 150 ft. from the centre line of the bridge, and a minute later two ¾-in. wire rope lines were passed out from the south mooring truss and connected to the hoisting engine on the south end of the span, and a few minutes later the same operation was performed at the north mooring truss. The span was then only 75 ft. east of its final floating position. Time, 7.34 a.m.

The 1¼-in. mooring lines were connected—two at each corner of the span—at 7.45, and the span was centered and the mooring lines brought up all tight at 7.55. The current had now slackened to two miles per hour, still flowing westward.

The mooring trusses were lowered to their vertical position at 8 a.m. and the span finally centered and the lifting chains lowered at 8.05.

The connecting of the lifting chains to the stub ends of the lower lifting girders (ELG1) was done at each end by one foreman, two assistant engineers (one in charge of each corner) and twenty men. Besides those forty-six men, Messrs. Johnson, Monsarrat, Mitchell, Duggan, Porter, Davie, Meyers and Atkinson were on board the span throughout its journey up the river, thus making a total "passenger list" for the trip of about fifty-four men.

Telephone connection between each end and the centre of the span was maintained during the trip. Orders to the tugs were shouted through a megaphone. A large platform was built across the centre of the span to enable

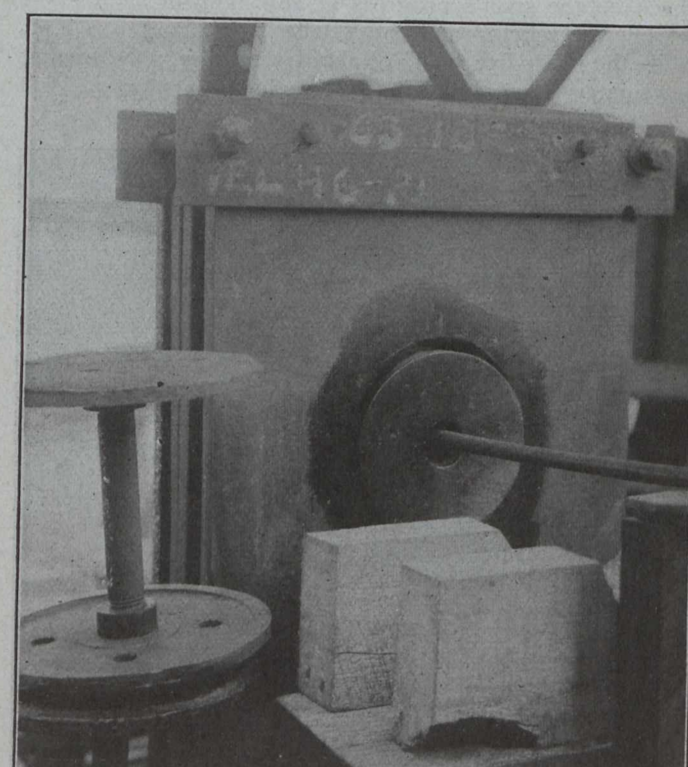


Fig. 9.—Close View of One of the Eight Forged-Steel Lifting Pins Resting in Hole in Stub End of Lifting Girder, Showing Clearance Workmen Had in Driving Lifting Pins. At the Left is a Pair of Caps for the Pin. The Connecting Bolt Goes Through the Centre of the Pin and the Caps Screw on

Mr. Davie to get readily from one side to the other while controlling the tugs.

When the span had been moored and the lifting chains lowered, S. P. Mitchell, the consulting engineer of erection, took charge and directed the placing of the lifting pins.

It required forty minutes to get the lower leafs of the lifting chains to engage with the leafs of the lifting girder stubs, and the chains had to be lowered another two feet in order to comply with the Board's specification that the pins must be driven in the upper third of the slot, so as to ensure plenty of time for driving all four pins before the load would come on any one of them.

The silicon steel plates of the lowest lifting links were found to be warped, so that they would not readily slide between the plates of the lifting girder stubs. This was expected, however, as all rolled plates are somewhat warped, and provision had been made to contend with this difficulty by field methods rather than by shop methods. By means of wedges and crowbars the links were driven into the stubs, the wedges being hammered down between the bars and the platform around the stubs.

At 8.45 all four corners were ready for driving the lifting pins, and at a given signal these were driven simultaneously, and their caps screwed on, the work in all four corners being completed within two minutes of each other. A block and fall from the first joint of each set of links to the end post of the span was used in straightening and holding the lower link.

Slack water was at 8.42. The downstream tugs cast off at 8.40 and stood by to handle the scows. The link tackle was then cut loose, the workmen being hoisted in cages lowered from the cantilevers by locomotive cranes.

The key bearings were finally examined with the draft of the scows at 3 ft. 11 ins. outside and 4 ft. inside, and jacking operations were started after all men had been taken off the span. Col. Monsarrat and Mr. Davie were the last to leave.

The weight of the span did not bear upon the hanging chains until the last 6 ins. of the first 2-ft. lift, on account of the lifting pins having been driven in the top third of the 4-ft. 9-in. slot. The first strain came on the lifting links at 9.32. Jacking started at 9.10.

The scows rose higher and higher from the water as the links relieved them of the weight, and at 10.28, at the end of the second full lift, the scows floated free within a half minute of each other, and were captured by the waiting tugs and taken back to Sillery.

The third lift, which was really the fourth jacking operation after the connection of the span to the lifting chains, was delayed to give the men time for lunch, as they all had very early breakfasts, and it was not completed until 11.55 a.m. An hour of rest was then permitted, the men having been at work since 4 a.m. or earlier.

About 1 p.m. work was started on putting up the tackle for handling the links after they were no longer needed, and at 2.05 p.m. the fourth lift was started.

Following is yesterday afternoon's lifting record:—

Fourth	lift	started	2.05;	finished	2.22
Fifth	"	"	2.23;	"	2.39
Sixth	"	"	2.39;	"	2.55
Seventh	"	"	2.57;	"	3.16
Eighth	"	"	3.16;	"	3.32
Ninth	"	"	3.33;	"	3.50
Tenth	"	"	3.51;	"	4.08
Eleventh	"	"	4.09;	"	4.25
Twelfth	"	"	4.25;	"	4.40

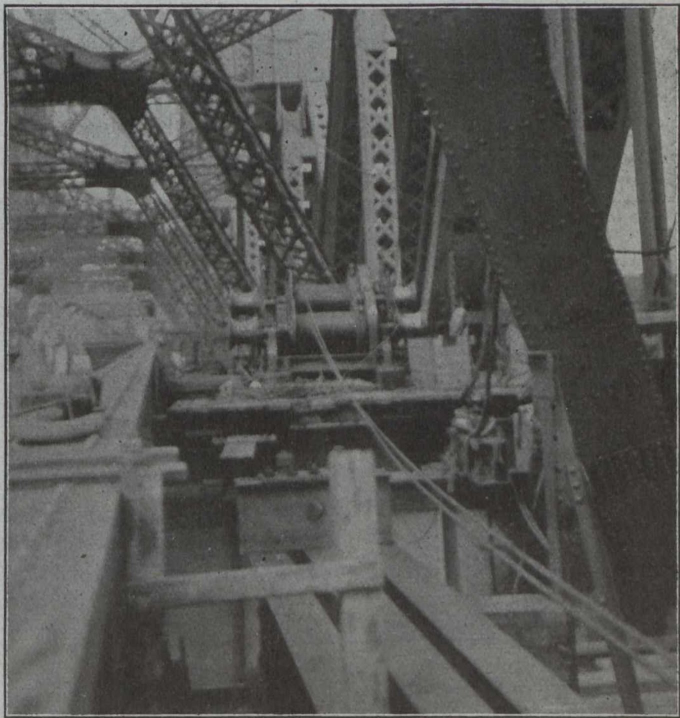


Fig. 10.—Electric Hoist on Cantilever Span Removes Lifting Links as Their Work is Completed

The remainder of the afternoon was spent in removing the mooring lines and putting the anchorage tackle into place for the night to anchor the span against possible heavy winds. The hydraulic jacks were left lowered and the safety screws hard up and all four pins in at each corner, pinning the lifting chains to both ELG2 and ELG3.

To-day the span was raised through twenty-two lifts, or forty-four feet. The record of lifts was as follows:—

Thirteenth lift, 7.42; finished 8.01; time taken for lift, 19 minutes. (NOTE.—In all cases where reference is made in this article to "time of lift," it means the time of the complete cycle of operation of the jacks from the time the lift begins until the jacks are back at the zero, or

(Concluded on page 266.)

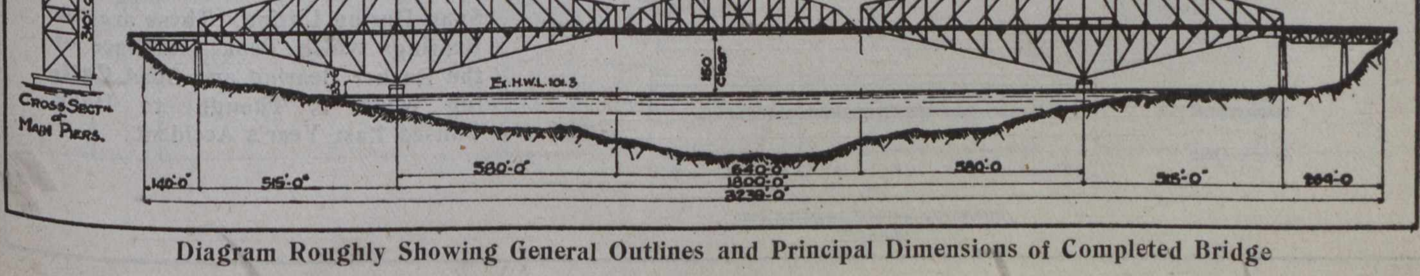


Diagram Roughly Showing General Outlines and Principal Dimensions of Completed Bridge

No Rocker Bearings nor Steel Castings This Year

Between the Lower Lifting Girders and the Suspended Span—Lower Shoe of Roller Bearing Riveted to Girder While Upper Shoe is Riveted to Span and Key Carries the Load During Lifting

By ARCHIBALD JOHN MEYERS
Chief Draftsman, Board of Engineers, Quebec Bridge

As soon as it was definitely established that the collapse of the first suspended span of the Quebec Bridge was due to the failure of the shoe casting at the south-west corner of the span, which transmitted the dead load of the span to the lower supporting girders attached to the lifting chains, the problem of designing a type of bearing which would present all the advantages of the first design and eliminate the objectionable features was taken up by the engineers of the contractor for the superstructure, and also by the engineers on the staff of the Board of Engineers for the Dominion Government.

The functions which a satisfactory design of shoe would have to perform were several and distinct. During the process of erection of the span on falsework at Sillery provision had to be made in these end bearings, not

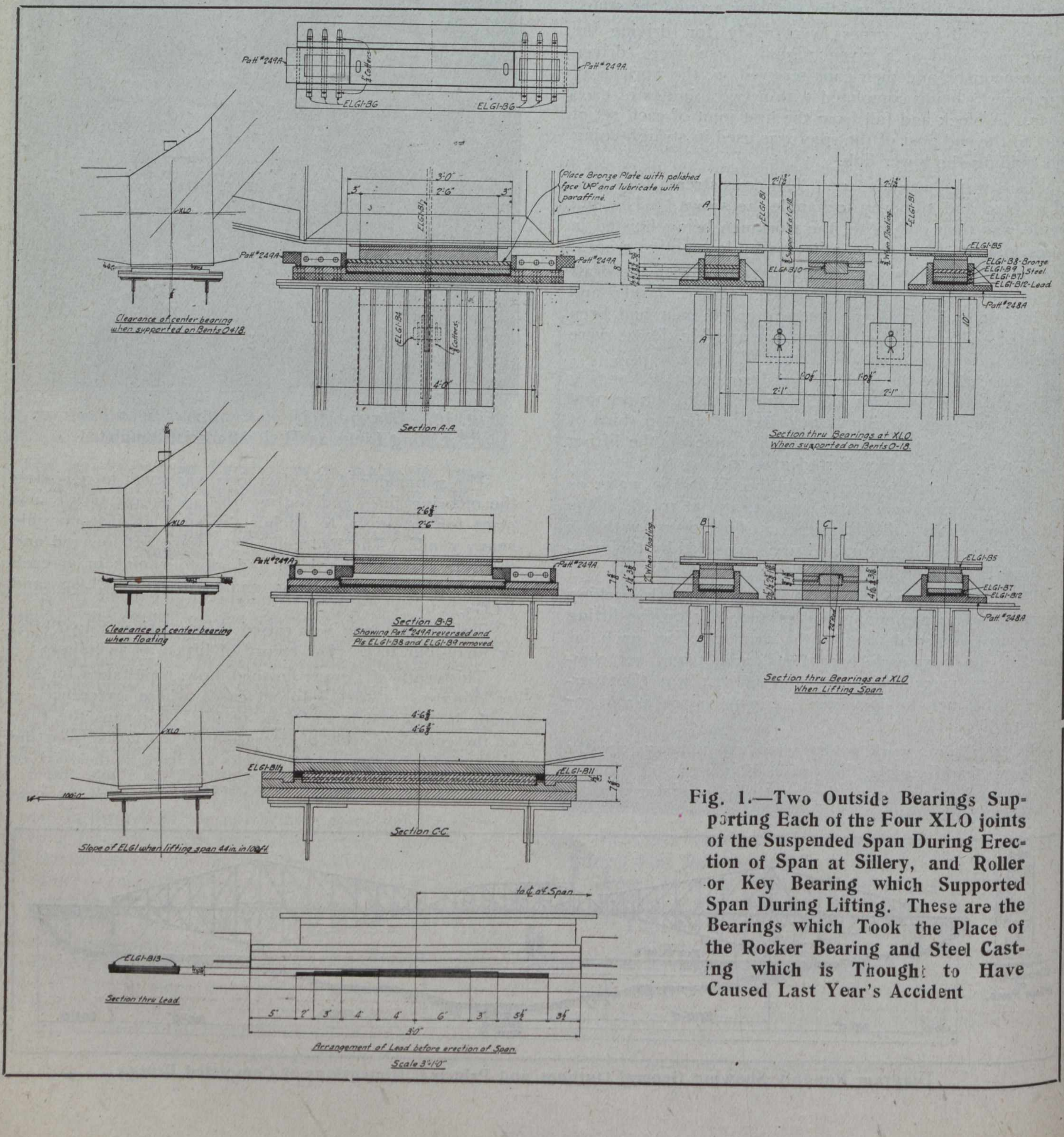
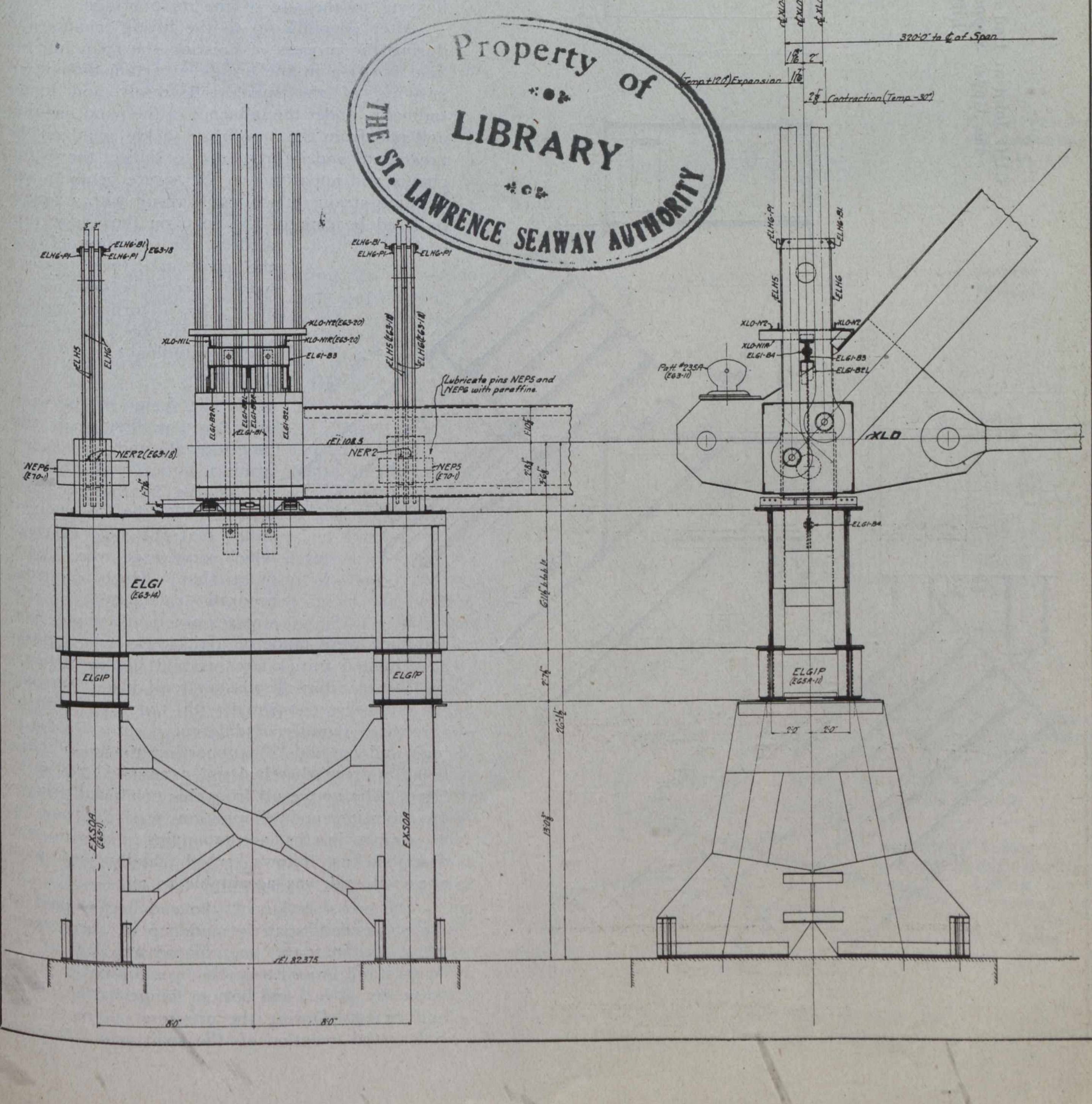


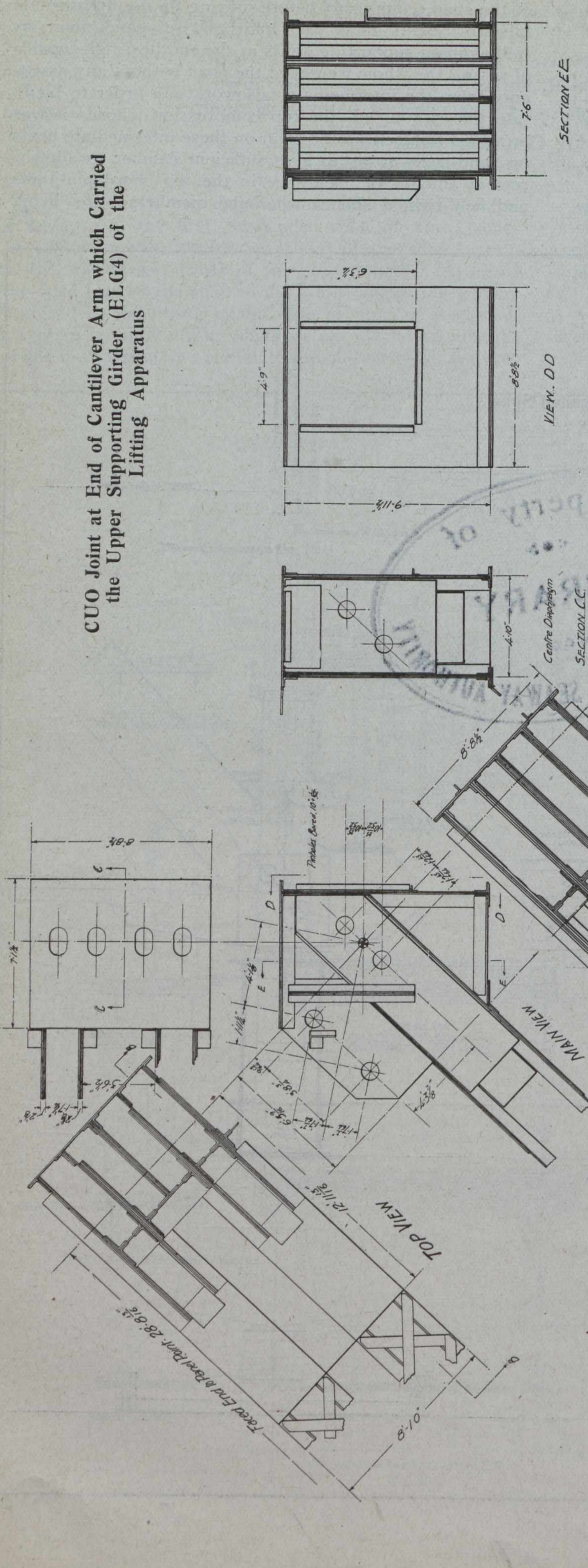
Fig. 1.—Two Outside Bearings Supporting Each of the Four XLO joints of the Suspended Span During Erection of Span at Sillery, and Roller or Key Bearing which Supported Span During Lifting. These are the Bearings which Took the Place of the Rocker Bearing and Steel Casting which is Thought to Have Caused Last Year's Accident

only to take the dead weight of the span, traveller and floor while the span rested on its end bents at Lo and L18, as shown in Fig. 2, but an expansion and contraction had to be provided for, due to a variation of temperature during the summer months in which the span was erected of about 90 deg. Fah.; that is, from a minimum of 30 deg. to a maximum of 120 deg. The amount of this expansion and contraction totalled from 3 to 3½ inches for each bearing.

The programme of erection called for the supporting of the span temporarily before moving up on intermediate falsework bents under each main and sub-panel point, as well as main supporting bents at the end bearings capable of taking the whole weight of the span without any assistance from the intermediate falsework. In order to facilitate the erection of the top compression chords it was proposed to block up the span on these intermediate bents on sand-jacks so as to give sufficient camber to allow a certain amount of clearance in the top chord pin-joints and field-riveted splices when the members were being

Fig. 2.—Support of the XLO Joint of the Suspended Span on Bearings at End Bents L0 and L18





CUO Joint at End of Cantilever Arm which Carried the Upper Supporting Girder (ELG4) of the Lifting Apparatus

placed in position in the span. As soon as the span was completely assembled these splices were to be efficiently bolted together with smaller diameter bolts than the finished holes would demand, and the sand-jacks were to be lowered until the span rested on its main end bents at Lo and L18. The faced splices and pin ends of the top chord would then come to a square bearing and the splices would be fully riveted.

