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Contractors' Camp for Workmen on the New Quebec Bridge

By L. G. Jost*

The camp of the St. Lawrence Bridge Co., Ltd., at Quebec Bridge, is situated near the north abutment of the bridge, on a natural plateau which slopes gradually toward the east. It is protected on the north by a wooded hill and on the east and south by a low ridge of rock. As the bridge site is about half a mile from the main road, three miles from a street-car line, and eight miles from Quebec, and as the housing facilities in the neighborhood were inadequate, it was necessary to build practically a complete village. This was commenced in August, 1912, on land leased from the National Transcontinental Ry.

The plan (Fig. 1) gives a general layout of the camp, including the water-supply and sewerage systems. Fig. 2 is a bird's-eye photograph of the camp taken from the traveler now in the course of erection in the rear of the north abutment. Figs. 3 and 4 are interior views of the dining and the cook house, respectively.

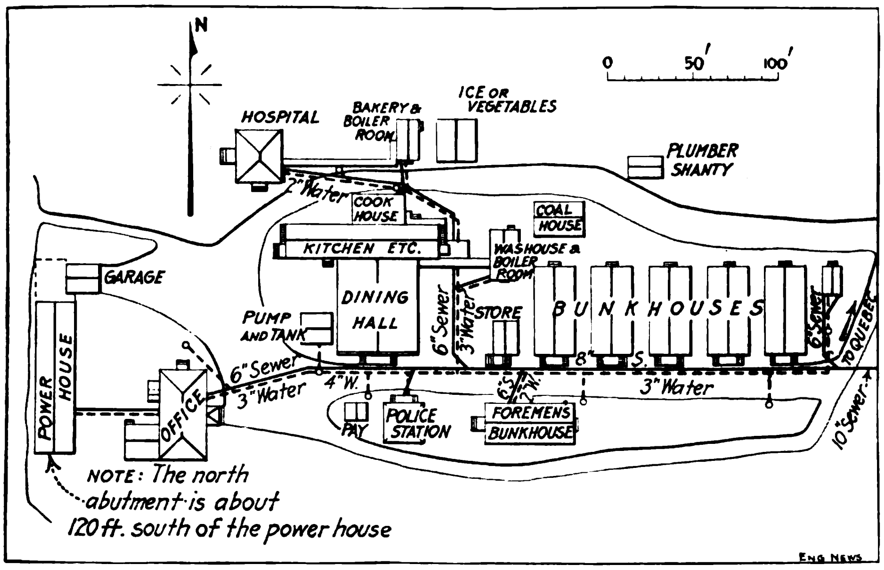


Fig. 1. Layout of Camp of the St. Lawrence Bridge Co., Ltd., at Quebec Bridge

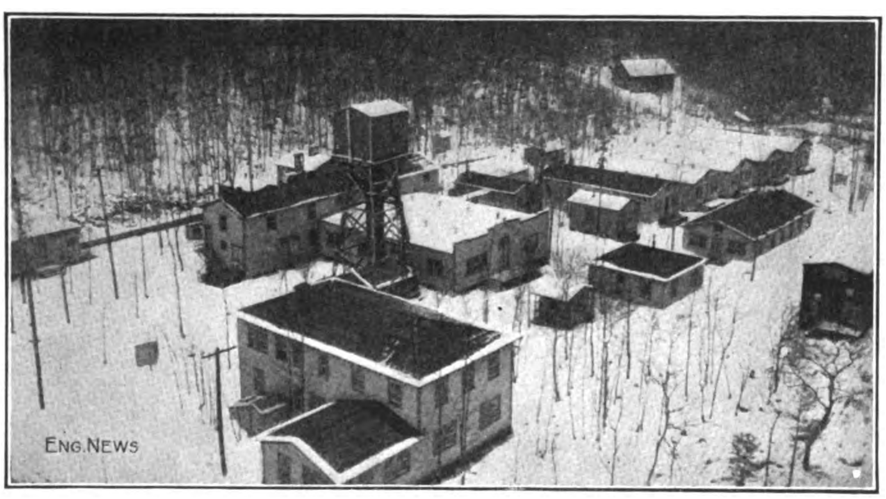


Fig. 2. General View of Camp from the Top of the Derrick Traveler at the Bridge

*Neilsonville, Que., Canada.