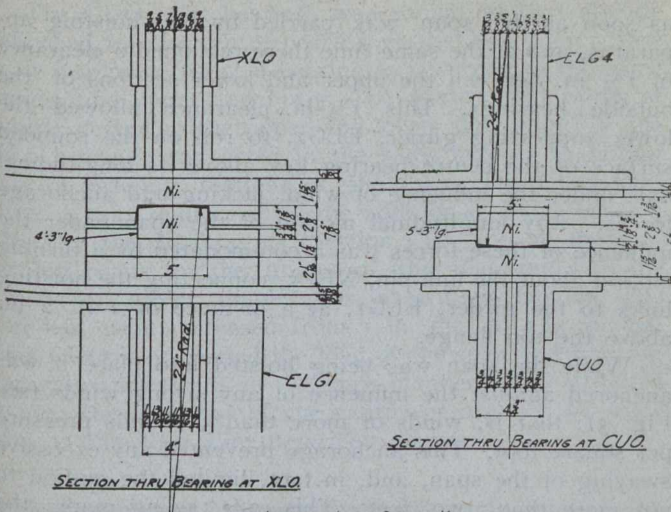
During this operation the members of the span would deform under their dead load stresses. The initial camber would be to a large extent removed and the span would straighten out, involving an adjustment in inclination to the horizontal of the end bearings, for which it would be necessary to make provision. A similar but reversed adjustment of the bearing would also take place when the span was being lifted and the load taken by the barges prior to floating to the site of the main bridge.

After coupling up to the lifting chains and during the process of hoisting the span into its final position in the bridge a certain amount of swaying of the span, transversely and longitudinally, under the influence of the wind, jacking and pull from the anchorage tackle could not be prevented, and in order not to subject the lifting chains and apparatus to the severe bending and twisting strains which would result with a square or fixed bearing of the span on the supporting girders, the bearings between the span and these girders would have to be designed to allow for a turning motion about both the transverse and longitudinal axes of these bearings.

In the design of the bearings for the first span this longitudinal and transverse turning motion was provided for in the bearing pedestal itself, with the consequent result that the steel castings which were used were made of a shape which rendered accurate calculation of the stresses practically impossible, and they were made of a thickness which probably invited initial stresses and hidden defects.

Considerable discussion took place between the engineers connected with the work, and as a result a number of different designs for bearings and methods of supporting the span while hoisting were sketched up, examined and criticized. The net result from this combined process of evolution and elimination was the bearing as shown in the accompanying diagram and described as follows, which finally met with approval and was acceptable to all:—

The cross-section of the end batter post of the suspended span is made up of three main webs, 45 in. x $1\frac{3}{4}$ in., spaced at 2 ft. 1 in. centres and braced together by a top cover plate, 60 in. by $\frac{1}{2}$ in., and bottom flange batten plates and lacing. During the progress of erection at Sillery the reaction of the span was taken on



LOAD PER LINEAR INCH ON KEY

	CUO lbs.	XLO lbs.
D	49300	48300
Tor	7900	9800
I	9900	9700
W	500	600
Σ	67600	68400

Fig. 4.—Section Showing Nickel-Steel Bearing Keys at XLO and CUO Joints

the two outside webs of this post at the XLo joint and transmitted to heavy transverse diaphragms in the lower supporting girders, ELG1, through part bearings specially designed to perform the functions described before necessary prior to the floating of the span.

These sections of the bearing, as shown in Fig. 1, were built up under each outside web of the XLo joint by a series of plates of varying thickness and of different materials, confined in a lower cast-steel guide casting, Pattern No. 248A. The upper portion of the bearing, ELG1-B5, or that part bolted to the bottom of the XLo joint, was 3⅜ in. thick, and was made of three plates of material taken from stock and riveted together with countersunk rivets, the thicker plate on the bottom being planed on the edges and the under surface. This upper part was guided between the planed sides of the lower bed casting, and slid, under the action of temperature and deformations of the span, on the top bronze plate of the lower portion of the bearing. The lower portion of the bearing was 4⅝ in. thick, and was made up of a loose-top bronze plate, ELG1-B8, ¾ in. thick, with the upper surface polished and lubricated with paraffin. Beneath this bronze plate was placed two loose steel plates, ELG1-B9 and B7, ⅞ in. and 1 in. thick, respectively, and underneath the steel plates a laminated section of sheet lead, ELG1-B12, 13/16 in. thick, before taking load. This sheet lead flowed under the weight of the span and took care of the change in inclination of the bearing to the horizontal as the span straightened out and the initial camber disappeared, when the dead load of the span was transferred from the intermediate staging supports to the main end bents at L0 and L18. The loose bronze and steel plates were confined transversely in the bed casting, Pattern No. 248A, and were held against longitudinal motion by cast-steel blocks, Pattern No. 249A, which were bolted into each end of the bed casting by three 1-in. dia. through bolts, ELG1-B16.

Three-inch clearance for longitudinal motion on the bronze plate was allowed between the ends of the upper bearing and the cast-steel blocks. The laminated section of sheet lead filling was prevented from squeezing out around the edges of the loose steel plates in the lower casting by a sheet steel retaining collar, ELG1-B13. This sheet lead, which was 13/16-in. thick before taking load, was expected to reduce to an average thickness of ½ in. under the reaction of the span and fill the space between the loose steel plates and the casting. Measurements taken at the four corners of the span after the full reaction had come on the bearings showed that the sheet lead had squeezed down to an average thickness of from ½ in. to ⅝ in.

The span, when it was floated by the barges, carried the lower supporting girder, ELG1 with it, suspended from the XLo joint by the small plate hangers, ELG1-B1. The length, back to back, of pin-holes on these hangers was made such that when the span floated there was a clearance of ⅜ of an inch between the upper and lower

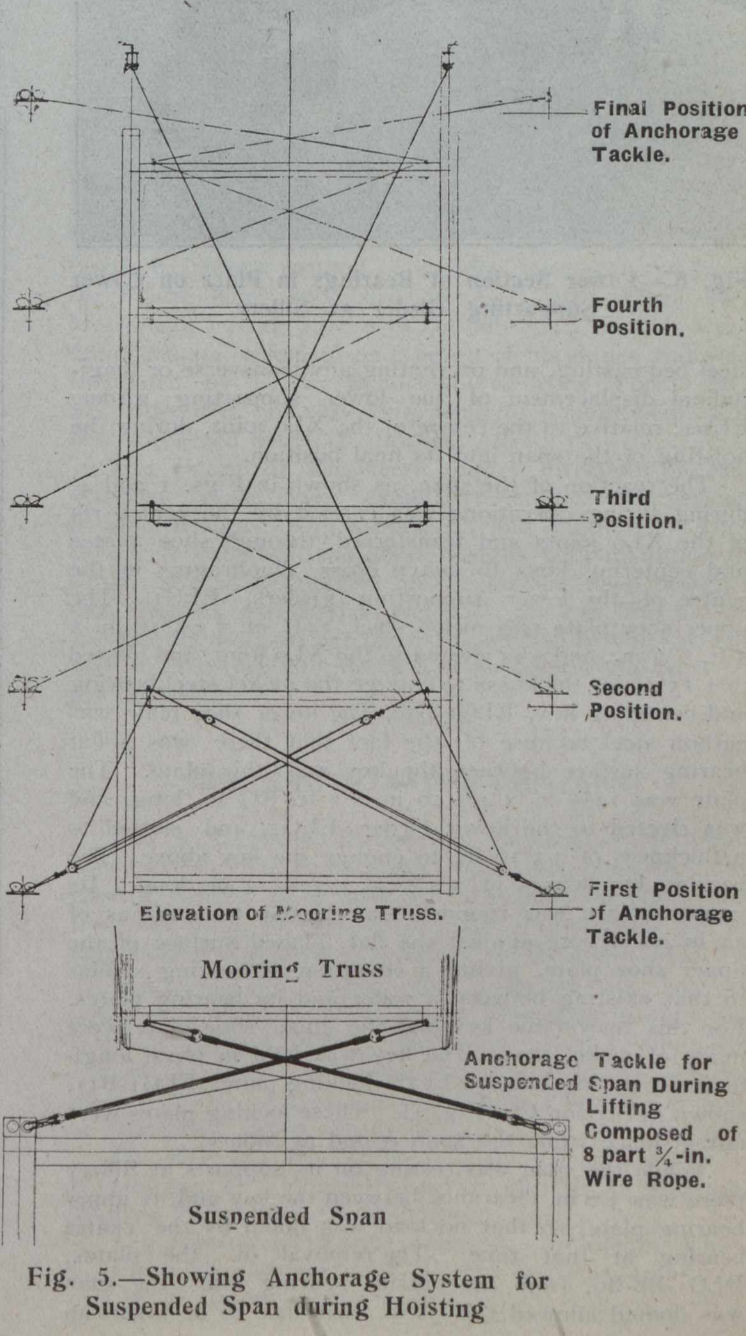


Fig. 5.—Showing Anchorage System for Suspended Span during Hoisting

portions of the outside bearings. In this condition the ¾-in. bronze plates, ELG1-B8 and the ⅞-in. steel plates, ELG1-B9, were removed, and the cast-steel blocks, Pattern No. 249A, were reversed in position, centering the girders, as shown in Section BB (Fig. 1), and effectually blocking the upper portion of the bearing into the lower

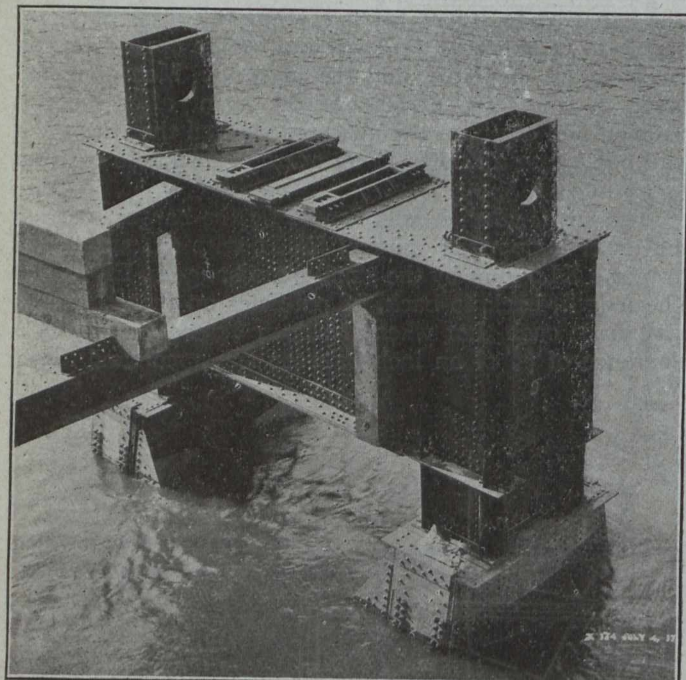


Fig. 6.—Lower Section of Bearings in Place on Lower Supporting Girder at Sillery

steel bed casting, and preventing any transverse or longitudinal displacement of the lower supporting girder, ELG1, relative to the centre of the XLo joint, during the hoisting of the span into its final position.

The reaction of the span, as shown in Figs. 1 and 4, during lifting operations was carried by the centre rib of the XLo joints and transferred through shoe plates and centering keys to heavy cross diaphragms in the centre of the lower supporting girders, ELG1. The upper shoe plate was nickel steel, 12¼ in. x 2 5/16 in. x 4 ft. 6⅜ in., and was riveted to the XLo joint, and slotted to 1 15/16-in. thickness to engage the nickel steel bearing and centering key, ELG1-B10. The lower shoe plate was carbon steel because of the fact that there was a flat bearing surface between the key and this plate. The plate was 12½ in. x 4 1/16 in. x 5 ft. 8½ in. long, and was riveted to the lower girder, ELG1, and slotted to a thickness of 3 5/16 in. to engage the key above. The bearing key was 5 in. x 2⅜ in. x 4 ft. 3 in. long. Its upper surface was rounded transversely to a radius of 24 in., and bore against the flat, planed surface of the upper shoe plate, giving a condition of bearing similar to that existing between a roller and its bearing plates. For this reason the key and the upper shoe plate were made of nickel steel. The key was held in place longitudinally during hoisting by the locking plates, ELG1-B11, shown in Section CC (Fig. 1). These locking plates were not in place while the span rested at Sillery.

While the span was resting on its supports at Sillery there was 3/8 in. clearance between the key and its upper bearing plate, so that no load was taken by the centre bearing at that time. The removal of the plates, ELG1-B8-B9, from the outside bearing after the span was floated allowed this ⅜-in. clearance to be taken up as soon as the span was carried by the hoisting apparatus, and at the same time there remained a clearance of 1¼ in. between the upper and lower sections of the outside bearings. This 1¼-in. clearance allowed the lower supporting girder, ELG1, to roll on the rounded surface of the centre bearing key above its longitudinal axis under the influence of wind, jacking and anchorage forces. Any longitudinal motion of the span under the influence of these forces was accommodated by a turning motion about the link-pin, NEP5, connecting the hoisting links to the girder, ELG1, at a distance of 2 ft. 2 in. above the top flange.

While the span was being hoisted into place it was anchored against the influence of any strong winds (see Fig. 5): that is, winds of more than 5 pounds pressure per square foot. This anchorage prevented any excessive swaying of the span, and, in fact, limited the motion to not more than two feet. This 2-ft. swing made the bearing of the XLo joint on the key eccentric about ¼ in.

The bearing castings of last year transferred a portion of the load to all three ribs of the XLo joint. The XLo joint in the new span was heavily reinforced throughout in order to make it capable of taking the whole reaction on the centre rib and distribute it properly throughout the joint. The bearing thickness of the centre rib was

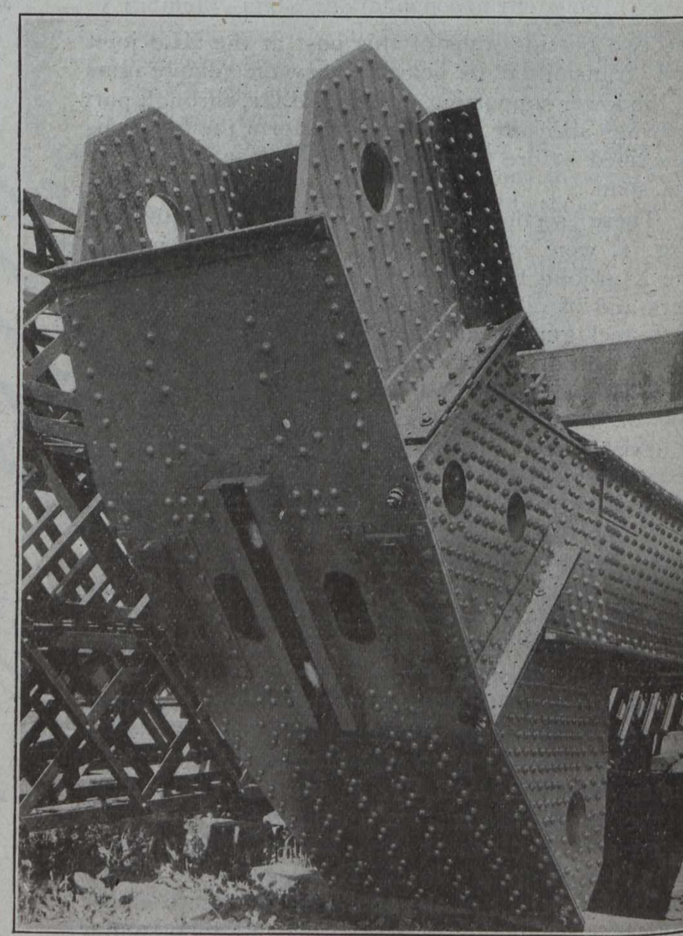


Fig. 7.—Upper Section of Bearings Riveted to Bottom of XL0 Joint

increased to 6 in., and the cross girder diaphragms were made very much stronger and deeper, and were capable of transferring that portion of the reaction which should go to the outside ribs without counting on any assistance from the exceptionally large tie-plates in the plane of the bottom chord and in the plane of the end batter post or the 60-in. x ½-in. cover plates of this end post.

The CUO joint bearing was similar to that already described for the centre bearing of the XLo joint; that is, it was made up of a bearing key, turned to the same radius, and upper and lower shoe plates, with slots of similar design to the upper and lower shoe plates of the XLo bearing, and acted in the same manner in taking care of swaying motions of the span. The CUO joint (Fig. 3) was reinforced by the addition of extra bearing plates to the diaphragms (Section CC), and all the connecting rivets for the angles of this diaphragm to the transverse girder diaphragms, shown in Section DD and EE, were increased from 1 in. dia. to 1⅛ in. dia. This CUO joint carried the load of the span and hoisting apparatus practically in the same manner and distributed it over the end post of the cantilever arm in the same way as the XLo joint described above.

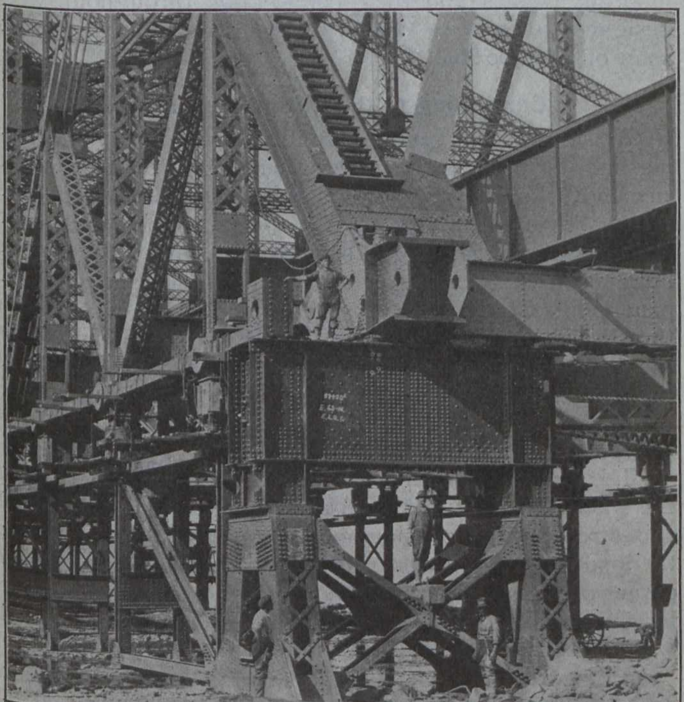


Fig. 8.—Lower Supporting Girder at XLO Joint Resting on End Bent

The nickel steel used for the bearing keys and shoe plates had to show an ultimate strength of 90,000 to 105,000 lbs. per sq. in. and a minimum yield point of 55,000 lbs. per sq. in. in the specimen test. To determine the bearing per lin. inch of key that would be allowed, a full-size test was made on a short section of a key made of carbon steel. The test load was caried up to 80,000 lbs. per lin. inch, and showed a small, permanent indentation in the upper shoe plate and a small flattening of the rounded surface of the key. No permanent distortion took place until the load had reached about 50,000 lbs. per lin. in. This test was made with carbon steel material. The keys and upper shoe plates were made of nickel steel, which is at least 40 per cent. stronger than carbon steel. The load per lineal inch on the lower keys from the dead weight of the span was about 50,000 lbs., with 20 per cent. impact and 5 lbs. wind. The load per lineal inch was about 60,000 lbs. A 5-lbs. wind was all that was considered with impact, because, if a stronger wind came up, hoisting would not be continued, and, therefore, there would be no chance of the impact occurring with the larger wind pressure.

THE HISTORY OF THE BRIDGE

Enterprising citizens of the city of Quebec advocated a bridge across the St. Lawrence River near Quebec as early as 1852, the cost then being estimated at $3,000,000. In 1882 a charter to construct the bridge was obtained by M. W. Baly, of Quebec, and two years later a design was submitted to the Quebec Board of Trade for a bridge at about the present site, but nothing was ever done until 1900, when the company was reorganized as the Quebec Bridge & Railway Co. and undertook a Dominion government contract to build the structure. The south cantilever arm of the first bridge collapsed on August 29th, 1907, with a loss of 70 men and about $8,000,000.

After the accident a commission was appointed to report on its causes. The members of that commission were J. G. G. Kerry, Henry Holgate and the late Dean Galbraith, of Toronto University. After their report the government decided to reconstruct the bridge, and in 1908 appointed a board of three engineers to prepare plans. The board consisted of H. E. Vautelet, chairman; Ralph Modjeski; and Maurice Fitzmaurice, chief engineer of the London (Eng.) county council.

After studies lasting about a year, the idea of inclined planes for the trusses was abandoned, and Mr. Fitzmaurice, who was a staunch advocate of the Forth type of bridge, resigned. He was temporarily succeeded by Mr. McDonald, and when bids were called, Messrs. Hodge and M. J. Butler were called in as consultants. Four out of the five recommended that the St. Lawrence Bridge Co.'s tender be accepted. Mr. Vautelet did not favor that firm's method of erection, and resigned from the Board, being succeeded by Col. Monsarrat. Mr. McDonald, having agreed to act only until the bids were decided upon, asked to be relieved of his duties and was succeeded by C. C. Schneider, of New York. Mr. Schneider died not very long after his appointment, and was succeeded by H. P. Borden.

The Board of Engineers, under direction of Mr. Vautelet, made very exhaustive studies of various possible designs, both suspension and cantilever. Tenders were called on cantilever designs with invitation to submit alternative tenders on the bidders' own designs. One German, one English and one American firm bid on the board's designs, but the St. Lawrence Bridge Co. bid only on their own alternative K-truss designs and received the contract.

THE CONTRACTING COMPANY

The St. Lawrence Bridge Co., Limited, of Montreal, was incorporated August 5th, 1910, to construct the Quebec Bridge. The company undertook no other contract excepting for the manufacture of some 9.2 and 6-inch shells. Its shops are located at Rockfield, P.Q.

The authorized capital stock is $3,000,000, half of which was subscribed for by the Dominion Bridge Co. and half by the Canadian Bridge Co. Poor's Manual of Industrials says: "Paid in to May 31st, 1917, $2,750,000. Shares, $100. Annual meeting, third Thursday in May at Montreal. Stock closely held."

The directors are Phelps Johnson, Chas. Cassils, G. H. Duggan, F. L. Wanklyn, J. F. Weber, F. C. McMath, Willard Pope and B. S. Colburne.

The officers are Phelps Johnson, president; Chas. Cassils, vice-president; J. F. Weber, secretary-treasurer; and W. P. Ladd, superintendent.

Mooring and Hoisting The Suspended Span

General Principles of Last Year's Operations Were Again Followed bu: the Design of the Apparatus and the Program for Its Use Differed in Many Impor:ant Details

By A. J. MEYERS and M. B. ATKINSON

Chief Draftsman and Asst. Chief Draftsman, Board of Engineers, Quebec Bridge

THE suspended span was floated on six barges from Sillery to a position between and approximately in line with the cantilever arms. The time of arrival was controlled by tugs so that the span was in position about one-half an hour after high tide when for a period of one hour the current did not exceed three miles an hour and during which it changed direction. It was necessary for the tugs to hold the span against the wind and current while the 1¼-inch mooring lines were being connected. The span was then pulled directly under its final position in the bridge by means of these 1¼-inch mooring lines.