There are five bunk houses, each 24x53 ft. in plan and comprising eight 10x10-ft. bedrooms and a 12x24-ft. sitting room in front. The inside partitions are of single sheathing 8 ft. high; each room contains two two-tier bunks. A large stove in the living room with a pipe running the full length of the building provides heat in winter. The bunk house of the foreman is similar but has nine smaller sleeping rooms and a bathroom.

The common toilet is at the east end of the camp, and is 11x18 ft. in plan with sloping concrete floor which may be hosed out. It contains two range closets; and the automatic flush tanks are kept warm in winter by coils from a jacket heater. The 17x30-ft. wash-house building contains 18 porcelain wash basins and six shower baths. At the rear is the boiler room with a low-pressure boiler from which housed pipes are run for heating the dining hall and kitchen.

The dining hall is a bright room 47x58 ft. in plan; the roof has a slope of 1 on 12, and is carried on six 15-in. 42-lb. I-beams resting on 8x8-in. posts on sides and center. It has a seating capacity for about 150 men.

The kitchen building is two stories high, the upper being used as office and bedroom for the camp steward and quarters for the culinary staff. One end of the ground floor is a storehouse for dry groceries, while the other end is a cold storage room (heretofore cooled by ice, but now by an ammonia plant).

The cooking is done in an 18x30-ft. addition at the rear. The steam cooking utensils are operated by a high-pressure boiler in a house at the rear. This boiler also supplies heat for the hospital. The latter is a 24x28-ft. building and contains an office and bedroom for the resident physician, and one ward with bathroom, etc. It is intended only to administer first-aid treatment. All serious cases will be removed as soon as possible to the Quebec hospital.

The office building is a two-story structure 28x53 ft., with a one-story wing 20x20 ft. used as a drafting room. The interior of the building is sheathed. On the second floor there are seven bedrooms, a bathroom and a living room; in a small concrete cellar, there is a low-pressure steam boiler.

Other buildings not mentioned are a garage, a small pay shack, a general store, coal house, plumber's shanty, stable, police station, and bungalows for the superintendent and the engineer in charge of construction work.

Water is supplied to the camp by an artesian well. The water is pumped to a 10,000-gal. tank on a 40-ft. tower. Five 3-in. fire hydrants take care of the entire camp. At each hydrant is a small hose house containing two 50-ft.

Fig. 3. Dining Table of Office Staff

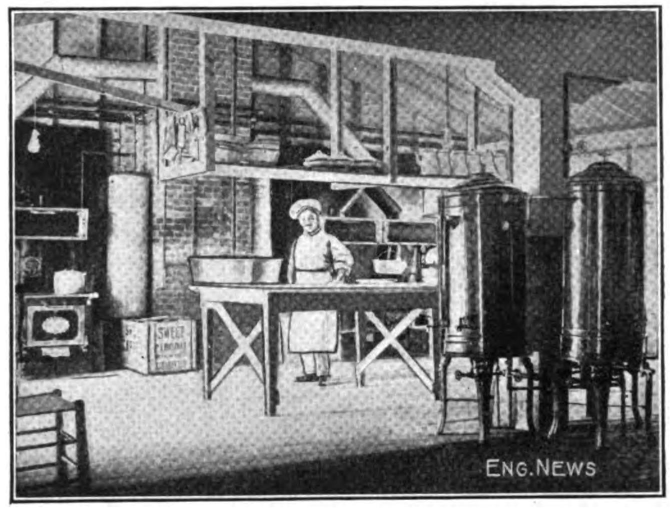


Fig. 4. Inside the Cookhouse

lengths of fire hose. The vitrified sewer mains range from 6 to 10 in. in diameter, and with the water line are buried about 2 ft. deep and follow the natural slope of the ground, which gives a gradient from of 3.5 to 0.5 ft. per 100. At the outlet the sewage runs into a 10x10-in. wooden chute and is carried over a bluff to a point below high-water level, some thousand feet downstream from the bridge.

The camp is lighted electrically by current transmitted from Quebec. Should this supply fail, the company's power house is able to carry the lighting load.