The mooring lines were eight in number, two at each corner of the span. At the end of each rope was a loop which, as soon as the span had come within reach and the speed controlled, was thrown over a double-headed cast-

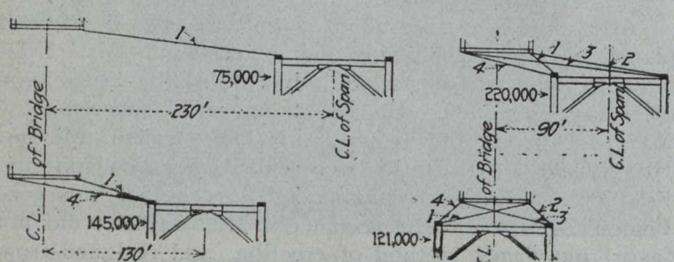


Fig. 1.—Showing Arrangement of Mooring Lines from Each End of Suspended Span to Adjacent Mooring Truss

steel snubbing block or towing bit (see Figs. 1 and 2) bolted to a seat provided at the XLo joint of the suspended span. Each 1¼-inch rope was calculated to take a pull of 75,000 lbs. The ropes passed through sheaves at the lower corner of the mooring trusses and from there ran vertically to the trusses of the cantilever arm, where they connected to a 9-part ¾-inch wire rope tackle, which led back to the drums of the derrick hoists, situated on the floor at the end of each cantilever arm.

It had been calculated that these mooring lines might have had to act separately, in pairs, or all four together at each end of the span. The diagrams (Fig. 1) show that the mooring lines might have been connected to the span in several positions. In each assumed case shown, the sum of the components of the 75,000-lb. line pull, at right angles to the axis of the bridge, is given, also the current and wind forces for which such a pull would be able to provide.

Mooring Truss

The mooring frames, as shown in Fig. 2, were made up of two steel trusses about 125 ft. long and 17 ft. deep over all, and spaced 54 ft. 0 in. centre to centre of trusses. These mooring trusses were braced together by one vertical plane of lateral bracing and three horizontal cross-sway frames at the intermediate panel points of the trusses. They were suspended from the cantilever arm floor beams at panel points CF1 and were hinged at the upper end as shown in detail A, so that they could be swung back practically up against the plane of the bottom chords of the cantilever arm in order not to obstruct the channel unnecessarily in case it would have been necessary to leave them in position for any considerable length of time, and also so that they could be drawn back sufficiently in order to allow plenty of clearance for the suspended span while it was being drawn into position by the mooring lines. A 9-part ⅞-inch wire rope tackle was used to handle the frames. This tackle led from the lower corners of the trusses to the connections to the floor between panel points CF5 and CF6 of the cantilever arms and from there to the main hoists situated at the floor level of the cantilever arms and on the centre line of the bridge. These mooring frames and their connections throughout were designed to take a transverse pull from the suspended span of 300,000 lbs.

Connecting to the Lifting Apparatus

As soon as the span was pulled into position the lifting links, which were made long enough to connect to the suspended span with water at elevation 82.48 ft., were swung down by means of the 6-part ¾-inch wire rope tackle and sling attached through hole No. 26 of the links and to the cantilever arm bottom chord adjacent to panel point CL4. The links were then connected through the slotted holes in the lower links ELH4 (see Fig. 3) by the pins at the top of the short hanger link, connecting to the supporting girders ELG1 under the XLo joint.

Lower Lifting Girders

The lower supporting girders ELG1 (see Fig. 4) were the same ones that had been used last year but reinforced and strengthened considerably throughout. They took the load from the key bearing under the XLo joint at each corner of the span during hoisting and on the two outside bearings while the span rested on the end bents Lo and L18 at Sillery.

Each girder was made up of two ribs, composed of one 83½-inch by 11/16-inch web plate and four 8 x 8 x 11/16-inch angles 20 ft. 0 in. long, the web plate was reinforced on the outside by one 66½ x ⅜-in. plate 18 ft. 4½ ins. long, the distance back to back of angles being 6 ft. 11½ ins. The two ribs were spaced 4 ft. 0 in. centre to centre of webs, the top flanges were connected by one 66 x ⅝-in. plate 20 ft. long and one 66 x ⅝-in. plate 6 ft. 8¾ ins. long. The bottom flanges were not connected except by the cross-diagrams, but each rib had two cover plates 18 x ⅝-in. and 12-ft. 1 13/16 ins. long and 6 ft. 8¾ ins. long respectively.

At the centre there was a cross-diagram composed of two 43 x 11/16-in. plates and four 8 x 8 x 13/16-in. angles running the depth of the girder. The diaphragms had reinforcing bearing plates of varying lengths which with the web made up a bearing thickness of 3¾ ins. at the top. Two 4 x 4 x ½-in. angles 2 ft. 5½ ins. long, riveted to the top flange plates and diaphragm web, engaged the rivets which connected the lower key bearing plates to the girder. Vertically on the centre line of the diaphragm 4 x 4 x ½-in. angles 5 ft. 4 ins. long connected the longitudinal diaphragm webs connecting the two cross-diaphragms, which took the load while the span was resting at Sillery, to the centre diaphragm. Between these centre angles and the end connection angles of the diaphragm were six sets of short vertical stiffener angles bearing against the 4 x 4 x ½-in. horizontal angles at the

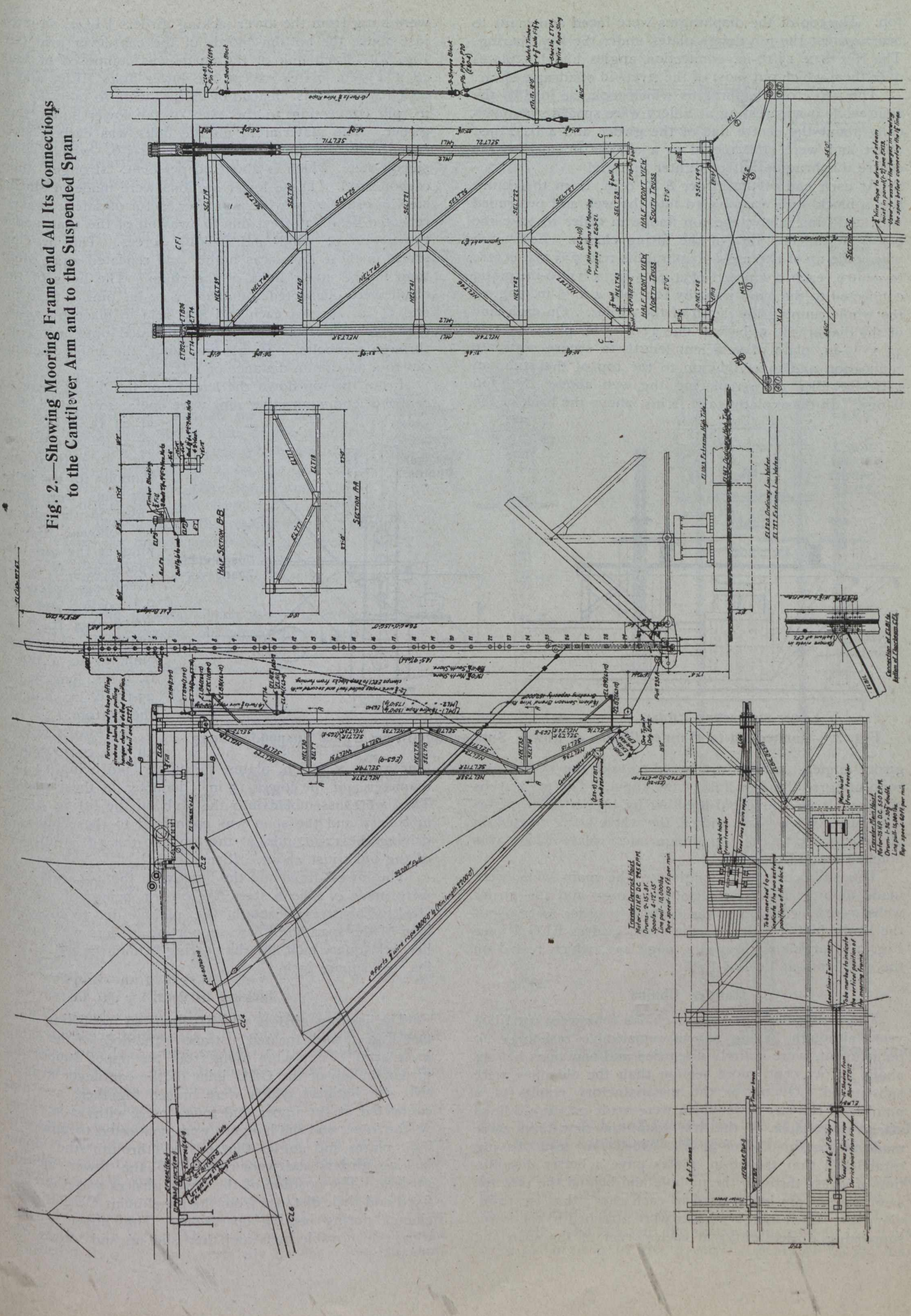


Fig. 2.—Showing Mooring Frame and All Its Connections to the Cantilever Arm and to the Suspended Span

top. The top of the diaphragms were faced and made to bear against the top flange plates under the key bearing. The 8 x 8 x 13/16-in. connection angles had lap plates over them which put most of the rivets in quadruple shear.

The two cross-diaphragms which took the load of the span while it was resting at Sillery were spaced symmetrically about the centre line of the girder and 4 ft. 2 ins. apart and were constructed in a similar manner to the centre diaphragms but of lighter material

At each end of the girder 8 ft. 0 in. from the centre line a block stub was riveted into the web and protruded 3 ft. 10½ ins. above the top flange in order to take the pin from the first link of the lifting chains. Each stub was made up of two cross-diaphragms 1 ft. 7⅝ ins. centre to centre, each composed of a 44½-in. x 11/16-in. plate, connected to the main webs by four 6 x 4 x ½-in. angles, the whole running the depth of the girder. On each side of the diaphragm webs was one 33 x ¾-in. plate and one 33 x ½-in. plate of sufficient length to secure adequate connection and extending up to the top of the stub, an 11/16-in. filler plate replacing the web above the top flange. In these plates, 2 ft. 2 ins. above the back of the girder angles, pinholes are bored to take the 15-inch pin connecting to the first link of the chain. The main webs of the stub are connected together by 14½ x ⅜-in. plates with two 4 x 4 x ½-in. angles, the whole being riveted on all four sides to the top flange cover plate which was notched around the stub.

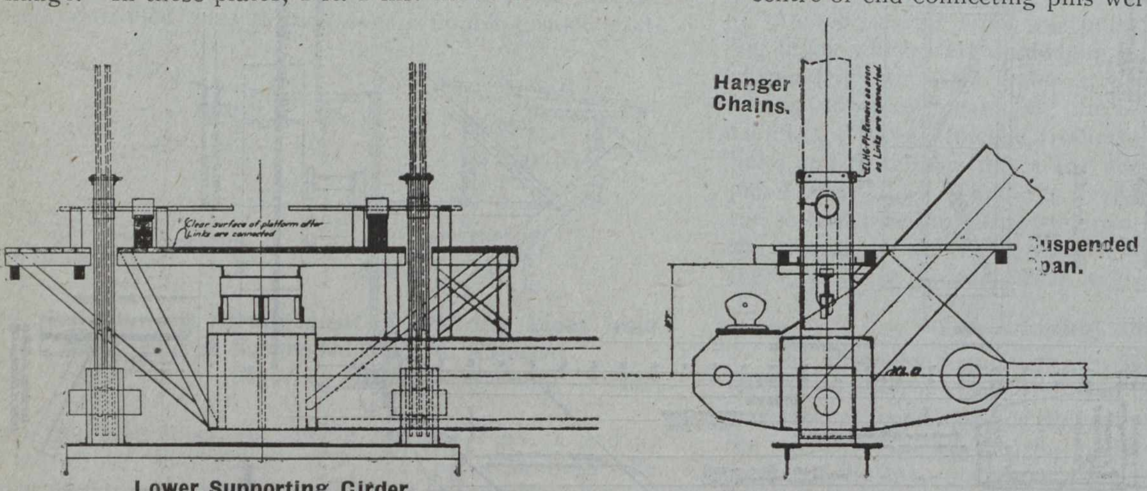


Fig. 3.—Connection to Hanger Chains of Lower Supporting Girder Under Suspended Span

The stub connecting angles to the main webs were made to bear against the bottom flange and the girder webs were faced flush at the bottom in order to transmit the load while at Sillery to the small girders ELG1P (see Fig. 2 of article on bearings, page 241, which rested on the end bents at L0 and L18.

Hanger Chains

The lifting links or hanger chains that were used last year, although having shown remarkable resistance to punishment, were entirely discarded and new ones having about 25 per cent. more section than the old ones were substituted. Owing to the unsatisfactory results of a number of full-size tests which were made on the old links after the collapse of the first span, the new links were made 28 ins. instead of 30 ins. wide, the tests having established that the 28-in. plates gave a better distribution of stress through the pinhole and behind the pin, the same size of pin being used in each case; that is, 12-in. diameter. Two hanger chains were attached to the lower supporting girder ELG1 at each corner of the span and were hung from the lower jacking girders ELG2, situated just above the bottom chords of the cantilever arm (see Fig. 4). Each link of the chain was composed of four 28 x 1½-in. plates except the lower links ELh5 and 6 which were 31 ins. instead of 28 ins. wide to take the 15-in. pin connecting to the stub of the lower supporting girder. The material in all the links was carbon steel except the two lower sections ELH4-5-6, which. were silicon steel, which is about 20 per cent. stronger than carbon steel. These lower sections were made of silicon steel in order to provide a greater factor of safety against repeated bending stress due to swaying of the span under the action of wind and jacking forces. The bending stress in these links was higher and repeated more often than in the upper links of the chain. The links of the chain were connected together by 12-in. pins 1 ft. 0 in. and ¾ in. long at each end and held in place by 14-in. diameter cast-steel pin caps, threaded onto a 1¾-in. through pin bolts, 1 ft. 2½ ins. long, the whole packing out to a maximum distance of 1 ft. 2¾ ins.

From the top down the lengths of the links centre to centre of end connecting pins were one at 29 ft. 0 in., five at 24 ft. 0 in., one at 23 ft. 9 ins. and one at 9 ft. 9 ins. Pinholes for the jacking pins were provided in the chains as follows: Beginning at the top two spaces at 4 ft. 0 in., twenty-six spaces at 6 ft. 0 in., and one space at 4 ft. 0 in. centre to centre of pinholes to a slot 4 ft. 9 ins. long in the 23-ft. 9 in. link at the bottom.

The lowest link of all was composed of four 31-in. x 1½-in. plates, the upper pin being 12 ins. diameter and the lower 15 ins. diameter in these plates. They were assembled into the stub on the lifting girder at Sillery and the span was connected to these links by driving the 12-in. pins at the top into the 4-ft. 9-in. slot of the links just above. (See Fig. 3.)

The lowest links and the links with the 4-ft. 9-in. slot were made of silicon steel, the others being of carbon steel. The stress through the pinhole due to the dead load of the span and lower supporting girders was 14,800 lbs. per square inch and the bearing behind the pin 21,000 lbs. per square inch.

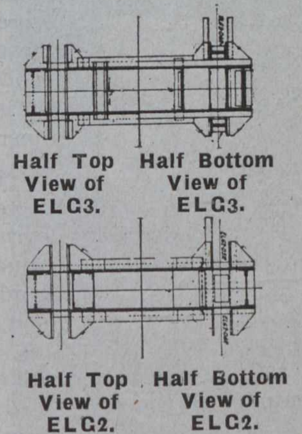
Jacking Girders

The jacking girders which supported the lifting links (see Fig. 4) were located just above the floor of the cantilever arm. They were hung from the upper supporting girder on top of the CUO joint of the cantilever arm by two stiff hangers which were braced together, and pin-connected at the upper and lower ends with 12-in. pins. At the lower end the hangers were connected to guides of built plates and angles that passed through the upper jacking girders and were riveted to the lower jacking girders. The position of the lower girders was therefore fixed and their distance from the panel point CUO did not change during jacking operations. The upper girders were the movable ones and they slid up and down the guides as the 1,000-ton hydraulic jacks were operated. These jacks, two between each pair of jacking girders, spaced 21 ft. 0 in. apart, eight in all, did the work of lifting the span.

In order to avoid binding of the jacks due to the deflection of the jacking girders under load, the jacks were provided with rocker seats at their upper and lower bearings. The ram of the jack was 22 inches in diameter and it had a stroke of 2 ft. 3 ins. The jacks were made by the Watson-Stilman Co. and were tested at their plant to a fluid pressure of 6,000 lbs. per square inch and at the bridge site in place to 5,200 lbs. per square inch. The dead load pressure during lifting was only 3,800 lbs. per square inch and the maximum calculated load during lifting, 5,250 lbs. per square inch, which included the effect of impact and wind. The jacks were supplied with water at the necessary pressure by a pair of direct acting double-plunger pumps operated by compressed air supplied at a pressure of 110 lbs. per square inch.

The pumps were located on the centre line of the bridge floor at the ends of the cantilever arms. They had safety valves which would have released at a pressure of 5,000 lbs. per square inch in case one set of jacks bound and as a result the other set might have taken considerable overload. The speed of the pumps was 60 strokes or 30 cycles per minute and it took 420 cycles, that is, 210 for each pump, to lift the span 2 ft. The time consumed in one lift at two feet was from 7 to 8 minutes.

The jacking girders, which were the same ones as were used last year, but strengthened about 25 per cent. throughout, were each made up of two ribs, channel section, flanges outstanding, 22 ft. 6 ins. long and 9 ft. 1½ ins. back to back of flange angles. The webs were 108 x ⅝-in. plates, reinforced at the ends in both girders. The upper girder flanges were made up of one 8 x 8 11/16-in. angle reinforced in the centre portion by a horizontal cover plate 12¼-in. x ½-in. and for the lower girder, the flanges were made up of one 8 x 8 x ⅞-in. angle with side plates 12¼ x 11/16-in. and one horizontal flange plate 13¼ ins. x 11/16 in. in the centre portion of the girder.



The upper girder ribs were spaced 4 ft. 4 ins. apart and the lower girder ribs 3 ft. 9 ins. At each end of the girders were cross-diaphragms taking the load from the hydraulic jacks. These diaphragms were made up of two ⅝-in. web plates reinforced at the top to a bearing thickness of 5⅛ ins. and connected to the ribs by 8 x 8 x 13/16-in. angles with ⅞-in. rivets. On each side of the centre line of the lifting links at each end of the girders were double cross-diaphragms to take the jacking pins. Each diaphragm rib was made up of silicon steel plates to a thickness of 2⅝ ins., the distance between ribs being 1 ft. 4⅜ ins., giving a clearance to the link cap on each side of 13/16 of an inch. The diaphragms has three pinholes spaced 2 ft. 0 in. apart vertically for the 12-in. diameter

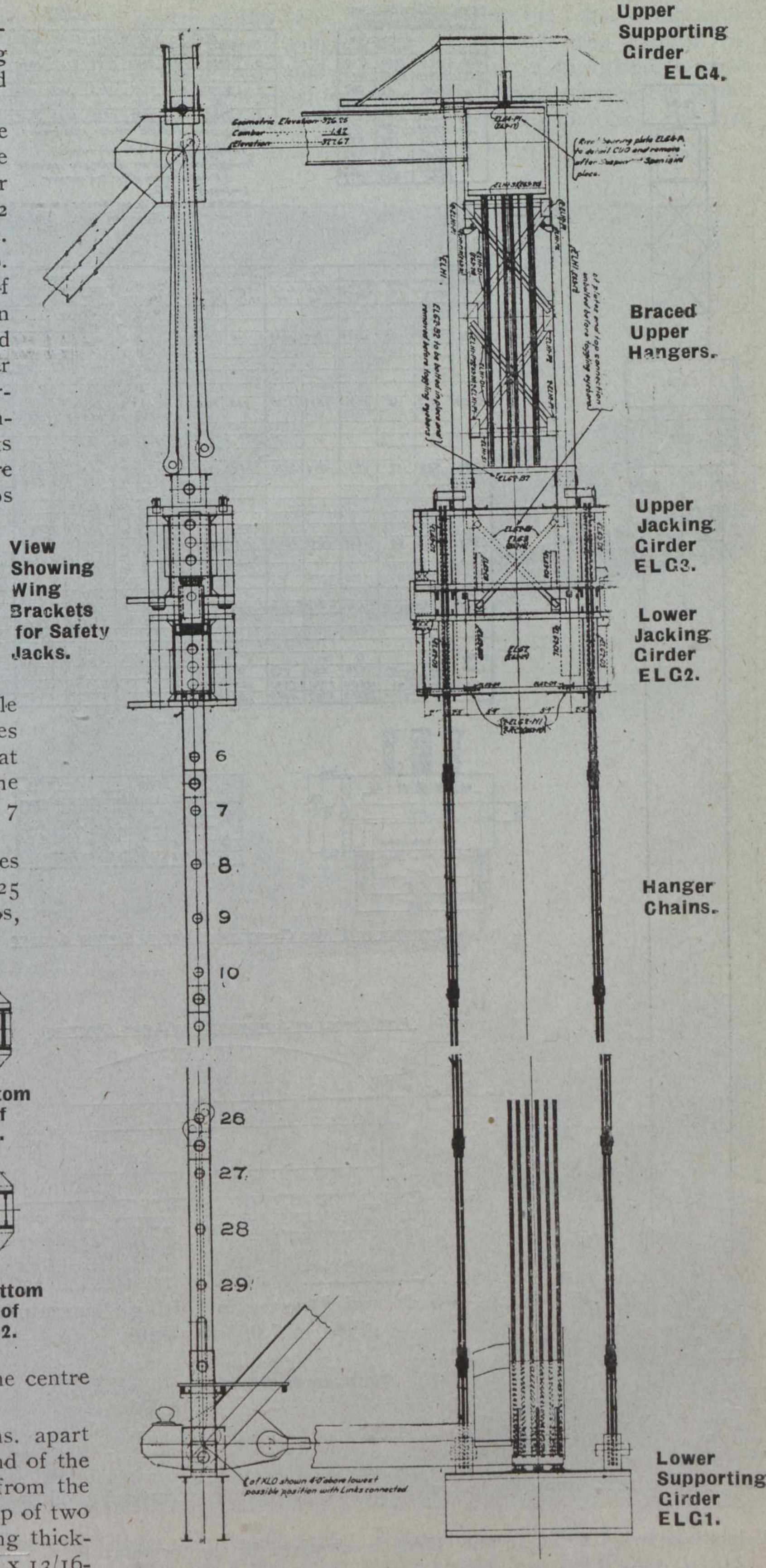
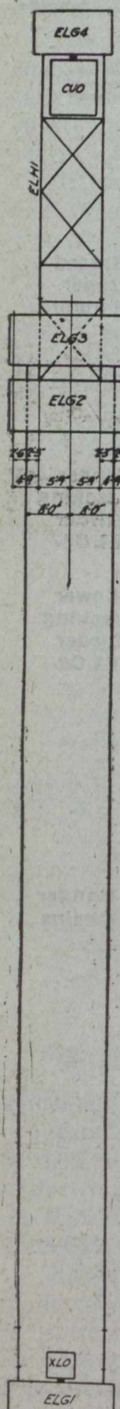


Fig. No. 4.—Side and Front Elevation of Supporting and Jacking Girders with Hanger Chains in Position

nickel steel jacking pins. The holes were so arranged in the upper and lower girders that the pinholes in the links could always engage pins in both girders for a 2-ft. lift. The holes in the diaphragms and in the links were so bored as to give a vertical clearance of 1¾ ins.



WEIGHT OF SUSP. SPAN AND LIFTING APPARATUS

		Per Corner lbs x 10³	Total lbs x 10³
Susp. Span	Erf'l incl.	2820	11280
	as floated	2460	9840
Lifting Apparatus	ELG1	65	260
	Links	227	908
	ELG3	88	352
	Jacks	31	124
	ELG2	122	488
	ELHI	58	232
	ELG4	52	208
	Total	643	2572

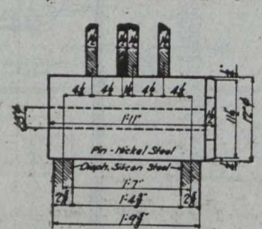
LOADS

	ELG1 Sillery lbs x 10³	ELG1 Lifting lbs x 10³	Lifting Links Bottom lbs x 10³	Lifting Links Top lbs x 10³	ELG3 lbs x 10³	Jacks lbs x 10³	ELG2 lbs x 10³	ELHI lbs x 10³	ELG4 lbs x 10³	CUO lbs x 10³
D	2820	2460	2525	2752	2752	2840	2871	2993	3051	3103
Tor		500	500	500	500	500	500	500	500	500
I	* 75	492	505	550	550	568	574	599	610	621
W	65	31	31	31	31	31	31	31	31	31
Σ	2960	3483	3561	3833	3833	3939	3976	4123	4192	4255

* Locomotive crane and car +20% impact.

D—Weight of material on member
Tor—Additional load on diagonally opposite corners when 3.3" higher than other two corners.
I—Impact ·020
W—Direct load due to overturning effect of 5lb. transverse wind on the Suspended Span. (10lb. of Sillery)

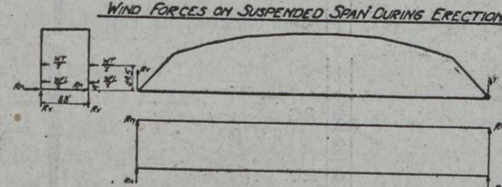
Girder	Reaction lbs x 10³	Lever ins.	Moment in. lbs x 10⁶	Sec. Mod. Req'd ins³	Sec. Mod. Fur'd ins³	Shear Area Req'd sq ins	Shear Area Fur'd sq ins	Material	Area Gross	Rivet Holes No.	Rivet Holes Dia.	Area Net	Moment of Inertia ins⁴
ELG1								2 83½ × 11/16 Pls.	114.8				66700
								2 66½ × ⅜ Pls.	49.9				18400
	1742	96	16723	7742	8030	145	165.0	2 66½ × 5/16 Pls.	41.6				15300
								8 8 × 8 × 11/16 Ls	84.4	3	1⅛	65.8	103500
								8 18 × ⅝ Pls.	90.0	2	1⅛	78.8	141500
													345400
ELG3								2 108 × ⅝ Pls.	135.0				131500
	1917	30	57.51	2662	4120	160	204.0	2 92 × ⅜ Pls.	69.0				
								4 8 × 8 × 11/16 Ls	42.1	3	1	33.9	92000
													223500
ELG2								2 108 × ⅝ Pls.	135.0				131500
								2 69 × ⅜ Pls.	51.8				
	1988	57	113.31	5246	5320	168	186.8	4 8 × 8 × ⅞ Ls	52.9	2	1⅛	44.15	119300
								4 12¼ × 11/16 Pls.	33.7	3	1⅛	23.40	37600
													288400
ELG4								2 89¼ × ¾ Pls.	133.9				88800
								2 73 × ⅜ Pls.	54.7				24300
								4 8 × 6 × ⅝ Ls	33.4	3	1⅛	24.0	43400
	2096	69	144.62	6696	6800	175	188.6	4 8 × 6 × ½ Ls	27.0	3	1⅛	19.5	35500
								2 57 × ½ Pls.	57.0	4	1⅛	37.0	75700
								4 13 × ½ Pls.	26.0	2	1⅛	21.0	44000
													311700
ELHI								4 4 × 4 × 9/16 Ls	16.72	2	1	12.22	
	2096000 @ 21600 = 97.0" net area req'd.							4 27 × 11/16 Pls.	74.25	3	1	66.00	
								2 27 × ½ Pls.	27.00	3	1	24.00	
									117.97			102.22	
Lifting Links			D	Tor.	I	W	Σ	4 28 × 1½ Pls.	168.0	1	12¼	93.0	
	Load lbs x 10³		1376	250	275	15	1916						
	Unit Stress across Pin Hole		14800	2700	2950	150	20600						
	Bearing on Pins		20300	3700	4060	200	28260						



	Load lbs x 10³	Mom. in lbs x 10⁶	Unit Stress lbs per sq in	Bearing lbs/□" On Links	Bearing lbs/□" on Diaphragms
D	1376	4.47	26800	20300	22800
Tor	250	.81	4850	3700	4200
I	275	.89	5350	4060	4560
W	15	.05	300	200	200
Σ	1916	6.22	37300	28260	31760

STRESSES IN 12" PINS CONNECTING LINKS TO LIFTING GIRDERS

WIND FORCES ON SUSPENDED SPAN DURING ERECTION



Wind Force #/□'	Wind Velocity miles hour	Loads WT	Loads WFL	Truss Rv	Truss Rn	Floor Rv	Floor Rn	Σ Rv	Σ Rn	H77	H235	Displacement of Key Bearing
1	19	27000	9600	6200	5500	490	2400	6690	7900	2"	6⅜	1/16"
5	43	110000	48000	31000	27500	2450	12000	33450	39500	10⅝"	2'-7"	5/32"
10	61	220000	96000	62000	55000	4900	24000	66900	79000	1'-8¾"	5'-2"	11/32"
15	75	330000	144000	93000	82500	7350	36000	100350	118500	2'-6¾"	7'-9"	15/16"

Fig. No. 5.—Loads and Stresses in Lifting Apparatus. All forces given in 1,000-lb. Units.

Transverse Wind.

WT—A normal wind load on the exposed surface of two trusses.
WFL—A " " " " " " " " " floors (supporting traveler) as used during erection.
Rn—Horizontal reaction at each corner of the span due to wind forces.
Rv—Vertical reaction at each corner of the span due to overturning effect of the wind forces.

Velocity of wind noted in the tables is as given by the formula—$V^2 = \dfrac{W}{.00271}$.

W=pressure in lbs. per sq. ft.
V=velocity in miles per hour.

The horizontal displacement of the span when supported on lifting apparatus and acted upon by wind forces is given under *H77* for span when in final position, and under *H235* when in lowest position, (neglecting weight of lifting apparatus).

Longitudinal Wind.

The longitudinal wind has been assumed equal to ½ the transverse wind force (i.e., for 1 lb. wind as 11,000) which is equivalent to assuming an exposed surface equal to the cross sectional area of the span.

in the diaphragms and ⅞ in. in the links with a lateral clearance of ½ in. on each side of the pins in the diaphragm pinholes and ¼ in. on each side of the pin in the link pinholes. Fourteen-inch diameter holes to correspond to the holes in the jacking pin diaphragm, were bored in the web of the hydraulic jack diaphragms for introducing the jacking pins.

Near the bottom of the link diaphragms in the lower girders were 3 1/16-in. diameter holes 2 ft. 0 in. apart to allow 3-in. adjustable screw guides to protrude. These guides were used to prevent the links from swaying in the jacking girders. At the top of the upper girders notched guides were used to keep the links properly centered on the upper jacking pins.

On the outside of the lower jacking girder and directly opposite the link diaphragm were brackets supporting the safety follower-up screws, the upper jacking girders having similar brackets to deliver the load. These forged-steel safety screws, four in each lower jacking girder, were 12 ins. in diameter, about 4 ft. 0 in. long with a 2-in. pitch standard Acme thread. The screws engaged a forged-steel nut nested in the lower diaphragm. They were counterweighted and turned by man power, one man operating each safety jack.

The safety screws last year were only two in number at each corner of the span, and they were situated between the stiff box guides on the longitudinal centre line of the jacking girders 7 ft. 6 ins. apart. The purpose of these following-up safety screws was to provide a safety device in case anything should go wrong with the pumping system for the hydraulic jacks or the jacks themselves, and in case they should fail to maintain the pressure necessary to hold the weight of the span while being lifted. The upper jacking girder would then rest squarely on the safety screws which would take the full load and release the hydraulic jacks. The four screws used this year at each corner gave a much more stable design than the two screws of last year's design. The counterweighting of the screws was done in order to practically eliminate any friction due to their own weight and the operator of the screws was able to turn them without difficulty and follow the lift of the hydraulic jack with equal speed and very little exertion.

The guide hangers connecting the upper stiff hangers to the lower jacking girders were each made of a box 3-ft. 9-ins. by 2-ft. 0¾-in., and are composed of two 44 x ⅝-in. webs and four 8 x 8 x ⅝-in. angles with two 23 x ⅜-in. and two 21 x ⅜-in. flange plates. The angles and flange plates were riveted to the webs of the lower jacking girder and the upper ends of the 44 x ⅝-in. plates have pinholes 14 ft. 7⅝ ins. above the top flange angles of the same girder, to connect to the stiff upper hangers. These pinholes are reinforced with pin plates to a thickness of 3⅝ ins., giving a stress through the pinhole due to dead load of 9,000 lbs. per square inch, and a bearing on the pin of 17,200 lbs. per square inch.

The stiff upper hangers, which were also strengthened over last year about 25 per cent., and were braced together by a double intersection system of lattice angles, were of box section 2 ft. 4⅛ ins. by 2 ft. 0 in. The main webs were each composed of two 27 x 11/16-in. and one 27 x ½-in. plates with 12-in. pinholes 43 ft. 4¼ ins. centre to centre, each pinhole being reinforced by pin plates to a thickness of 5 5/16 of an inch. The stress through the net section of the head for dead load was 11,300 lbs. per square inch, and the bearing pressure behind the pinhole was 12,000 lbs. per square inch. The flanges were composed of 4 x 4 x 9/16-in. angles and two 24 x 5/16-in. cover plates, 39 ft. 10½ ins. long, these latter being nickel steel taken from the north shore traveler.

The upper supporting girder which took the load from the stiff upper hangers and delivered it to the upper key bearing, situated on top of the CUO joint, was a box girder 14 ft. 3 ins. long, 7 ft. 6 ins. back to back of angles in depth and 3 ft. 8 ins. centre to centre of webs. At each end 11 ft. 6 ins. centre to centre are double cross-diaphragms extending from the top flange through the girder to 1 ft. 6 ins. below the bottom flange angles. The centre of the 12-in. pins connecting the cross diaphragms to the upper hangers is 1⅞-ins. above the back of the lower flange angle. At the centre of the girder a cross diaphragm faced to bear at the bottom relieves the girder webs of part of the load and tends to distribute the load along the key bearing.

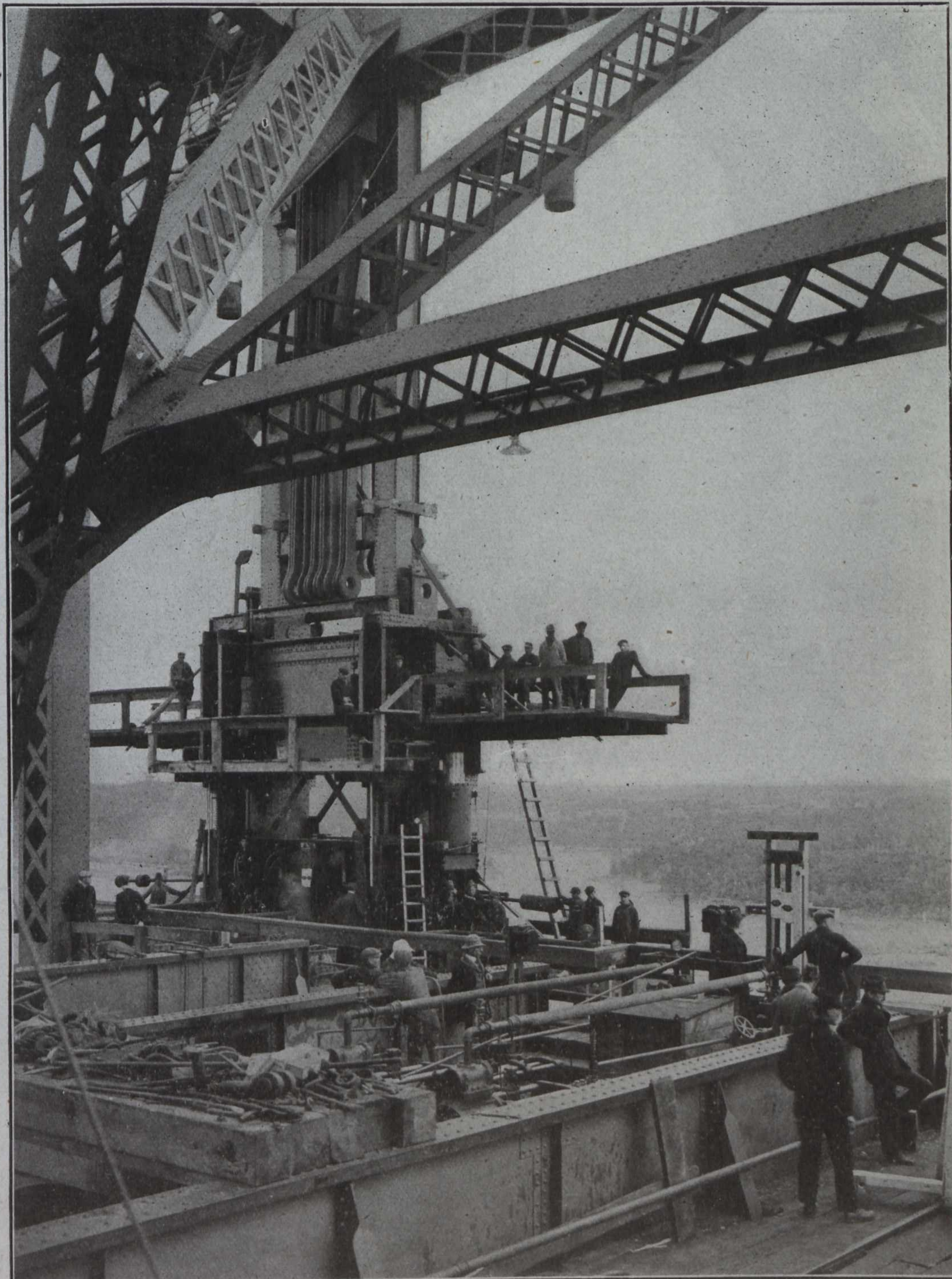


Fig. 6.—View of Jacking Girder, Hydraulic Jacks, Safety Screws, Central Tell-Tales, Pumps, Operating Valves and CMO Joint Above Upper Jacking Girder (ELG3)

The girders were the same ones that were used last year, strengthened and reinforced. They were made up of two web plates 89¼ x ¾-in. reinforced on the outside with two 73 x ⅜-in. plates running the whole length of the girder and the full depth between the flange angles. The flanges were each made up of two 8 x 6 x ⅝-in.

angles outside, and two 8 x 6 x ½-in. angles inside with the 8-in. leg vertical and cover plates 57 x ½-in. x 9-ft. 0½-in., and two 13 x ½-in. plates 8-ft. 0½-in. long.

The double cross diaphragms at the ends were composed of two ribs, each made up of a web plate 42 x ¾-in. by 7-ft. 6¼-ins., extending from 1-ft. 6-ins. below the girder to 1-ft. 5¾-ins. below the top flange and connected to the girder by four 6 x 4 x ½-in. angles, the 6-in. leg connecting to the diaphragm. The bottom of the web was reinforced with pin plates to a thickness of 3¾-ins., the distance between the ribs being 1-ft. 8¼-ins. The upper hangers fitted inside the protruding stubs which were braced by 21½-in. x ⅜-in. plates 1-ft. 5-ins. long, riveted to the angles and connected to the bottom flange. The stress through the net section of the head for dead load was 11,400 lbs. per sq. inch, and the bearing on the pin 17,000 lbs. per square inch.

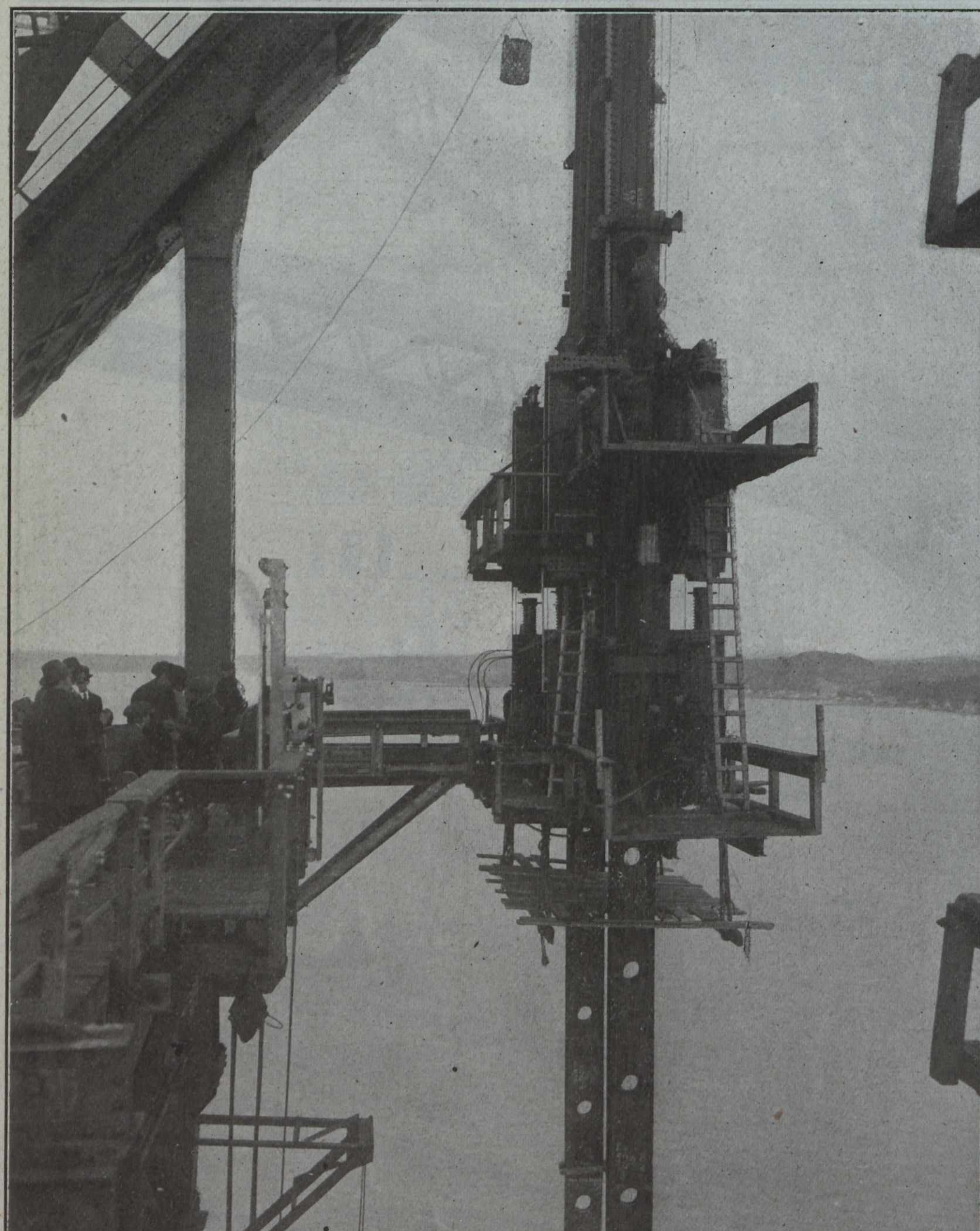


Fig. 7.—End Elevation of Jacking Equipment, Showing Hydraulic Jacks in Top Position, also Showing Safety Screws Not Quite in Contact with Lifting Girder. During Lifting Operations it was Intended that an Operator Should Attend to Each Safety Screw and Keep it Close Up Against the Lifting Girder ELG3

The centre diaphragm was made up of two 41 x ½-in. webs 7 ft. 4⅝ ins. long connected to the girder by four 8 x 8 x 1-in. angles with rivets in double and quadruple shear, the web being heavily reinforced at the bottom to a total thickness of 4½ ins. and stiffened with stiffener angles. Outside the girder web are short stiffeners with heavy fillers. The whole was fabricated to give a good square bearing on the bearing shoe plate.

Operation of Lifting the Span

The operation of lifting the span was accomplished in stages of 2-ft. 0-in. lifts.

The span being supported by the lifting links on pins in the lower jacking girders and the upper jacking girders resting on the hydraulic jacks in their low position, with the safety screws also in their low position, the upper jacking pins were introduced, engaging all holes having the same number in the lifting link and the same letter in the diaphragms.

The release valves being closed, the power was turned on slowly until the upper jacking pins were carrying the load and the lower pins cleared so that they could be removed. The power was then turned off, the fluid being prevented from flowing back by the check valves in the piping system. The lower jacking pins were then removed and the safety screws brought up close to the upper girder. The power was then turned on and the span slowly lifted the two-foot interval by the hydraulic jacks, the safety screws following up hard all the time.

When a position had been reached, as indicated by the centre tell-tale, where the lower pins could be introduced, the power was shut off, the pins were driven and the safety screws turned down far enough so that the girder would free the upper pins when lowered.

The release valves were then open, causing the girder to be lowered slowly until the load was transferred to the lower jacking girder and clearance enough for removing the upper pins obtained. The release valves were then closed, the upper pins removed, the safety screws rapidly lowered, the release valves again reopened and the girder lowered to the position for re-entering the upper pins in another hole.

All operations at all four cantilever corners were performed simultaneously, and a rigid supervision was kept to insure that the proper schedule and sequence of operations was carried out. The cantilever arms had direct telephone communication with each other. The power on each side of the river for lifting the span was furnished by three electrically-driven compressors, the capacity of each being 534 cu. ft. of free air per minute, delivered at 110 lbs. pressure per sq. inch, with a normal speed of 165 revolutions per minute. The compressed air was delivered to two 18-in. x 2-in. x 16-in. direct-acting double-plunger pumps. The pumps, situated on the centre line of the span, delivered the fluid into two 1½-in. dia. XX piping systems, each piping system supplying a pair of jacks at one corner of the span.