Electric Air Drills at the Kensico Dam

The performance of electric air drills used in quarrying operations at the Kensico Dam of the Catskill Aqueduct system, at Valhalla, N. Y., was observed and recorded by the Construction Service Co. In a paper before the annual meeting of the American Institute of Mining Engineers at New York City, on Feb. 16, W. L. Saunders, 11 Broadway, New York City, described the results. The drills observed are known as the Temple-Ingersoll Type of 5-F, with cylinder 5⅝x8 in. The pulsators were driven by 5-hp. 220-volt electric motors, and full speed gave 400 strokes per minute. The bits were changed every 25 min. The drills were operated by one drill runner and a helper. The diameter of the starting bits was 3½ to 4 in. and decreased ⅛ in. for each succeeding length of steel, down to 1¾ in. The steels were from 2 ft. 6 in. to 28 ft. 6 in. in length, octagonal in section, from 1¼ to 1½ in., with square cross bits. The oil consumption of the drills was about 3 qt. per shift; the power consumption was from 30 to 40 kw.-hr. per drill shift of 8 hours.

OBSERVED TIME REQUIRED FOR THE SEVERAL OPERATIONS

Operation	No. of observations	Avg. time min.	sec.	Consumed time as per cent. of total time
Drill cutting	16	14	18	51.1
Raising drill	15	1	5	3.6
Removing bit	12	0	32	1.4
Bailing hole	11	1	23	3.4
Putting bit in hole	12	0	22	1.0
Inserting bit in chuck	16	0	23	1.4

Other operations (e.g., tightening chuck, shifting drill, etc.) consumed 38.1% of the total time.

The average cutting speed was 0.135 ft. per min. Based on these tests the following costs per foot of hole and per cubic yard of rock which was loosened by blasting, were deduced:

Item and price	Unit cost per lin.ft.	per cu.yd.
6 drillers @ $2.50	$0.1275	$0.0123
6 helpers @ $1.75,—$25.50		
Blacksmithing, $13.13	0.0657	0.0064
Total labor drilling, $38.63	$0.1932	$0.0187
Coal, 500 lb., $0.87		
Oil, 3 qt. per drill, $1.35		
Power, 35 kw.-hr. each, @ 1c	$0.0216	$0.0021
Total drilling cost, $42.95	$0.2148	$0.0208
Interest and depreciation, 2% per month, $7.70	$0.0385	$0.0037
3 powdermen @ $2		
1035 lb. dynamite @ 12c		
25 detonators @ 3c —$130.95	$0.6547	$0.0632
Total cost, $181.60	$0.9080	$0.0877

One drill cut 31 ft. in 448 min., equivalent to 200 ft. by 6 drills in 8 hours. The low cost per cubic yard is due to the wide spacing of the holes (av. 17.5x16 ft.) which were heavily charged with dynamite.

The Kensico Dam has been described in a long article which was published in Engineering News of Apr. 25, 1912, p. 772.

The Oldest Isolated Power Plant in a New York City office building is stated by C. M. Ripley, of New York, in a recent paper before the American Society of Heating and Ventilating Engineers, to be that in the Mills Building, Broad St., New York City, dating back to 1883 or four years after the announcement of Edison's incandescent lamp. Here are the fifteenth and nineteenth Edison dynamos built, still running with their original steam engines. These units have run every day for 31 years and are still in use furnishing regular daily service.

Plants 29 years old were reported in the Wells Building and the Dakota and the Osborne Apartments. Plants 25 years old were reported in the buildings of the Bank of New York, the Union Trust Co., and the Evelyn Apartments. One 24-year-old plant was noted in the Tower Building; one 23-year-old plant in the Nevada Apartments, and 22-year-old plants in Delmonico's at Beaver St., and the United Surety Building. An incomplete list of city plants was given as follows: 21 years' service, Presbysterian Building; 20 years, Evening Mail and St. Paul Buildings, New York Clearing House, Grace Chapel and Liederkrantz Club; 19 years, Potter, American Surety, Metropolitan, Commercial, Criminal Courts, Old Times Buildings and St. Luke's Hospital; 18 years, Bennett Building, Polhemus Memorial Clinic Dispensary, and Mechanics Bank of Brooklyn; 17 years, Waldorf-Astoria Hotel, Empire, Vincent and Sterling Buildings, Germania Bank, New York Athletic Club, National Bank of Commerce, O'Neill's Store, Terrace Garden, and Church of the Holy Trinity; 16 years, St. Paul's Methodist Church, the Ormonde and Seminole Apartments, Sprague Building, Metropolitan Museum of Arts, and Old Astor Library.