Each piping system was controlled by a central feed valve having the check valve located close to it, between it and the jacks. Between the central valve and the pump was a cut-off valve, and the two systems were connected together by a cross-over with a cut-off valve, situated between the pump cut-outs and the central valve. This cross-over enabled one pump to feed both systems, and also serve equal fluid pressure from both pumps. Each piping system ran from the check valve to a branch at a cantilever corner. This branch connected into two 1-in. dia. XX pipes, each controlled by a valve situated at the corner. Each 1-in. dia. XX pipe connected through a special union to two flexible ⅜-in. interior diameter copper pipes, which deliver the fluid to the jacks. The

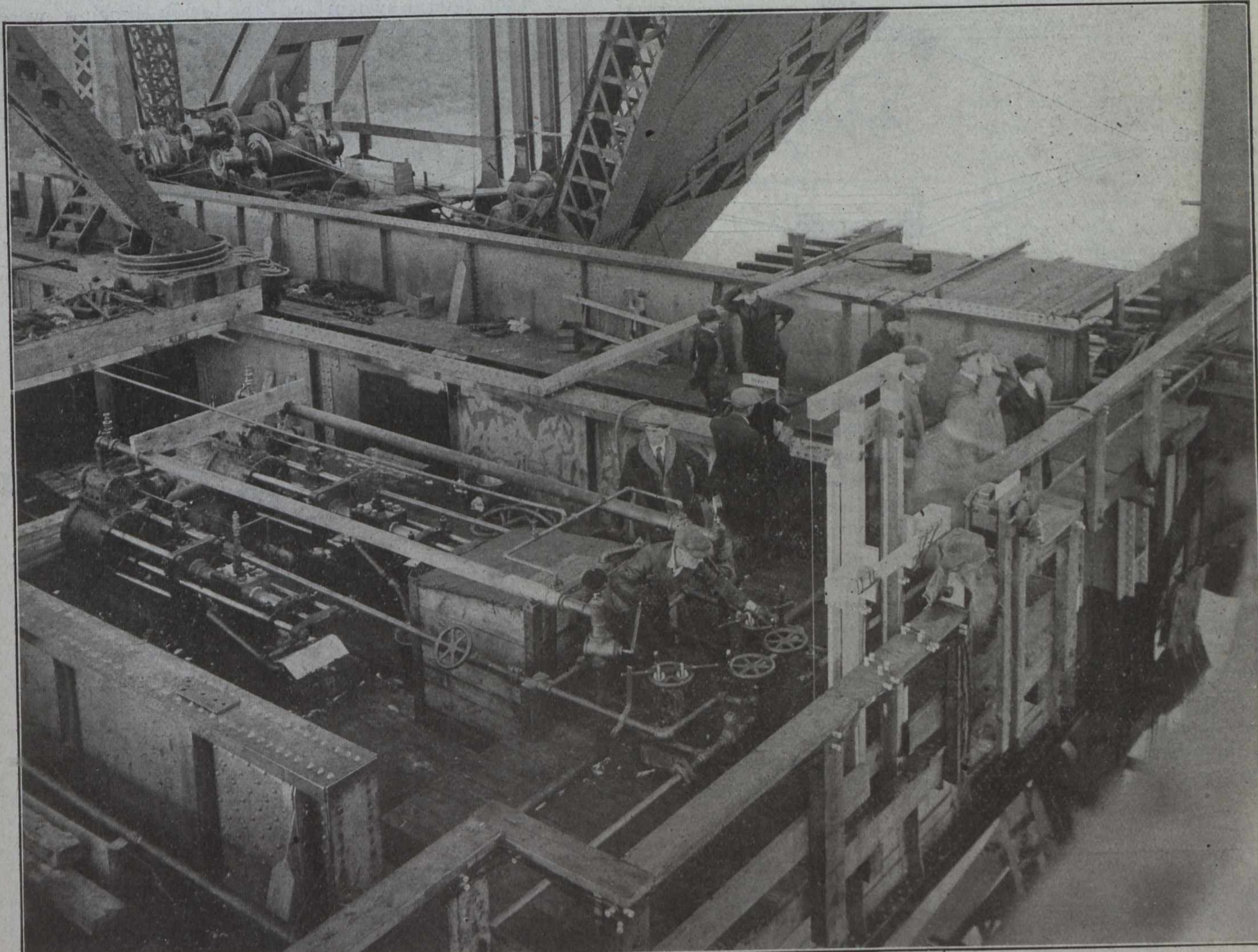


Fig. 8.—View of Hydraulic Pumps and Central Valve Operator's Station, North Cantilever. The Vertical White Framework in Front of Valve Operator Carried the Multiplying Tell-tales. The Canvas-Covered Box Back of that Framework Signalled the Number of the Lift to the Opposite Cantilever as a Check on the Telephones. One Telephone Connected with the Office and the Other with the South Cantilever. Valve Operator's Right Hand is on Air Throttle while Left Hand is Regulating a Gate Valve on Water Pressure Line. He is Intently Watching the Climbing of the All-Important Tell-tales

(Continued on page 262)

Erection and Floating of Central Span

New Span Differs from the One that was Lost Last Year Only In Lower End Joints —Floating Arrangements, Influence of the Tides and the General Plan of Operations

WITH the exception of the XLo joints, which were redesigned to suit the new form of bearings, and which were also strengthened, the new suspended span of the Quebec Bridge is exactly the same as the former span. It is 640 ft. long centre to centre of end supports, 88 ft. wide centre to centre of trusses, 113 ft.

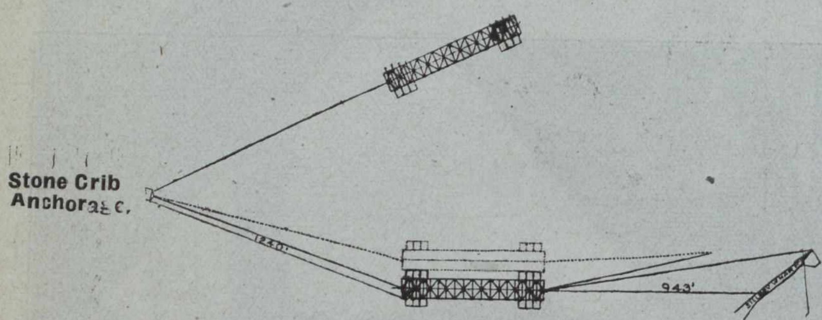


Fig. No. 1.—Diagram Showing Method of Anchorage While Floating Span Away from Erection Site

high overall, and will weigh about 5,600 tons when completed with floor system, stringers, track, etc.

As lifted, the weight of the permanent structure was 4,831 tons, but there were 20 tons of erection steel on the span and 69 tons of timber, hoists, etc. The lifting girders weighed 160 tons, so that the total load carried by the hanger chains was 5,080 tons.

The top chords are in the form of a parabola, the depth of the trusses at the hip being 70 ft. and at the middle of the span 110 ft. centre to centre of chords. The web is a subpanel Pratt system, the main verticals being compression posts, and the main diagonals, tension members. The main panels vary in length from 65 ft. in the end panels to 80 ft. for the panels at the centre of span.

For the bottom chords throughout and for the first main tension diagonals of the web, eye-bars were used All the other truss members were of built-up construction. The top chords were pin-connected at all the main panel points with shop or field splices at the intermediate subpanel points. The web members were connected and riveted together at the main panel points of the top and bottom chords by means of gusset plates which engage the pins of the top and bottom chord members.

Nickel steel was used throughout for all the main truss members and the bottom lateral system, but the top lateral system and the sway bracing were built of carbon steel, as well as the minor web members which carried no moving-load stresses.

The top and bottom lateral systems, as well as the sway bracing, are double intersecting systems, designed to take both tension and compression in each member.

The erection site for the suspended span was the same as used last year,—about three miles below the bridge site, in the shallow waters of

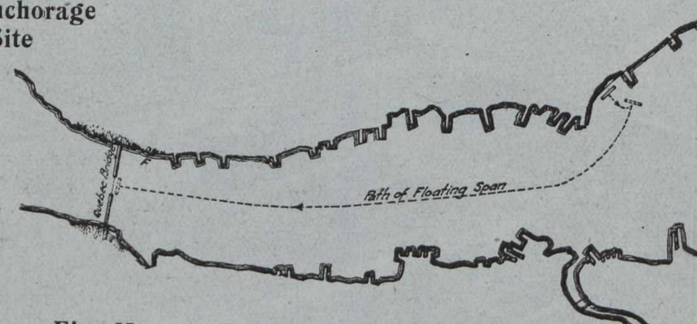


Fig. No. 2.—Path of Floating Span from Erection Site to Bridge

Sillery Cove, where the bed of the river is exposed at low tide. The old falsework bents, approach tracks, etc., were used again.

During erection, the span was supported on staging bents placed under each panel point. These bents each consisted of two columns, spaced 6 ft apar and tied together by batten plates and latticing. In designing the

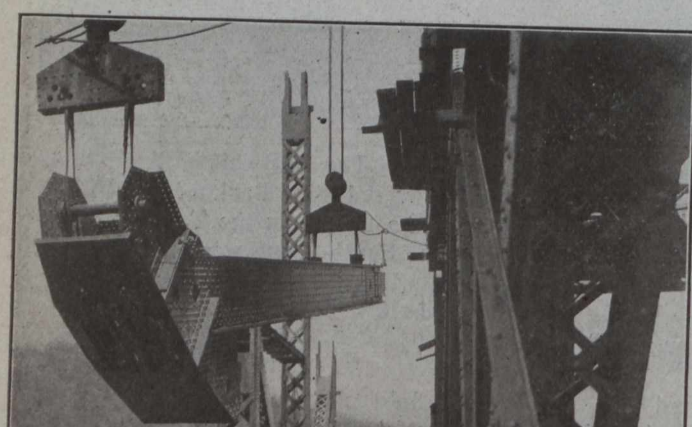


Fig. 3.—Traveller Placing End Batter Post and XL0 Joint of Suspended Span

Fig. 4.—Driving Pin Connecting Bottom Chord Eye Bars at XL0 Joint

columns it was assumed that the total weight of the bridge material, the weight of the traveler, including the member being erected, and the vertical reaction from wind on the traveler and span were carried by the outside columns of the bents.

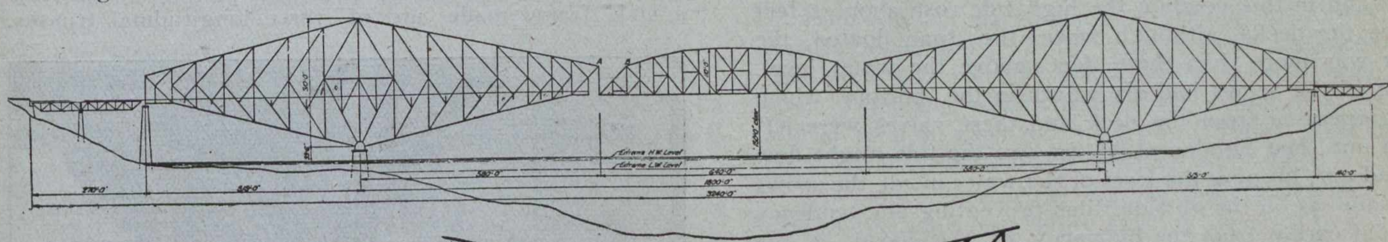
As the traveler moved forward, it erected the staging and longitudinal bracing, floor-beams, bottom chords, bottom laterals and web members, except the upper half of the main diagonals and the vertical sub-struts. On its backward trip the traveller completed the erection of the trusses by placing the top chords and upper web members as well as the top laterals and sway bracing.

Sand jacks were used at the even panel points, directly under and with tops bolted to the vertical posts of the trusses, to transfer the load to the outer columns of the staging. Timber blocking was used for the same purpose at the subpanel points and also between the floor-beams and inside columns of the staging at all panel points. The vertical posts and vertical hangers into which the floor-beams framed had their inside webs slotted to allow the end web plates of the floor-beams to pass through. In order to make this connection the floor-beams were placed on timber blocking over the inside staging columns, about 3 ins. above their camber position; and after placing the vertical posts or hangers and bolting the floor-beam connection, the floor-beams and verticals were together lowered to their correct camber positions to a bearing on the sand jacks or timber blocking over the outside staging columns by means of hydraulic jacks.

The span being completely assembled, the timber blocking at the intermediate staging supports and under the floor-beams was taken out, and the sand jacks were lowered until the span rested on its end supports. The top-chord members then came to a bearing on the pins in the half-pin holes at the main panel points; and the bearing surfaces at the field splices came together, the nuts on the bolts attaching the splice material having previously been loosened. The field rivets in these splices were then driven. All the vertical subposts supporting the top chord members at the subpanel points were made of such a length that these chord members would be straight between main-panel points under the dead weight of the span as it rested on its end supports.

After the suspended span of the Quebec Bridge had been completely assembled and riveted, it was allowed to rest on the end staging bents, while three scows were floated, at high tide, under each end of the span and allowed to rest with the receding tide on concrete and timber foundations prepared for them at a height that permitted blocking and steel girders for distributing the load upon the scows, to be placed between the scows and the span.

The scows, six in number, have heavy steel frames



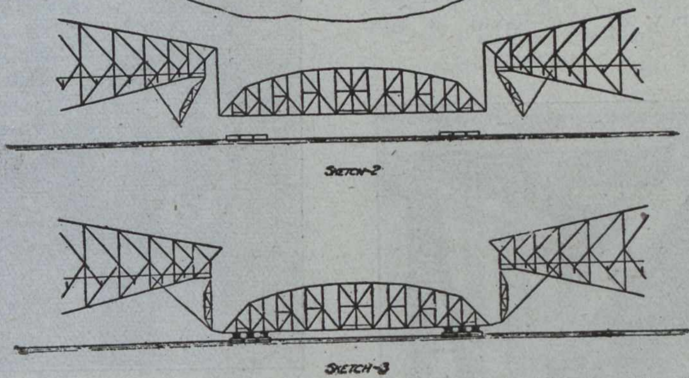


Fig. 5.—Three Stages in the Lifting: (1) In Final Position; (2) In Position of Late Tuesday Afternoon Before Mooring Trusses Were Lowered for Adjusting Mooring Tackle for Safety at Night or During High Winds; (3) When Moored but Not Yet Lifted from Scows. Lifting Chains Not Indicated in Third Sketch

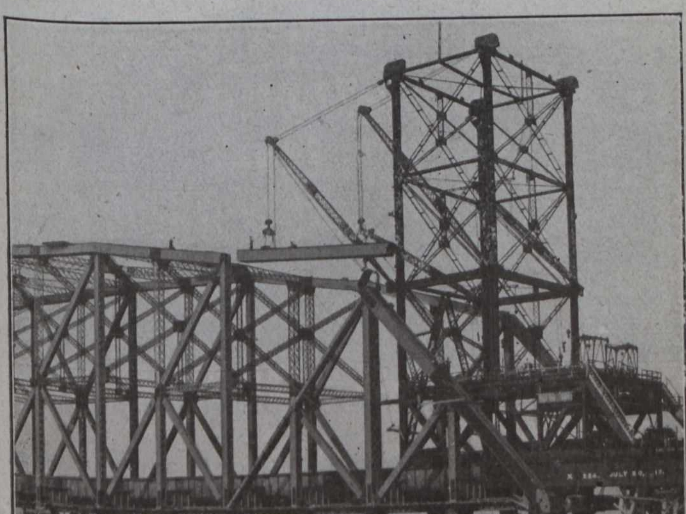


Fig. 6.—Traveller Placing Final Section of Top Chord of New Suspended Span

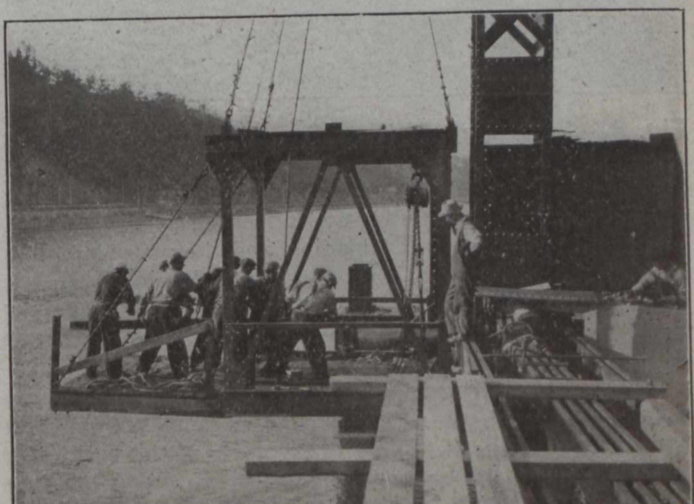


Fig. 7.—Showing Type of Riveting and Pin Cage Used During Erection of Span

and steel plate girder bulkheads calculated to support the large concentrated loads to which they have twice been subjected, the same scows as were used last year, being used again this year.

The scows required about 8 feet 2 inches of draught to float the span; their bottoms were placed at elevation 83, a considerable distance above low water at spring tides and in this position the high tide rose about 2 feet above the decks, although when the span floated, the deck was more than three feet above the water level. Each of the scows had six valves in the bottom which were operated from the deck and these valves were left open until last Sunday night, so that the tide might flow in and out, keeping the level of the water inside the scows the same as on the outside, thus preventing any tendency to float except from the buoyancy of the wood in the timber skin.

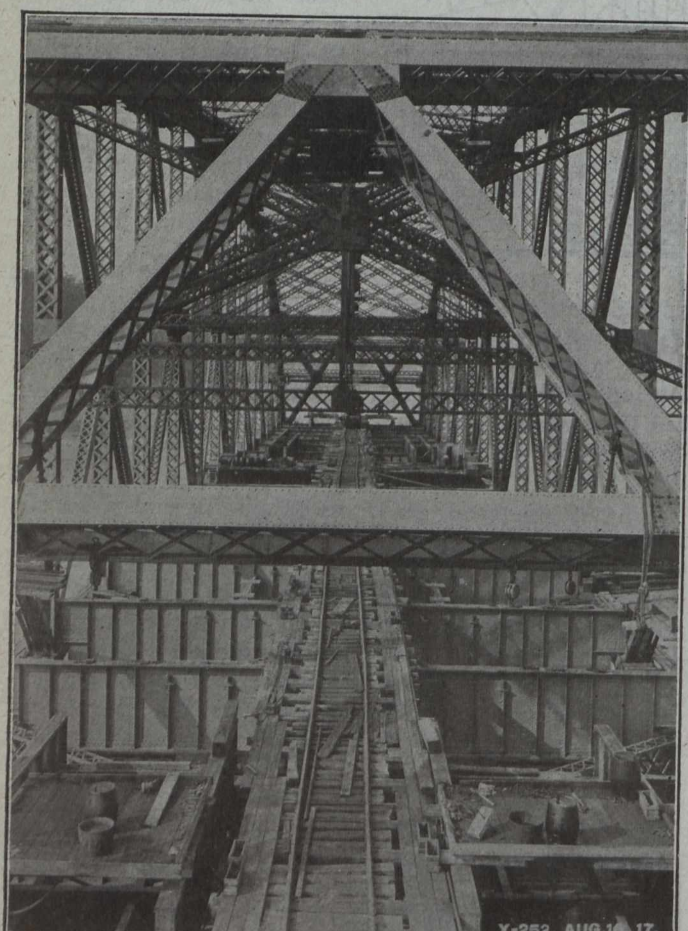


Fig. 8.—View Through New Suspended Span, Showing Sway Bracing

The design of the scows was governed by the arrangement and requirements of loading and the possible condition of the surface of the river during floating-in operations; also so that they might have some commercial value now that their work of floating-in the suspended span has been completed.

The average length from crest to crest of wave at the bridge site is about 40 ft.; the maximum wave height allowed for was 4 ft. This unevenness of the surface of the river produced unequal upward pressures at the four corners of the span and consequent stresses in the sway and lateral bracing. The inequality of pressure was proportional to the horizontal cross-section of the loaded scows near the surface of the water. To reduce wave effect as much as possible, long, narrow scows with a deep draft were used. With the design of scow adopted the oscillation of the span from wave action produced only stresses in the sway and lateral bracing, which these systems were well able to resist.

The scows are 32 ft. 5½ ins. wide, 164 ft. 6 ins. long and 11 ft. 7½ ins. draft over bilge timbers. Each has a steel frame made up of three longitudinal trusses,

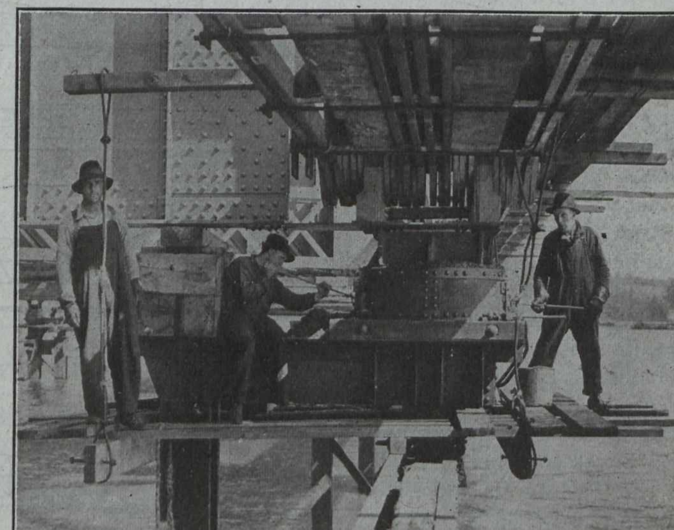


Fig. 9.—Removing Sand from Sand Jacks on Top of Intermediate Supporting Bents of Suspended Span

spaced 10 ft. 6 ins. centre to centre and braced transversely by four water-tight steel bulkheads with intermediate cross-frames between the bulkheads, spaced 8 ft. 4 ins. centre to centre. No special longitudinal bracing in the horizontal planes was provided, as the 11½ x 5½-in. cross-timbers, spaced 2 ft. 9 ins. centre to centre were bolted directly to the steel framework of the scow; and the 4-in. timber covering was spiked to these cross-timbers with 8 x 7/16-in. boat spikes, three at each inter-

Fig. 10.—Suspended Span Erected at Sillery Cove

section, providing an efficient resistance to any transverse or longitudinal horizontal-shearing and bending forces.

The load of the suspended span was transferred to the bulkheads by means of cross-girders and I-beams. The bulkheads transferred this load to the longitudinal trusses, which distributed it over the length of the scows.

As previously mentioned, the weight of the span as lifted was 5,080 tons. The total load on the six scows, however, was 5,753 tons, or about 959 tons per scow, as the material between the scows and the span weighed 673 tons. This consisted of 73½ tons of track stringers, 1 ton of shims between floor beams and stringers, 327 tons of track girders, 10½ tons of track girder bracing, 211 tons of staging posts and hitch channels, and 50 tons of timber.

The elevation of the beds of the scows was high enough to ensure that the scows would be emptied through the bottom valves during the last low tide before floating the span into position. The water drained out about as fast as the tide fell, as each interior area of the scows had unobstructed access to a valve.

Just before the span was lifted off the end supports, the load was nearly all taken by the scows, and the span could have been easily displaced from its position by the current and wind, unless it was anchored against their combined effect. It was considered desirable to prevent this shifting off before the actual moment of starting arrived, inasmuch as it might have happened that after deciding to raise the span preparatory to moving out, a change in the weather conditions might have made it desirable not to proceed on the journey and the span would have had to have been returned again to the staging bents, to await the next favorable opportunity for making a trial.

To keep the span in its position until the final decision to float away was made, timber bents were placed between the lower end panel points and the adjacent scows and also bents on the shore side of the span, against which the scows guided themselves as the span was raised on its supports.

In order to float the span a high-tide elevation of at least 92 feet was required, and, in order to drain the scows, a previous low tide elevation of not more than 82 feet. The bottom of the scows were at elevation 83 feet, and the bed of the river over the course the scows were to take was cleared off to elevation 82. Inasmuch as elevations of high and low tide may vary 2½ feet at the erection site of the suspended span, a tide would have been preferred whose elevations as given by the tide tables would have been 94.5 feet at high tide and 79.5 feet at low tide. The tides of September 3rd met those conditions better than the tides of any other day of this month, but the hoisting arrangements were not completed in time to take the opportunity. The tides of the 15th to 19th were the next most favorable, and had not the span been floated some one of those five days, about two weeks would have had to elapse before elevations would have re-occurred suitable both to drain the scows and to float the span.

Elaborate preparations were made to foretell the weather conditions. A full statement of meteorological conditions was received and carefully studied twice each day. Telephone communication was obtained at 10 a.m. and at 10 p.m. each day with the Toronto Observatory. Barometric observations were also taken daily.

The appearance of the sky and the velocity and direction of the wind just before starting, were also well considered before deciding to start. It was estimated that any winds which would exert a greater pressure than 2 lbs. per square foot could be foreseen, and in that event no start would have been made. The current velocity at the bridge site is a maximum at one hour before high tide, and flows westward at a rate of 6.3 to 7.3 miles per hour. At high tide the current velocity is less by about 1 mile per hour. The change of current from a westward to an eastward direction, when the velocity is zero, occurs about 1 hour after the time of high tide.

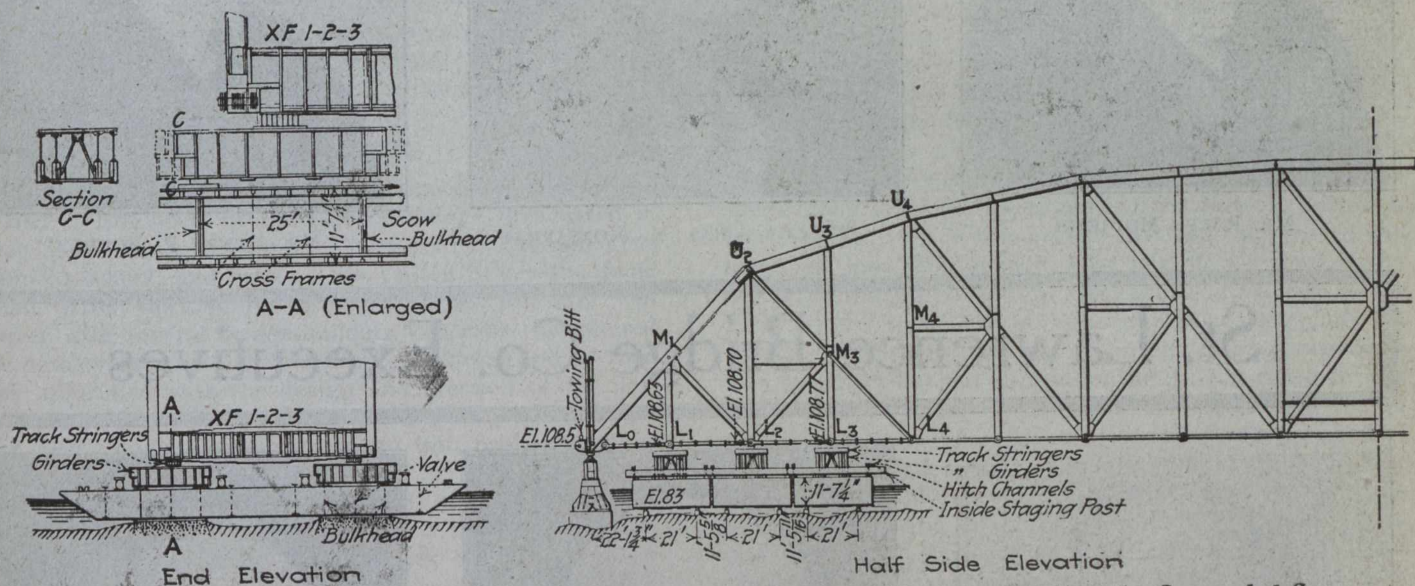


Fig. 11.—Details of Scows and the Intermediate Supporting Steel Used for Floating the Suspended Span

QUEBEC BRIDGE EQUIPMENT

E. C. Kerrigan, chief draftsman of the St. Lawrence Bridge Co., has supplied the information for the following partial list of equipment used in building the Quebec Bridge:—

Hoisting equipment, including hydraulic jacks, pumps, valves, piping and safety jacks, by the Watson-Stillman Co., Aldene, N.J.

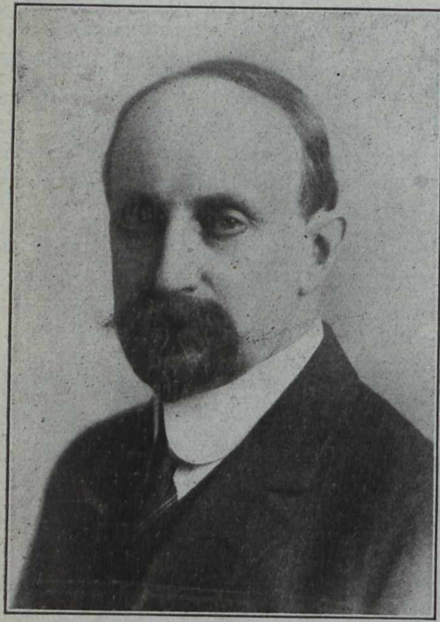
Locomotive cranes, by the Bay City Iron Co., Bay City, Mich.

Wire rope, by the Dominion Wire Rope Co., Limited, Montreal.

Motors, by the Canadian General Electric Co., Limited, Toronto.

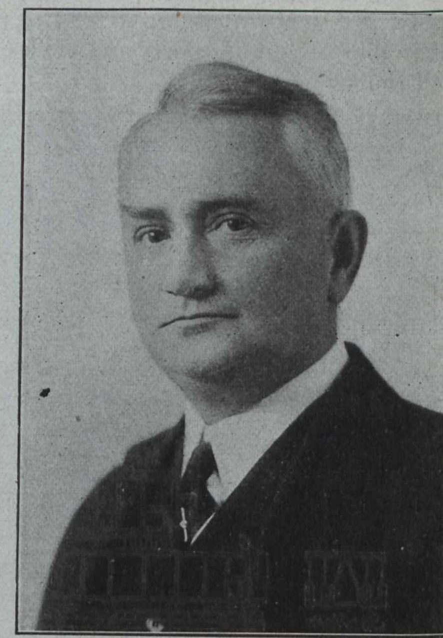
Electric hoists, by Mead-Morrison Manufacturing Co., Cambridge, Mass.

Board of Engineers, Quebec Bridge



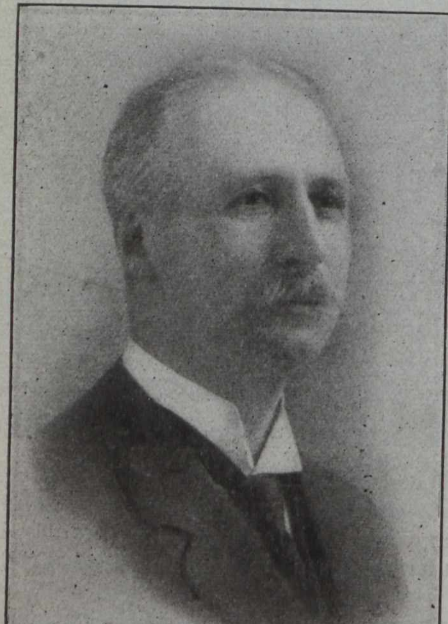
MR. RALPH MODJESKI.

LT.-COL. CHAS. N. MONSARRAT.

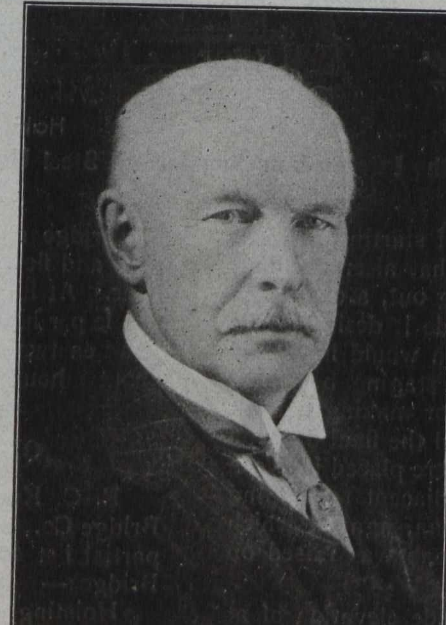


MR. HENRY P. BORDEN.

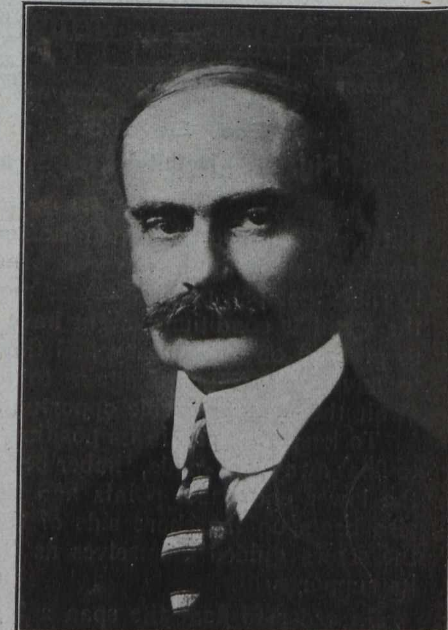
St. Lawrence Bridge Co. Executives



MR. GEO. H. DUGGAN.



MR. PHELPS JOHNSON.



MR. GEO. F. PORTER.

TO these six men the success of the world's biggest and most unique bridge is very largely due. In the Quebec Bridge they have developed the science of bridge design far beyond anything previously attempted. At the same time they have placed shop and field methods upon a more systematic and highly organized basis than has probably ever previously obtained in such work. These facts, coupled with ingenuity and daring in erection methods, marks the Quebec Bridge as one of the outstanding engineering feats of the century. Yet it is hardly fair to say that they are the builders of the bridge, because so many engineers have contributed such valuable ideas to the structure, and all their efforts have been so interdependent, that there are easily twenty-five men whose photographs should be included in any comprehensive illustration of the brains behind the Quebec Bridge. Mitchell, McMath, Kerrigan, Schneider, Fortune, Meyers, Mayer, Vautelet, Davis, Harkness, Atkinson, Rankin, Copp and McMillan, are only a few of the many outstanding names that crowd into one's memory when speaking of the men responsible for the great Quebec Bridge.

SNAPSHOTS ON THE JOB

PHELPS JOHNSON, president of the St. Lawrence Bridge Co., is the designer of the K-truss system used for the Quebec Bridge. He was the organizer of the St. Lawrence Bridge Co. and the man who decided that there should be at least one bid from a Canadian company for the building of such an important Canadian structure. He evolved the idea of floating and hoisting the suspended span into position, and he was the executive head of the whole big organization that planned and carried out the great undertaking. Mr. Johnson is president of the Dominion Bridge Co., having succeeded the late James Ross about four or five years ago. He joined the Dominion Bridge Co. as vice-president thirty-five years ago, after having had several years' experience with three different United States bridge companies. After having acted for some years as vice-president of the Dominion Bridge Co., Mr. Johnson undertook the duties of chief engineer, and was for a considerable period very active in the design and erection of many of Western Canada's most important structures. Subsequently he returned to Montreal as vice-president and general manager of the company, which office he filled until he succeeded to the presidency.

* * * * *

RALPH MODJESKI, one of the three memebers of the Board of Engineers of the Quebec Bridge, graduated in 1885 at the Ecole des Ponts Chausses, the Parisian government school, and went to the United States the same year. After working for seven years as assistant engineer with several bridge-building concerns, he entered the employ of a firm of consulting engineers and in 1900 was associated with the design and erection of the first Memphis bridge. After seven years with the consulting firm, Mr. Modjeski resigned to go into private practice on his own account, and established an office in Chicago in 1903. He designed and erected the Rock Island bridge, the Columbia River bridges on the Pacific Coast, the new Memphis bridge, the Thebes bridge and many others. During his career Mr. Modjeski has designed and erected bridges having a total cost of approximately $30,000,000.

* * * * *

H. P. BORDEN is the most recently appointed member of the Board of Engineers. He was assistant engineer of the commission from the time of its inception in 1908, and rendered such valuable assistance that Col. Monsarrat and Mr. Modjeski advocated his appointment by the government in February, 1916, to fill the vacancy caused by the death of Mr. Schneider. While assistant engineer, Mr. Borden wrote a considerable number of articles on the Quebec Bridge for *The Canadian Engineer* and other technical papers. He is a graduate of McGill University, having obtained his degree in 1902. After 1½ years spent with the C.P.R. as architectural engineer, he was for three years assistant chief engineer of the structural department of the Montreal Locomotive Works. He then rejoined the C.P.R. as assistant engineer of the bridge department, resigning that position to join the commission under Mr. Vautelet.

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GEORGE H. DUGGAN, chief engineer of the St. Lawrence Bridge Co., is also vice-president of the Dominion Bridge Co. He graduated at S.P.S., Toronto University, about 1883, and a few years later entered the employ of the Dominion Bridge Co., becoming chief engineer in 1891. In 1902 he resigned to become assistant to the president of the Dominion Iron & Steel Co., and in 1904 was appointed second vice-president and general manager of the Dominion Coal Co. About six years later he rejoined the Dominion Bridge as general manager. Mr. Duggan is also a director of the Royal Bank of Canada, the Montreal Trust Co., the Hillcrest Collieries, and many other engineering and business concerns.

* * * * *

F. C. MCMATH, consulting engineer of the St. Lawrence Bridge Co. since its inception, is the president of the Canadian Bridge Co., of Walkerville. The government had invited both the Dominion and Canadian bridge companies to tender on the new Quebec Bridge, but Mr. McMath thought that if the Canadian bridge companies were to present a solid front in the bidding—in other words, to pool their organizations and experience and facilities—that Canada would have a better chance of being successful in the bidding, and his company therefore joined with the Dominion Bridge Co. as joint owners of the capital stock of the St. Lawrence Bridge Co., and the only bids made by any Canadian firm were submitted in the name of the St. Lawrence Bridge Co. Mr. McMath was the founder of the Canadian Bridge Co. After his graduation at Washington University, in Missouri, he spent many years in the employ of the Detroit Bridge & Iron Works Co. When he left that firm he crossed the boundary line and founded what has since grown to be one of the largest structural steel plants in Canada. Mr. McMath has given careful consideration to all plans for the Quebec Bridge, and his entire engineering organization was placed at the disposal of the St. Lawrence Bridge Co. to aid in checking calculations and design and in assisting in erection wherever needed. Two of Mr. McMath's sons assisted in the operation of the jacks last year, and one of them had a narrow escape at the time of the accident. In fact, Mr. McMath, himself, was standing on the connecting walk leading to the jacking platforms when that walk was smashed, and he was forced to jump to the cantilever. This year one of his sons is operating the control valves at one corner. The other son is at the Front with the Aviation Corps.

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GEORGE F. PORTER, superintendentt of erection for the St. Lawrence Bridge Co., began his bridge-building career, as have so many other well-known structural men, with the Detroit Bridge & Iron Works. After five years there he went to Pittsburgh, with the Keystone Bridge Works, but later rejoined the Detroit firm, where he stayed until Mr. McMath persuaded him to cross the river and become chief draftsman for the Canadian Bridge Co. at Walkerville. When the Quebec Bridge Commission was formed in 1908 he was engaged as chief draftsman. At that time a number of bridge companies "loaned" men to the commission upon a tacit understanding that the successful bidder would be allowed to "borrow" any members of the commission's engineering staff who might be needed in carrying out the contract. When the St. Lawrence Bridge Co. were successful in the bidding, they "drafted" Mr. Porter, who was appointed as assistant to Mr. Johnson and Mr. Duggan in directing the design of the bridge and later in superintending its erection. Mr. Porter is the only one of the higher officials of the St. Lawrence Bridge Co. who has devoted his whole time exclusively to the Quebec Bridge and he is therefore exceedingly well informed in regard to every minute detail of its design and erection.

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Lieut.-Col. C. N. Monsarrat, chief engineer and chairman of the Board of Engineers, Quebec Bridge (or, as it was formerly called, the Quebec Bridge Commission), was a member of the class of 1888 at McGill University. He joined the C.P.R. bridge engineering staff as an inspector of erection, and remained for twenty years a member of that department. From 1903 to 1911 he was head of the department, resigning in 1911 to succeed Mr. Vautelet on the Quebec Bridge Commission. He is the commanding officer of the Fifth Royal Highlanders of Montreal.

* * * * *

Chas. Cassils, the vice-president of the St. Lawrence Bridge Co., is a well-known Montreal citizen, having been prominent in business circles there for the past forty years. When the company was first formed, Mr. McMath, president of the Canadian Bridge Co., was vice-president and consulting engineer, but as Mr. McMath's headquarters are at Walkerville, the arrangement as regards the vice-presidency was found inconvenient, due to certain duties of the office which developed and which would have required his almost constant residence at Montreal, so he resigned in favor of Mr. Cassils, whose wide business experience has been of great value to the organization in many ways. Mr. Cassils was in the iron and steel business in Scotland as early as 1853. He came to Canada in 1873 and for many years was Canadian representative of the Carnegie Steel Co. He is a director of the Northern Electric Co., president of the Structural Steel Co., and a director of a large number of other general business firms, such as the Dominion Transport Co., the Windsor Hotel Co., etc., etc.

W. B. Fortune, the superintendent of erection for the St. Lawrence Bridge Co., has been engaged in structural steel erection for the past thirty years. He started with the Edge-Moor Bridge Works, of Wilmington, Del., and when he left that firm he was its general foreman. After ten years with the New York Shipbuilding Co., Mr. Fortune became superintendent of the Seaboard Construction Co. and since then has superintended the erection of most of Mr. S. P. Mitchell's important structures. He has been engaged on the Quebec Bridge work for the past five years.

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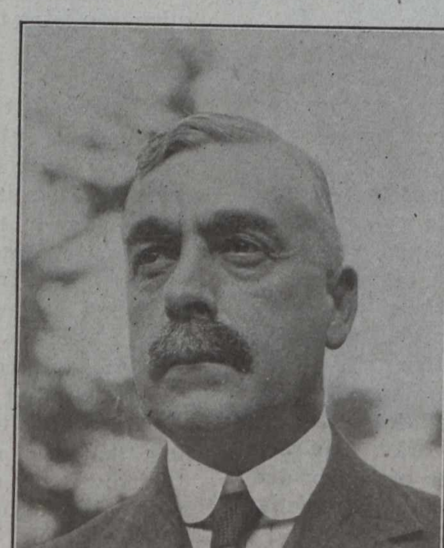
Ernest C. Kerrigan, chief draftsman of the St. Lawrence Bridge Co., has had upwards of twenty years' experience with various Canadian structural companies. For some years previous to the formation of the St. Lawrence Bridge Co., he was with the Canadian Bridge Co. at Walkerville. He was "loaned" by Mr. McMath to the new company, and has proved a valuable asset, having directed the purchasing department, the sale of used equipment and many other departments besides the drafting room.

* * * * *

S. P. Mitchell, the consulting engineer of erection for the St. Lawrence Bridge Co., graduated from the University of Virginia in civil engineering, class of 1883. He was engaged in railway engineering until 1887, when he became engineer of erection of the Edge-Moor Bridge Works. In 1890 he was appointed manager of that firm and retained that position for ten years, or until his appointment as chief engineer of the American Bridge Co. In 1906 he resigned in order to go into private practice. Mr. Mitchell is now also president of the Seaboard Construction Co. Among the many other structures with

Three Prominent St. Lawrence Officials

Mr. Ernest C. Kerrigan, *Chief Draftsman.*



Mr. F. C. McMath, *Consulting Engineer.*

Mr. W. B. Fortune, *Gen. Supt. of Erection.*

which he has been identified are the Beaver Bridge on the P. & L.E. Railway and the Sciotoville Bridge across the Ohio River, near Cincinnati.

* * * * *

JOSEPH MAYER, principal assistant engineer of the Board, is the chief mathematical expert of the Quebec Bridge. He has had charge of the derivation of all formulæ and has supervised all the important calculations. Most of his time on the work has been occupied in the preparation of theses on the theoretical problems involved, and in the design of certain difficult details. In other words, he has been in charge of what might be called the "pure science" end of the commission's work. Mr. Mayer graduated from the University of Vienna in 1879 and went to the United States the following year. He was with the Delaware Bridge Co. for four years, and then with the Union Bridge Co. for eleven years, latterly as chief engineer. He resigned that position to open a consulting office in New York, and has prepared the designs for a large number of bridges, including the Poughkeepsie Bridge, the Kentucky and Indiana Bridge across the Ohio River at Louisville, the Kentucky River Bridge, the Kanawha River Bridge in West Virginia, and other cantilever bridges. He also prepared the designs for a firm which bid on the Sydney Harbor, Australia, cantilever bridge, and for both cantilever and suspension bridges across the Hudson River at New York, the cantilever bridge having 2,000 ft. span and the suspension bridge 2,800 ft. span. Mr. Mayer came to Canada in 1908, to join the Commission, upon invitation from Mr. Modjeski. He became a Canadian citizen in 1913.