Design of the Superstructure of the New Quebec Bridge

[*With Insert Sheet Showing Design*]

By H. P. Borden*

Work on the reconstruction of the new Quebec Bridge over the St. Lawrence River is making very satisfactory progress. The work in connection with the substructure has extended over the past four years, and was finally completed during the past season with the exception of finishing the bridge seats and other minor details.

While the masonry included in this contract, amounting to some 106,000 cu.yd., probably constitutes a record for four piers, yet the only problems of any serious nature were those encountered in the sinking of the caissons for the two main piers. The first caisson for the north pier, as is known, met with an accident when it first grounded and had to be removed and repaired in dry dock. The bottom, as indicated by the borings, was composed of a mass of boulders on the north side and chiefly sand on the south side. In view of the very large size of this caisson (180x55 ft.), it was considered desirable by the Board of Engineers to minimize the chances of further accident as much as possible, and as a consequence, this larger caisson was transferred to the south shore and two separate caissons constructed for the north main pier.

These caissons were floated into position in June, 1911, and were carried to their final location by the end of October, of the same year, the westerly caisson being sunk at the rate of 0.37 ft. per day and the easterly caisson at the rate of 0.47 ft. per day. It was the original intention to sink these caissons to rock, but as the work progressed the sinking became more difficult, and finally, when the caissons had reached an elevation 20 ft. above rock and about 50 ft. below the bed of the river, it was considered that the foundations at this point were quite satisfactory for many times the load that the piers would be called upon to carry.

The foundation at this point was composed almost entirely of large and small boulders firmly wedged together, with only sufficient loose material to fill the interstices. Bearing tests were made on this material, with the result that a load of 59 tons per sq.ft. was carried with a settlement of ⅛ in., practically no settlement at all being noticed at from 20 to 30 tons. As the average working load on the foot of this pier will be only 8 tons per sq.ft., it was considered that there would be no occasion for carrying the foundation to a lower level.

In the sinking of the caisson on the south side, sand was met with for practically the whole depth, and this caisson was carried down to rock. The sinking was started in the latter part of July, 1912, and was completed the latter part of October. The caisson penetrated about 86 ft. below the bed of the river; the average penetration per day was 0.75 ft. The unique feature in the sinking of this caisson was the fact that no load was carried on the cutting edge, the caisson being supported under its bulkheads by means of sandjacks which were lowered simultaneously when the material had been excavated sufficiently under the cutting edge. This was found to work very satisfactorily, and some days the caisson was lowered as much as 24 in., most of the material being excavated by blowpipes.