* * * * *

A. J. MEYERS, the Board's chief draftsman, graduated from McGill University in 1902. He spent the next five years acquiring experience with a number of structural steel companies, including the Dominion Bridge Co., the Hamilton Bridge Co., Milliken Bros., of New York, Riter-Conley Co., of Pittsburgh, and McClintic-Marshall Co., of Pittsburgh. In 1907 he joined the Structural Steel Co., of Montreal, as chief draftsman and in 1909 went with the Board as a calculator and designer. In 1911, when Mr. Porter left the Board to become superintendent of erection for the contractor, Mr. Meyers succeeded him as chief draftsman. During the past couple years Mr. Meyers has written most of the articles on the Quebec Bridge which have appeared in *The Canadian Engineer* and other technical papers, and he is one of the recognized authorities upon the details of the whole undertaking. By special arrangement, *The Canadian Engineer* is fortunate in being able to present in this issue an article by Mr. Meyers on the key bearings and an article in which Mr. Meyers and Mr. M. Brodie Atkinson collaborated, on the lifting apparatus.

* * * * *

M. BRODIE ATKINSON, who is assistant chief engineer of the Board, graduated from McGill University in 1904 with the B.Sc. degree. After two years with the structural department of the Locomotive & Machine Co., and the successors to that department, the Structural Steel Co., Mr. Atkinson joined the G.T.P. and was first assistant bridge engineer for about 2½ years, while most of the bridges of that road were being designed. In 1909 he joined the Board of Engineers of the Quebec Bridge as chief inspector on the substructural work, and later was appointed assistant chief draftsman. He has been associated throughout with the calculations and design of the bridge. The article in this issue is the first that Mr. Atkinson has consented to write on Quebec Bridge work.

* * * * *

A. L. HARKNESS, assistant engineer of the St. Lawrence Bridge Co., is a graduate of S.P.S., Toronto University, class of 1906. Soon after obtaining his degree, Mr. Harkness entered the employ of the Dominion Bridge Co. in the designing department, and when the St. Lawrence Bridge Co. was formed, he was transferred to that concern.

* * * * *

JOHN RANKIN, the Board's resident engineer, graduated at McGill University, class of 1893, Applied Science. For four years he worked in various rolling mills and then, until 1906, was assistant engineer with various structural steel companies. After about nine years in the C.P.R. bridge engineering department, Mr. Rankin was engaged by the Board of Engineers in 1914, and for the past 2½ years has acted as resident engineer.

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HERBERT MCMILLAN, chief shop inspector for the Board of Engineers, is one of the men who had a very narrow escape in last year's accident. At the time of the fall he was on a lower lifting girder (ELG2) platform. When the lifting girders swung out the connection was broken between the ELG2 platform and the walk leading from it to the cantilever. When the lifting girders swung back toward the cantilever, the broken connecting walk jammed Mr. McMillan against the girder, fracturing a leg. Then the girder and girder platform swung out again. Realizing that a second blow from the connecting walk might strike higher up, Mr. McMillan, despite the broken leg, made a desperate jump and succeeded in landing on the cantilever. Mr. McMillan secured his first engineering experience as a bridge inspector for the C.P.R., subsequently rising to superintendent of inspection of eastern lines. When the St. Lawrence Bridge Co. began work, he was employed for six months as chief shop inspector, then resigning to accept a similar position with the Board of Engineers.

* * * * *

W. P. COPP, the Board's chief field inspector, was engaged in Dominion land surveys for a year after securing his B.Sc at McGill University in 1908, and was then in charge of one of the Dominion Bridge Co.'s drafting offices for four years. In 1913 he was appointed as an inspector for the St. Lawrence Bridge Co., and for the past year has been in charge of all field inspection.

* * * * *

J. W. GHILON, popularly known among the men as "Big Jake," was foreman of erection of the north cantilever and anchor arms, and also of the erection of the suspended span which was lost last year. Most of his career, with the exception of the past three years with the St. Lawrence Bridge Co., has been as general foreman for the Pittsburgh Construction Co. He was foreman of erection of the Caugag Viaduct at Cleveland, Ohio, of the Harlem River improvements at New York City, of the Eighth Coast Artillery Armory in New York City, etc. Last fall, while waiting for the erection of the new suspended span, he was given leave of absence by the St. Lawrence Bridge Co., and since then has erected several bridges on the New York Central Lines and an addition to the Newcastle, Pa., mill of the American Sheet & Tin Plate Co.

* * * * *

W. S. O'BRIEN, foreman of erection of the south shore cantilever and anchor arms and of the new suspended span, earned many a word of praise from the chiefs for the very shipshape manner in which everything at his end was arranged for the hoisting well in advance. Of course, "Bill" was not bothered with so many visitors as were his North Shore confreres, but nevertheless he certainly did set the pace. When the final day came, however, it found both sides equally well prepared and with no further inclination to race each other. Mr. O'Brien is an old Walkerville foreman; and having said that, further remarks would be considered superfluous by those who know Mr. McMath's training ability.

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EUGENE M. FINN has been the official photographer of the St. Lawrence Bridge Co. for the past 2½ years and it is to his skill in photography and his daring in climbing to any position which will afford the best view that the reading public are indebted for most of the splendid illustrations of the Quebec Bridge work which have appeared in various newspapers and journals during the past couple years. Mr. Finn was engaged in commercial photography in Montreal for some years before joining the bridge company, and had experience in securing progress photographs of the Montreal grain elevators and many other engineering structures.

MOORING AND HOISTING THE SUSPENDED SPAN

(Continued from page 253)

fluid, when releasing the jacks, passed back through the same system of piping until it reached a 1½-in. dia. return pipe leading to the feed tank and connecting to the main pipe in front of the check valve. The return pipe had a release valve, which, of course, was closed during feeding operations.

The tank was covered and the feed pipe to the pumps was screened at its mouth to keep out dirt. The central valve, release valves, cross-over valves, cut-off valves and air throttle valves were all within reach of the central valve operator. There was one central valve operator on each cantilever, who controlled the feed system and release system of all four jacks. A tell-tale, situated in front of the operator, which magnified twice the difference in elevation of the two upper jacking girders, had to be continuously observed, and guided the operator in his efforts to keep the two corners at one end of the span level. These tell-tales also showed by marks the positions for introducing the upper and lower jacking pins. There were four electric lamps in front of the central operator which went out when the safety follower-up screws reached a position where, if the lower pins were in and bearing, the upper girder would rest on the safety jacks.

At each corner of the span a valve operator was located whose duty was to keep the upper jacking girder level during raising or lowering. A tell-tale, which magnified twice, was situated in front of this operator, which showed any difference in elevation of the two corners of the girder.

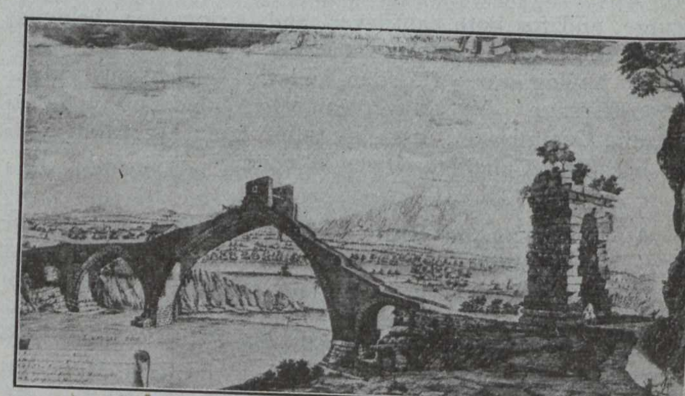
The dead load required a fluid pressure of about 3,800 lbs. per sq. inch. The maximum calculated load was 5,250 lbs. per sq. inch, including impact and 5-lb. wind. The system had been tested to 6,000 lbs. per sq. inch. On the air-feed from the pumps was placed a safety valve set to 5,000 lbs. per sq. inch.

The whole lifting apparatus had been designed (see Fig. 5) to take care of the dead load, plus impact of 20 per cent. of the lifted weight, the effect of a cross-wind of 5 lbs. per sq. ft. and a transferred load at opposite corners of the span of 500,000 lbs., due to a deflection by two opposite corners to an amount of 3¼ ins. below the other two corners.

TWO THOUSAND YEARS' PROGRESS IN BRIDGE BUILDING

THE new Quebec Bridge represents the highest development, to date, of the bridge builders' art. It is a far cry indeed, from the methods of two thousand years ago, as evidenced by the old print reproduced here of what is thought to be the oldest bridge in existence, viz., the Devil's bridge, over the Llobregat River at Matorell, in the province of Barcelona, Spain.

According to the well-known American bridge engineer, Gustav Lindenthal, this bridge was built about 219 B.C., during the time of Hannibal, and some accounts



It's a Far Cry From This to the Quebec Bridge.

ascribe the building of the structure to Hannibal. It is built of hewn stone, the largest opening having a span of 121 ft.

The bridge is of pointed arch type and shows a correct understanding of load distribution, indicated by the massive high walls on top of the apex. It is not suited for wagon traffic, but only for pack mules and pedestrians.

The Cleveland Engineering Society, to whom we are indebted for the accompanying print, states that the bridge was thoroughly repaired in 1766 under Charles III. of Spain. To the right of the bridge there is shown a triumphal arch built in commemoration of Hannibal's father, Amilcaris.

27 PER CENT. NICKEL STEEL IN QUEBEC BRIDGE

The Quebec Bridge was mostly fabricated from carbon steel shapes, but about 27 per cent. by weight is nickel steel. Among the nickel steel members are the trusses of the suspended span; a large portion of the cantilever trusses, such as the bottom chords and those web members in which it was deemed advantageous to use nickel steel without the sections becoming too small in consequence; the eye-bars of the suspended span, and the bottom laterals of the cantilever arms. . . . . .

The Quebec Bridge in Tabloid

LENGTHS

Length of suspended span .... 640 ft.
Length of cantilever arm from centre of main pier to end of cantilever .... 580 ft.
Centre to centre of main piers .... 1,800 ft.
Centre to centre of anchor piers............. 2,830 ft.
Centre of main pier to centre of anchor pier... 515 ft.
First north approach span .... 110 ft. 7 9/16 in.
Second north approach span.......... 157 ft. 10½ in.
South approach span .... 140 ft. 4 3/16 in.
Abutment to abutment, face to face... 3,238 ft. 10¼ in.
Sub-chord (compression member) .... 84 ft.
Sub-chord (balancing member for symmetry).. 84 ft.
From anchor pier to main pier, panels of anchor arm measure 23 ft., 42 ft., 28 ft., 31 ft., 32½ ft., 34½ ft., 30 ft. and 7 of 42 ft. each.
From main pier to end of cantilever, panels of cantilever arm are 8 of 42 ft. each, 40 ft., 32 ft., 31 ft., 28 ft., 25½ ft., 22½ ft., 42 ft., 23 ft.
From cantilever arm to centre of suspended span there are 4 panels of 32½ ft. each, 2 of 35 ft. each, 3 of 40 ft. each.
Length 150 ft. clear of high water level...... 686 ft.

WIDTHS

Centre to centre of trusses .... 88 ft.
Two-track system, 15 ft. each.
Space between track systems .... 17 ft. 6 in.
Centre to centre of track................. 32 ft. 6 in.
Cantilevered sidewalk, from centre of track girders to centre of railing.......... 5 ft. 9¾ in.

HEIGHTS

North anchor pier, from el. 77 to el. 239.
North anchor pier concrete to el. 85.
North anchor pier granite facing above el. 85.
South anchor pier from els. 81 and 99 (variable) to el. 239.
South anchor pier concrete to el. 110.
South anchor pier granite facing above el. 110.
South main pier el. 128 to el. 0.94.
South main pier concrete to el. 75.
North main piers el. 128 to el. 20.
North main pier concrete to el. 77.
North main pier granite facing above el. 77.
North main pier solid granite for top ten courses.
Depth of suspended span from centre of top chord to centre of bottom chord.......... 110 ft.
Depth of suspended span at hip............. 70 ft.
Depth of cantilever arms at outer ends........ 70 ft.
Main posts, centre of bottom pins to centre of top pins .... 310 ft.
Depth of anchor arms at anchor piers......... 70 ft.
Bridge floor is at a level 40 ft. below main panel points.
Suspended span clear of high water .... 150 ft.
Extreme high water level 101.3
Extreme low water level 79.7
Ordinary high water level 94.7
Ordinary low water level 82.0

QUANTITIES AND WEIGHTS

Timber in south main pier .... 3,845 cu. yds.
Concrete in south main pier .... 24,050 " "
Masonry in south main pier .... 10,385 " "
Timber in north main pier .... 5,450 " "
Concrete in north main pier .... 16,480 " "
Masonry in north main pier .... 9,940 " "
Concrete in south anchor pier .... 2,135 " "
Masonry in south anchor pier .... 14,279 " "
Concrete in north anchor pier .... 1,123 " "
Masonry in north anchor pier .... 17,954 " "
Steel in main anchorage each anchor pier. 732,000 lbs.
Steel in wind anchorage each anchor pier. 58,000 lbs.
Four main shoes weigh about 450 tons each.
Four main posts, 10′ square, weigh about 1,200 tons each.
Pins at shoe connection to bottom chord and main post are 45″ diam. Weight, 12 tons each, including sleeve.
Weight of each anchor arm inclusive of main posts .... 17,000 tons
Weight of each cantilever arm exclusive of main posts .... 12,000 tons
Weight of suspended span, erected complete. 6,000 tons
Weight of approach spans .... 1,500 tons
Anchorages .... 790 tons
Total structural steel received at bridge site for the structure, exclusive of rails and reinforcing steel in sidewalk, 133,310,570 lbs., or 66,655 tons.

MISCELLANEOUS INFORMATION

Bridge floor is on 1 per cent. grade almost to the suspended span.
The suspended span and the adjacent end panels of the cantilever arms are level.
Rivets up to 1¼ in. diameter were used, whereas the biggest previously used were 1 1/16 in. Later, 1½-in. rivets were used for Hell Gate Bridge.
Batter of main piers, 1 in. in 1 ft.
Batter of anchor piers, ½ in. in 1 ft.
North anchor pier at bottom.. 137 ft. 10 in. x 31 ft. 4 in.
North anchor pier at top..... 130 ft. 6 in. x 24 ft.
South anchor pier at bottom.. 137 ft. x 30 ft. 6 in.
South anchor pier at top..... 137 ft. 6 in. x 24 ft.
Main piers at top .... 160 ft. 8 in. x 32 ft. 4 in.
Concrete foundation for main piers .... 168 ft. 3 in. x 60 ft.
Caisson for north main pier.. 181 ft. x 55 ft. 6 in.
Main steel shoes .... 26 ft. 4 in. x 20 ft. 10 in.
Top of concrete shoe bearings 27 ft. 6 in. x 22 ft.

Cost estimated at approximately $17,000,000.

Estimated total cost of first Quebec Bridge and new bridge, including interest on all moneys expended, estimated at about $35,000,000. This figure has been variously estimated by different authorities. W. F. Tye places it as high as $40,000,000.

Contractors for superstructure, the St. Lawrence Bridge Co., Limited.

Contractors for substructure, M. P. & J. T. Davis, Quebec.

Contract for superstructure let 1911.

Work on substructure started December, 1911.

Editorial

THE QUEBEC BRIDGE

The eyes of the engineering world have turned toward Canada this week. Practically the final, and certainly the most difficult, stage of the erection of the Quebec Bridge is being successfully accomplished as we go to press.

This bridge, which will carry the main line of the National Transcontinental Railway across the St. Lawrence River near Quebec city, is without doubt the most remarkable steel structure ever built.

The K-truss design solved many of the most difficult problems confronting those who tendered on the project. On account of the comparatively short erection season, only about seven months at the bridge site, and also on account of the scarcity of expert ship builders and similar classes of erectors in Canada, it was necessary to design the Quebec Bridge in such manner that all members could be built in the shop and erected in the field in large units. On the contrary, in the construction of the Forth Bridge, which has so often been compared with the Quebec Bridge, the plates were transported to the bridge site and riveted there, so that the weights handled were quite trifling. Only small cranes were required for erection, whereas in the Quebec Bridge, the huge size of some of the members required the use of travellers of unprecedented capacity.

The prescribed wind pressure was the determining factor in the design of the Forth Bridge. Only a few years before its erection the Tay Bridge had blown down, carrying a train with it and drowning the passengers. The Tay Bridge had been very imperfectly designed, but as a result of the accident the Board of Trade required that a 56-lb. wind pressure be specified for the Forth Bridge. As a result, all of the compression sections are circular. The trusses are not in vertical planes and they narrow toward the outer ends of the cantilever arms, so that the suspended spans are comparatively narrow.

In the design of the Quebec Bridge, the heavy live load and the necessity of ease of erection were the governing factors rather than the wind pressure. The specifications for the Quebec Bridge called for a design adequate to withstand 30 lbs. wind stresses per square foot of trusses, and one and a half times that pressure per square foot of projected area of floor and one train. The prescribed load was two Cooper's E-60 engines followed by 10,000 lbs. per lineal foot. To meet all conditions and to simplify erection it was necessary that the trusses should be in vertical planes, as it would have been extremely difficult to design a traveller that could place heavy members in decidedly inclined planes.

Where a cantilever is required of such great height and length as the Quebec Bridge, it would be very difficult for a traveller to reach the tremendous distance that would be required by a doubly subdivided triangular truss, while with a subdivided triangular truss the panels would be extraordinarily long or the diagonals too steep for economy of section.

Mr. Johnson's K-truss design solved the problem, as in the K-system the diagonals are only half the height of the post, and they are therefore all at practically the most efficient angle of 45 degs. Moreover, the K-system made it possible to arrange a favorable angle for the diagonals without the use of a great number of subsidiary members. The latticed trusses supporting the eye-bars of the top chords are the only subsidiary members in the Quebec Bridge. The reach required of the traveller was only a half panel.

Another advantage of the K-system was the splendid uniformity of erection which it afforded, and which would have been impossible with steep diagonals or lengthy panels. The erection of the cantilever arms, and to a great extent of the anchor arms also, was merely a repetition from panel to panel and practically no false members were needed for erecting purposes. Moreover, the K-system unites all the advantages of a double system with those of a statically determinate one. Each diagonal and each suspender of the K-system carries about half the web shear, while in the triangular truss, for instance, each diagonal carries all the web shear, making the members very much larger.

The appearance of the K-truss is also a noteworthy point, being decidedly superior to all other designs. The diagonals all having the same slope present a very simple and regular appearance which is extremely pleasing.

In carrying out the design of the K-system for the Quebec Bridge, pure science was taken into consideration to a greater extent than has ever before been done. The investigation into and allowance for temperature changes and sun stresses is a case in point. To refer to the Forth Bridge again: it is a matter of common knowledge that the sliding plates of the bottom chord no longer slide. In the Forth Bridge the largest section is only approximately 800 square inches, whereas on the Quebec Bridge the largest member is over 2,000 square inches. The range of temperature at Quebec is twice as great as at the Firth of Forth. The many great circular piers of the Forth Bridge are 49 ft. in diameter and built very deep to withstand such forces, but the Quebec substructure would have been enormously expensive if built upon the Forth plan.

For each main post in the Quebec Bridge there are two in the Forth, the main posts having been built as towers rather than as columns, and the anchor and cantilever arms having been built from the main towers with practically no falsework. At Quebec this would have been impossible for a number of reasons. Ice jams were feared if too many large piers were built in the river; and with the range of temperature at Quebec, the stresses due to expansion and contraction between the various parts of the main piers would have been tremendous and it could not have been assumed that the piers would adjust themselves to the motion, as they are built in closely packed sand and gravel.

The ingenious and daring method of erection of the bridge also marks it as an exceptional structure. It would have been impossible to have cantilevered the suspended span without making the cantilever panels bigger and heavier. Many of the members would have undergone maximum stress during erection, and the weight of the bridge would have been excessive for the live load.

Many simple spans have been lifted into place where they could be handled from barges with ordinary derrick cars, but the Quebec Bridge suspended span is the first span of a cantilever bridge which has ever been lifted into place. It is the first span ever hoisted by hydraulic jacks, and is by long odds the largest span of any kind which has ever been hoisted.

A number of simple spans on falsework have been floated at high tide and lowered into position on their piers with the fall of the tide, but the Quebec Bridge span is the first span of a cantilever bridge that has ever been floated on scows, and it is considerably larger than any other span of any kind which has ever previously been floated. The next largest span ever floated, so far as we are aware, was the Hawkesbury Bridge in New South Wales, a 420-ft. simple span which was floated into place on high scaffolding at flood tide.

It is problematical whether the success of the Quebec Bridge will lead to any extensive use of the K-system for smaller bridges. The bridge engineer of the Sante Fe Railway recently adapted the system to a simple span between 300 and 400 feet long, and we understand that the design in this case did not prove to be economical. The economy of the K-system due to each diagonal taking only half the web shear may be lost if the span be such a light one that it is better to concentrate the shear in one diagonal than to try to divide it between two members. But for cantilever bridges with long spans, the Quebec Bridge has proven the complete superiority of the K-system.

Many scientific points of design which have hitherto been totally ignored or very indefinitely determined has to be most carefully calculated for the Quebec Bridge on account of the extraordinary proportions of the structure. The friction breaks, for instance, between the cantilever arms and the suspended span are very unusual. It was calculated that a friction of 250,000 lbs. would be required at each corner of the suspended span to prevent its being swung back by a locomotive or by longitudinal wind force. Provision had to be made to get this friction and yet permit the span to expand or contract before the stresses due to temperature changes could become too large. As a result of much study, friction plates have been inserted which can be adjusted initially, and from time to time, to give any desired friction, a hydraulic arrangement has been devised for pulling out a test plate at any time and measuring the amount of friction which must be overcome in doing so, and by a screw adjustment the friction can be increased or decreased as may be found necessary and the other plates similarly adjusted. There are about a dozen of these friction plates at each end of the bridge, fitting into each other much as would the fingers of levelly outstretched interlocking human hands. The rails or track are in no way depended upon to keep the suspended span from moving.

Many other similar points which have been previously largely ignored in bridge design, have been most carefully calculated. As another example, there is a very unusual contrivance at the anchor pier, where the big lengths and sections involved make the motion at the pier a very complicated one.

Cross-winds bend the anchor span in a horizontal plane, while live loads bend it in a vertical plane, and also the end struts may rise or fall either levelly or unevenly with the expansion and contraction of the anchor chains, which may or may not be uniform, and at the same time the motion tending to distortion, due to train on one track, must be considered. This means that motion of practically every describable description must be provided for at one point. Therefore, there has been imbedded in the anchor piers a deep steel frame, carrying a vertical pin, and at the end of the pin there is a spherical thimble that allows vertical motion as the thimble slides up and down on the pin, and longitudinal motion as both pin and thimble slide between vertical plates, and any other kind of motion on account of its spherical shape. This new type of joint is situated at the centre of the bottom end strut of each anchor arm, and may be said to be a combination of universal hinge and sliding joint. This complicated motion is partly due to the unusual length of the span and the great length of the anchor chains, and partly to the heavy trains for which allowance has been made.

Temperature stresses were without doubt never before so carefully calculated. A difference of 25 degrees in temperature was assumed between the parts exposed to the sun and the shaded parts. Between the piers and the bridge proper a difference of 50 degrees temperature was considered. Secondary stresses of all sorts were considered and allowed for in an unprecedented manner. Needless to state, the weight of the paint and every other known feature of dead weight, however slight, was taken into consideration.

Only stresses due to the absence of pins at joints or due to friction at pins, were considered as secondary stresses, the wind and all other stresses being considered primary. For compression members, primary stresses were not permitted to exceed 14,000 lbs. per sq. in., or 18,000 lbs. per sq. in. inclusive of all secondary stresses. For eye-bars, tension, 20,000 lbs. per sq. in. was allowed for primary stresses, and 24,000 lbs. per sq. in. inclusive of all secondary stresses. For riveted tension members, 18,000 lbs. per sq. in. was allowed for primary stresses and 24,000 lbs. per sq. in. inclusive of all secondary stresses. In this regard one must remember that the K-system eliminates most of the secondary stresses, and the greatest secondary stress can never co-exist with the greatest primary stress.

Probably no other bridge has ever been erected so carefully as has the Quebec Bridge. The sections were put into place one at a time, every main member being riveted as the work progressed. The plans for the erection of the centre span received the best care and thought from many of the most experienced bridge engineers in Canada and the United States. When one looks at the tremendous centre span and sees the great height to which it must be lifted, one is inclined to say that it will be a miracle if the bridge is ever successfully completed. Even a comparatively brief study of the plans, however, serves to show that every minutest detail has been so carefully calculated that one readjusts his opinion and decides that it would be a miracle if the suspended span were not to be readily hoisted into place exactly as planned.

The accident of last year was very unfortunate, yet it is hardly to be expected that such an enormous undertaking could be carried out without more or less serious accident, and the accident that did happen chanced to be of the more serious type. The same care has been taken in regard to the lifting appliances as was shown in the design of the bridge proper. For instance, due allowance was made for the difference in length between the various lifting chains due to the fact that certain chains might be in the sun and others in the shade.

The suspended span has been designed to permit of a torsion of about 3¼ inches in diagonally opposite corners. For example, if the southwest and northeast corners of the span were to be held steady while the southeast and northwest corners were both to be dropped or both to be raised simultaneously to the extent of about 3¼ inches, the allowable unit stress would not be exceeded. Even if greater distortion were to take place and some laterals were to be strained to the elastic limit, resulting in permanent distortion, even then a collapse of the span would not necessarily follow, and a few twisted laterals could be replaced even after the suspended span was raised to its final level.

If the laterals and sway bracing were to be strained to a point approaching the ultimate limit, however, of course the sway bracing would go, and the top and bottom laterals; that is, the span would collapse. It is for this reason that such extreme care has been taken throughout in the lifting, and many devices have been adopted to ensure that the span will be raised with all four corners in a horizontal plane as nearly as possible at all times.

At the time of last year's collapse, one end of the span had been raised six lifts when the other end had only been raised five. While it was not known that this contributed in any way to the accident, and in fact theoretically should not have done so, nevertheless the greatest care has been taken this year to ensure that even the two ends of the span be lifted at the same time and at the same speed, and of course even greater precautions are taken that the two corners of either end should not be lifted at different times or speeds.

The valve operators are all thoroughly reliable and trained men, nearly all being engineers who have been in structural work for many years. Every man knows the duties of two positions which are of a different nature, the one not involving such nervous strain as the other, and all the men are paired off in this manner so that they can relieve each other at any time, and it was made obligatory that the men in the more serious positions be relieved at least every two hours.

For example, the north central control operator and the north "end engineer" are interchangeable, both knowing each other's duties. Similarly, the assistant end engineer in charge of, say, the northwest corner, is interchangeable with the assistant valve operator in charge of the northwest corner, and those two men must know each other's duties and act as mutual relief regularly.

There are consulting engineers and bridge company officials on both cantilevers, carefully watching every stage of the operation, and also expert electricians, hydraulic engineers, skilled mechanics, etc., ready to deal with any emergency.

In other words, the lifting of the suspended span of the Quebec Bridge reminds one of the old puzzle about the irresistible force meeting the immovable object. It is a miracle of engineering if it succeeds and it is a miracle if it doesn't!

CANADIAN ENGINEERING HAS TRIUMPHED AT QUEBEC

(Continued from page 239.)

low position, with the pins changed and ready for another lift whenever desired.)

Fourteenth lift, 8.01 to 8.18, 17 mins.

The second link of each of the lifting hangers was removed after the fourteenth lift, taking about 40 mins.

Fifteenth lift, 9.06 to 9.25, 19 mins.
Sixteenth lift, 9.25 to 9.37, 12 mins.
Seventeenth lift, 9.39 to 9.57, 18 mins.
Eighteenth lift, 9.57 to 10.09, 12 mins.
Nineteenth lift, 10.09 to 10.21, 12 mins.
Twentieth lift, 10.22 to 10.35, 13 mins.
Twenty-first lift, 10.36 to 10.50, 14 mins.
Twenty-second lift, 10.51 to 11.03, 12 mins.
Twenty-third lift, 11.03 to 11.17, 14 mins.
Twenty-fourth lift, 11.19 to 11.31, 12 mins.
Twenty-fifth lift, 11.34 to 11.46, 12 mins.
Twenty-sixth lift, 11.48 to 12.01, 13 mins.

After the twenty-sixth lift the work was stopped to permit the men to relax and to have lunch. After lunch another set of links were removed, and the breaking of a minor part of the hoist for handling these links delayed the work for about 1½ hours while the part was being replaced.

Twenty-seventh lift, 3.36 to 4.00, 24 mins.
Twenty-eighth lift, 4.00 to 4.13, 13 mins.
Twenty-ninth lift, 4.13 to 4.27, 14 mins.
Thirtieth lift, 4.29 to 4.40, 11 mins.
Thirty-first lift, 4.40 to 4.52, 12 mins.
Thirty-second lift, 4.53 to 5.06, 13 mins.
Thirty-third lift, 5.07 to 5.19, 12 mins.
Thirty-fourth lift, 5.20 to 5.31, 11 mins.

The span is anchored for the night, with eighty-two feet still to go. The weather reports received here tonight indicate moderate southwest winds, fair weather. There is a tropical storm on the Atlantic coast but it is going east and may not affect Quebec. With continued good weather the span should be in place by Thursday night.

TIME OF FLOATING DEPENDED ON TIDE

The time of floating the suspended span of the Quebec Bridge depended—after everything was erected and arrangements completed—upon only two things, the tide and the weather. Both of these had to be satisfactory; the weather for many obvious reasons, the tide so that the scows would (a) be drained at low tide, and (b) rise high enough to navigate the proposed course at high tide.

The plan for floating the suspended span was based upon the following tide tables, a plus or minus variation of 2½ ft. at the bridge site being allowed for:—

High Water

Date.	— a.m. —		— p.m. —	
September.	Time.	Elevation.	Time.	Elevation.
Saturday, 15	6.13	95.2	18.37	95.1
Sunday, 16	6.54	95.6	19.15	95.7
Monday, 17	7.32	95.7	19.48	96.0
Tuesday, 18	8.06	95.5	20.18	96.2
Wednesday, 19 ..	8.38	95.2	20.47	96.3
Thursday, 20	9.09	94.7	21.15	96.2
Friday, 21	9.39	93.9	21.45	96.0
Saturday, 22	10.12	93.1	22.21	95.5
Sunday, 23	10.57	92.2	23.11	94.7

Low Water

Date.	— a.m. —		— p.m. —	
September.	Time.	Elevation.	Time.	Elevation.
Saturday, 15	0.46	80.8	13.15	80.4
Sunday, 16	1.31	80.7	13.33	80.7
Monday, 17	2.12	80.8	14.28	81.0
Tuesday, 18	2.50	81.0	15.00	81.4
Wednesday, 19 ..	3.26	81.3	15.29	81.6
Thursday, 20	4.06	81.5	15.56	81.6
Friday, 21	4.33	81.8	16.22	81.7
Saturday, 22	5.08	82.0	16.51	81.8
Sunday, 23	5.53	82.4	17.30	82.1

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NORTHERN ELECTRIC COMPANY
CALGARY, ALBERTA

The Roof for All First-class Structures—

THE roof is apt to be one of the very last things to be considered in the construction of a building. It really ought to be among the first.

Owners wake up to this fact when they begin to pay bills for repairs and damage caused by leaks.

Permanent buildings deserve permanent roofs.

The most economical and altogether satisfactory permanent roof is a Barrett Specification Roof, with a unit cost (the cost per square foot per year of service) lower than that of any other permanent roofing.

It is for this reason that most of the great manufacturing plants and textile mills of the Dominion, the great railroad terminals and skyscrapers, and hundreds of less pretentious buildings, carry roofs of this type. Some of these are from twenty to thirty years old and are still in serviceable condition.

A Barrett Specification Roof is not a ready-made roofing. It is built layer upon layer on the building, following time-tested methods which are recognized as standard by technical men generally.

A copy of The Barrett 20-Year Specification, with roofing diagrams, mailed free on request to any one interested.

Our 20-Year Guaranty Bond

We are now prepared to give a 20-Year Surety Bond Guaranty on every Barrett Specification Roof of fifty squares and over in all towns of 25,000 population and more, and *in smaller places where our Inspection Service is available.*

This Surety Bond will be issued by the United States Fidelity and Guaranty Company and will be furnished by us *without charge.*

Our only requirements are that the roofing contractor shall be approved by us, and that the Barrett Specification, dated May 1, 1916, shall be strictly followed.

If you wish any further information regarding this Guaranty, write to our nearest office and the matter will be given prompt attention.

THE PATERSON MANUFACTURING COMPANY, LIMITED
MONTREAL TORONTO WINNIPEG VANCOUVER

THE CARRITTE-PATERSON MANUFACTURING CO. LIMITED
ST. JOHN, N.B. HALIFAX, N.S. SYDNEY, N.S.

Coast to Coast

Bridgeburg, Ont.—The Toronto, Hamilton and Buffalo Railway have started laying out for yard and terminal purposes the 50 acres acquired recently by the company here. The work being done is probably of only a preparatory character, as plans for laying out the area had not been settled.

Brigden, Ont.—Hydro transmission lines are now within a mile of the town, and alterations are being made in the local wiring system to accommodate the new system, which will be ready shortly.

Dartmouth, N.S.—It is likely that a shipbuilding plant will be established here in the near future. At a recent meeting of the Town Council and the Board of Trade an appropriation of $200,000 was voted to any responsible firm who would undertake the building of such a plant.

Kenora, Ont.—The Kenora and English Bay Railway has been incorporated and will build a railway from the National Transcontinental Railway, in Kenora District, west of Superior Junction, northerly and westerly, crossing the English River west of Lac Seul, thence northerly and westerly in the District of Patricia, thence westerly and southerly to and in Manitoba to Winnipeg. The head office of the company is to be in Toronto; the authorized capital is $1,000,000, and the company may issue securities for $50,000 a mile. The provisional directors are: W. Miller, A. LeR. Williams, E. Miller, C. Flatt and A. A. Macdonald, Toronto.

Kitchener, Ont.—Work on the sewage farm has been delayed somewhat owing to scarcity of labor, but it will probably be completed within a few weeks.

Lachine, P.Q.—The Dominion Bridge Co. are equipping a portion of their plant for the manufacture of marine engines and boilers.

Locust Hill, Ont.—Work has recently been commenced on a new concrete bridge over the Rouge River, for the York Highway Board. The work of cutting down the grade on the approaches was also started, and it is expected that the bridge will be ready for traffic by December 1st.

Orillia, Ont.—The government bridge over the North River, on the 6th Concession of North Orillia, about a mile east of Uhtpoff C.P.R. station, has been completed. It is a reinforced concrete trestle structure, and stands on 16 concrete piles, driven to the rock.

Ottawa, Ont.—An order-in-Council has been passed under the War Measures Act, prohibiting the exportation of steel rails from Canada abroad to countries other than the United Kingdom, British possessions and protectorates.

Ottawa, Ont.—$3,000,000 has been voted by the Dominion Government for the completion of the work on the Hudson Bay Railway.

Owen Sound, Ont.—A meeting of a good roads deputation from Guelph, Fergus and Elora met the local board of trade to discuss the proposed Hamilton-Guelph-Owen Sound highway. A convention will be held at Guelph on October 15, at which representatives from all the municipalities from Hamilton to Owen Sound inclusive are to be present.

Peace River, Alta.—Assurance that the bridge across the Peace River would be finished in time for next harvest and that the railway would by then be constructed up country, was given by J. D. McArthur, president of the E.D. and B.C. Railway during his recent visit to Peace River.

Princeton, B.C.—The Canada Copper Co. contemplate opening up a large mine near here. Power to operate the mine will be supplied by the West Kootenay Power and Light Co., while the copper and other products will be smelted and refined by the Consolidated Mining and Smelting Co., at Trail.

Sarnia, Ont.—The preliminary work of laying out the sites for the new $200,000 industry to be located in this city, has commenced. Mr. W. Brunt is superintending the work. The new plant, which is being fostered by Senator Lyman Holmes, president of the Romeo Foundry Co., Port Huron, will manufacture automobile parts and castings. It is expected that excavation work will commence at an early date, when the details and negotiations will be finally arranged. The Grand Trunk Railway Co. will lay a switch through the centre of the property, according to present plans. Four buildings, 90 feet by 200 feet, will be erected and 150 men will be employed at the outset.

Sault Ste. Marie, Ont.—The municipal councils of the city of Sault Ste. Marie, Ont., and the town of Steelton have approved an agreement providing for the amalgamation of these two municipalities. A by-law will be submitted for the approval of the ratepayers.

Tillsonburg, Ont.—Work has been started on the large extension to the Maple Leaf Harvest Tool Company's plant on Tillson Ave. When the extensions are completed, the local plant will be the largest of its kind in Canada. The main building will have a frontage of 330 feet on Tillson Ave. It will include the office and paint shop. The finishing room and storage room are also included. To the rear will be a factory storeroom and a finished products storeroom.

Toronto, Ont.—Referring to the extension of the civic car line on Bloor St., Works Commissioner R. C. Harris stated that the materials have been ordered and delivery is expected within thirty days. He also said that there is a possibility of the work being completed and the line placed in service before Christmas.

Vancouver, B.C.—British Columbia's shipbuilding programme provides for the construction of some 117,000 gross tons of commercial shipping, to have a total carrying capacity of nearly 185,000 tons. The value of these ships, some 50 in all, is in the neighborhood of $25,000,000. While this estimate of construction is only approximate, it includes practically every ship of importance in freight carrying. The amount of tonnage given represents definite contracts only.

Vancouver, B.C.—J. Coughlan and Sons are constructing 16 steamers of 88,000 dead weight tons capacity.

Vancouver, B.C.—Prospects before the coal mining industry of Vancouver Island and, indeed, of the province, are of the brightest according to information received by the Provincial Mines Department. At the present time production on the island is greater than at any previous time and with the prospect that, sooner or later, the importations of fuel oil into British Columbia will be materially curtailed, if not altogether prohibited by the United States government, because of the enormous increase in the demand for fuel oil for war purposes, the big coal producing companies are looking for a great increase in output and a decided impetus to the industry in British Columbia.

Windsor, Ont.—An extensive network of good roads incorporated in the long-deferred report of the suburban area commissioners, was filed with the city clerk of Windsor. The report allows for several terminals of the new provincial highway from Montreal. It will give branches running into Ford City, Walkerville, Windsor, Sandwich and Ojibway. There will be three distinct parts of the county system. One will be the suburban area, another will be three roads on the frontage basis—Walker Rd. to Oldcastle, front road from Island View to Sans Sauci and front road from Turkey Creek to Amhertsburg—and the third is the county road system as defined in the by-law approved of a year ago last July. The suburban commissioners include the road commencing at the town of Sandwich at the intersection of Tecumseh or Tunnel Rd. with the Huron line and running along Huron line to the fourth concession in Sandwich west to the northerly limit of the town line between Sandwich West and Anderdon, at Lukerville.

Windsor, Ont.—Work has started on the new C.P.R. bridge across London St.

Winnipeg, Man.—An effort will be made to force the Winnipeg Electric Railway to contribute towards the cost of erecting the Provencher Ave. bridge. The St. Boniface Council will also ask the Provincial Government to contribute to the bridge funds.

Winnipeg, Man.—Owing to the rush to the harvest fields a labor shortage of considerable seriousness has arisen during the last fortnight on the big aqueduct of the Greater Winnipeg Water District, it was reported at the meeting of the administration board. The contractors are thus faced with a serious difficulty during the most vital period of their season's work, when the task of completing work laid down and general clearing-up for the winter, is most important. In this, relief cannot be expected until after the harvest is reaped, and then, perhaps, at a considerable advance in wages, in order to induce men to return before going into the cities for the winter as is their custom.

Construction News Section

Readers will confer a great favor by sending in news items from time to time. We are particularly eager to get notes regarding engineering work in hand or projected, contracts awarded, changes in staffs, etc.

▲—Denotes an item regarding work advertised in *The Canadian Engineer.*
+—Denotes contract awarded. The names of successful contractors are printed in CAPITALS

ADDITIONAL TENDERS PENDING

Not Including Those Reported in This Issue.

Further information may be had from the issues of *The Canadian Engineer* to which reference is made.

PLACE OF WORK	TENDERS CLOSE	ISSUE OF	PAGE
Maisonneuve, Que., erection of factory	Oct. 15.	Sept. 13.	50
Myruam, Alta., erection of school	Sept. 25.	Sept. 13.	50
Ottawa, Ont., roofing and sheet metal work	Sept. 20.	Sept. 13.	56
Toronto, Ont., stop valves, valve operating pumps, special castings	Oct. 2.	Sept. 6.	56

BRIDGES, ROADS AND STREETS

Bruce Tp., Ont.—Township Council contemplates erection of bridges to cost $15,000, to replace structures damaged in floods. Clerk, J. G. MacKay, Underwood, Ont.

Calgary, Alta.—City Council plans construction of a temporary bridge across the north channel of the Bow River at Fourth St. W. City engineer, Geo. W. Craig.

+—**Grand Falls, N.B.**—Provincial Government department of Public Works, Fredericton, N.B., has awarded the general contract for embankment and concrete arch to F. L. BOONE, Devon, N.B.

+—**Kincardine, Ont.**—THE HUNTER BRIDGE & BOILER CO., Queen St., has been awarded the contract for a class "A" 50 ft. steel arch on Russell St.

Montreal, Que.—According to estimates submitted to the Board of Control by W. Chace Thomson, consulting engineer, 618 New Birks Building, it will cost $7,500 to repair the Cartierville bridge over the Back River and to put it in condition for traffic for a period of two years, for vehicles not exceeding four tons in weight. For eight-ton traffic it would cost $26,000. A new modern structure, with accommodation for vehicular, street car and foot traffic, would cost $200,000.

Oakbank, Man.—Tenders will be received by C. Christopherson, secretary-treasurer, at his office at Oakbank, Man., up to noon, September 22nd, 1917, for building and supplying all material for a 60 ft. pile bridge; also a 52 ft. pile bridge.

Ottawa, Ont.—Tenders for the metallic fittings for the Dominion Public Buildings at Ottawa will be received up to Wednesday, September 26. R. C. Desrochers, Secretary, Department of Public Works, Ottawa.

Preston, Ont.—The Board of Works of the Town Council has a large gang at work on street improvements on side streets. Drains are being built along the sides of the roads and the grader is at work levelling the streets.

Sarnia, Ont.—The Board of Works presented a report showing that the repairs and resurfacing of pavements would cost $19,800.

+—**Shawinigan Falls, Que.**—NAPOLEON LAPOINTE has the general contract for laying of sidewalks on Rue du Coton for the Town Council, and will require 400 bags of cement. Town engineer, Raould Reinfret.

+—**Shawinigan Falls, Que.**—The Town Council has let the contract for grading work for proposed concrete highway to J. D. JACOB. Concrete work to be done next spring. Mr. Jacob also has contract for grading for a macadam road.

+—**Stonewall, Man.**—DAVID WOOD, of Teulon, Man., has been awarded the contract for a 25-foot span reinforced concrete bridge over the Netley Creek, Rockwood Municipality.

+—**Sydenham Tp., Ont.**—MR. CORBETT, care of the township clerk, J. M. Thompson, Bognor, Ont., has been awarded the general contract for erection of a steel bridge for the Township Council.

+—**Tansley, Ont.**—NORMAN McLEOD, LTD., Kent Building, Toronto, has the general contract for concrete and steel bridge, costing $60,000, for the Halton County Council.

+—**Three Rivers, Que.**—City Council has let general contract for crib work and filling from aqueduct to highway bridge abutment to CHARLES PAGE, at $12,000. Spikes and bolts will be required by general contractor.

Three Rivers, Que.—In last week's issue, on page 46, it stated that the distance between Grand Mere and Three Rivers was three miles. It should have read thirty miles.

Toronto, Ont.—On the recommendation of Works Commissioner R. C. Harris, the Civic Works Committee decided to abandon the scheme for the extension of Wilton Ave., through to Coxwell Ave., so as to form a new cross-town thoroughfare north of Queen St. The work was estimated to cost $622,000. The difficulty of financing the project at the present time is the chief cause for its abandonment. It may be revived again after the war.

Toronto, Ont.—Property-owners in the vicinity of Shaw, Crawford, Montrose and College Streets are urging the City Council to complete Crawford Street extension and fill in Sully Crescent.

Walkerville, Ont.—Tenders will be received up to noon of Friday, September 21, for the construction of concrete pavements. Plans and specifications may be seen on application to Owen McKay, C.E., Walkerville, Ont.

Winnipeg, Man.—Arthur St. is to be repaved with asphalt.

Winnipeg, Man.—C.N.R. will have a strip of asphalt, 500 feet by 20 feet, laid on Water St.

WATER, SEWAGE AND REFUSE

Aylmer, Ont.—Town Council wants prices at once on glazed tile, 18, 20 and 22 in. Clerk, Jos. Senior.

Charleswood, Man.—Tenders will be received by the undersigned up to Saturday, September 22, 1917, for the drilling of one 5-inch well, and equipping same with pump and platform, on the Arboro St., West Winnipeg. A. B. Blakely, secretary-treasurer.

Edmonton, Alta.—Acting City Engineer A. W. Haddow reported that the cost of laying a 6-inch sanitary sewer on 63½ lane, to connect with the existing sewer on 61½ lane, would be $1,750.

Guelph, Ont.—The Sewerage and Public Works Commission recommended the construction of a sewer on Allan Ave. from the Edinburgh Rd. to Meadowview Ave. City Engineer, F. McArthur.

Halifax, N.S.—The sewer on Well St. may be extended about 200 feet. Assistant city engineer, H. Johnston.

Hamilton, Ont.—Sealed tenders will be received by S. H. Kent, City Clerk, addressed to Chas. G. Booker, Esq., Mayor, Chairman Board of Control, City Hall, Hamilton, up to 10 o'clock a.m., Tuesday, Sept. 25th, 1917, for the construction of the following pipe sewers:—Roxborough Avenue from Robins Avenue, to H.W.W. Pipe Line; Gibson Avenue, from Princess