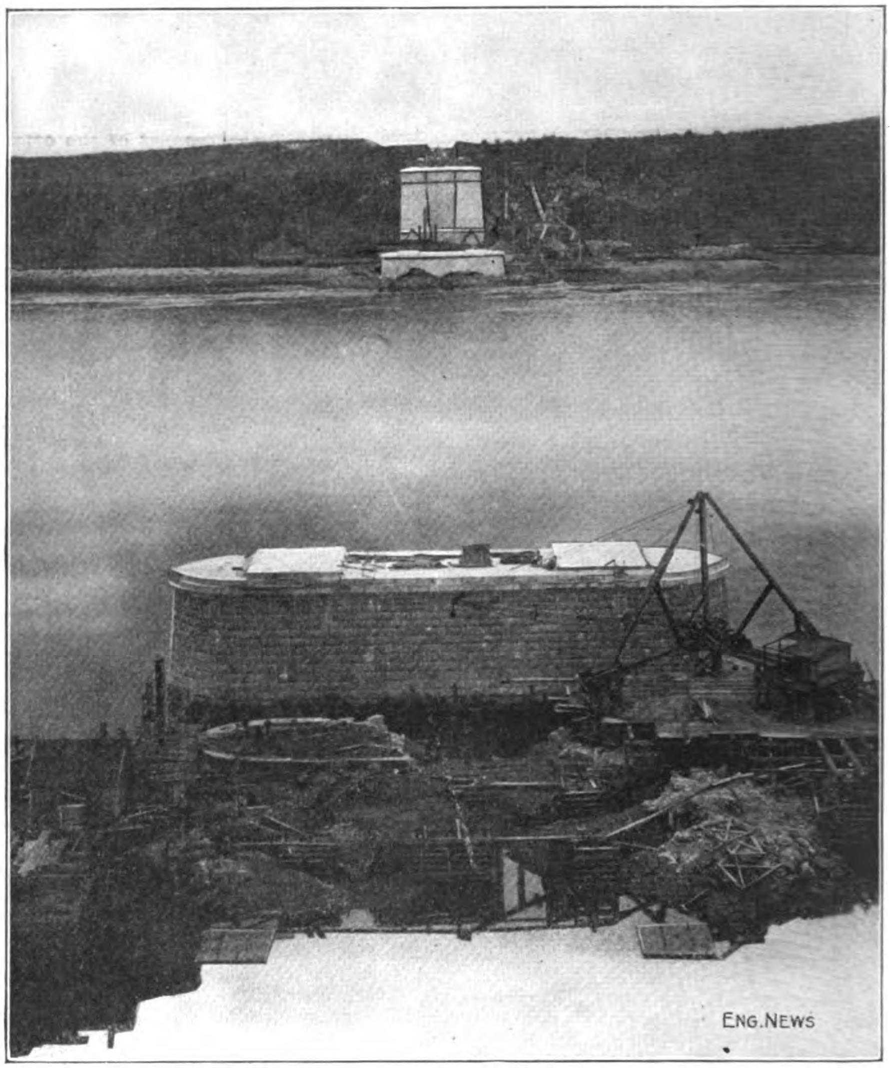


Fig. 1. Completed Substructure of New Quebec Bridge; View from North Anchor Pier

(North main pier in foreground; in front of it is the main pier of the old bridge, being demolished. In the background, south main pier and south anchor pier.)

The total amount of concrete and stonework in the various piers is as follows:

	Cu.yd.
North abutment (alterations)	404.05
Intermediate pier	1,665.06
Anchor pier	17,736.00
Main pier	31,870.04
South main pier	38,279.04
Anchor pier	16,073.00
Abutment (alterations)	61.01
Total	106,090.00

*Assistant to Chief Engineer, Board of Engineers, Quebec Bridge; New Birks Bldg., Montreal, Que.

Superstructure

The coming season will see considerable progress on the erection of the superstructure, it being expected that by the end of the present year the greater part of the north anchor arm will have been completed. The two north approach spans between the abutment and the intermediate pier were erected last season.

The work of fabrication has been going on in the shops for the past year and during this time some 8000 to 9000 tons of material—chiefly floor material— have been fabricated and shipped to the site.

Nickel Steel—The superstructure of the bridge is constructed partly of carbon steel and partly of nickel steel. The floor throughout is made of carbon steel, as in many cases the minimum allowable thickness of material governs the section of the member. The truss members of the suspended span are, however, all nickel steel, and the greater part of the cantilever arms. Practically all the anchor arms are carbon steel; the exceptions are a few members where it is necessary to use nickel steel in order to keep the grip of the rivets down to practical limits.

The proportions of the bridge as a whole as well as a large number of the members, constitute a record in bridge construction. The work of designing and erection has little or no precedent to govern it, and had to be worked out entirely independent of any previous experiences.

Principal Dimensions—The bridge provides for two railway tracks and two sidewalks. There is no provision made for highway traffic.

The bridge is 3239 ft. long face to face of abutments, including one 140-ft. approach span on the south end and two spans aggregating 269 ft. on the north end; thus the main cantilever structure is 2830 ft. long, or 30 ft. longer than the old Quebec Bridge. The span between main piers is the same, however—1800 ft. The anchor arms are 515 ft. long, the cantilever arms 580 ft. long, and the suspended span 640 ft. long, center to center of pins. The bridge is 88 ft. wide center to center of trusses. At midspan it has a clear height, above extreme high water, of 150 ft. The trusses are vertical.

The Truss System—The type of truss is of a somewhat unusual design, being of the so called "K" system. This type offers a number of advantages for a bridge of great size, especially during erection. Its chief advantage in this respect is that during the erection of the cantilever arms, it is possible to complete each panel by itself and move the traveler out to the end of the completed portion. This would not be possible with the Warren or single-intersection type without a large number of temporary supports for the members. With this construction, all permanent redundant members for supporting the main members are dispensed with, as every member of the bridge carries live-load with the exception of one small horizontal member at the main post and the supporting trusses carrying the top-chord eye-bars.

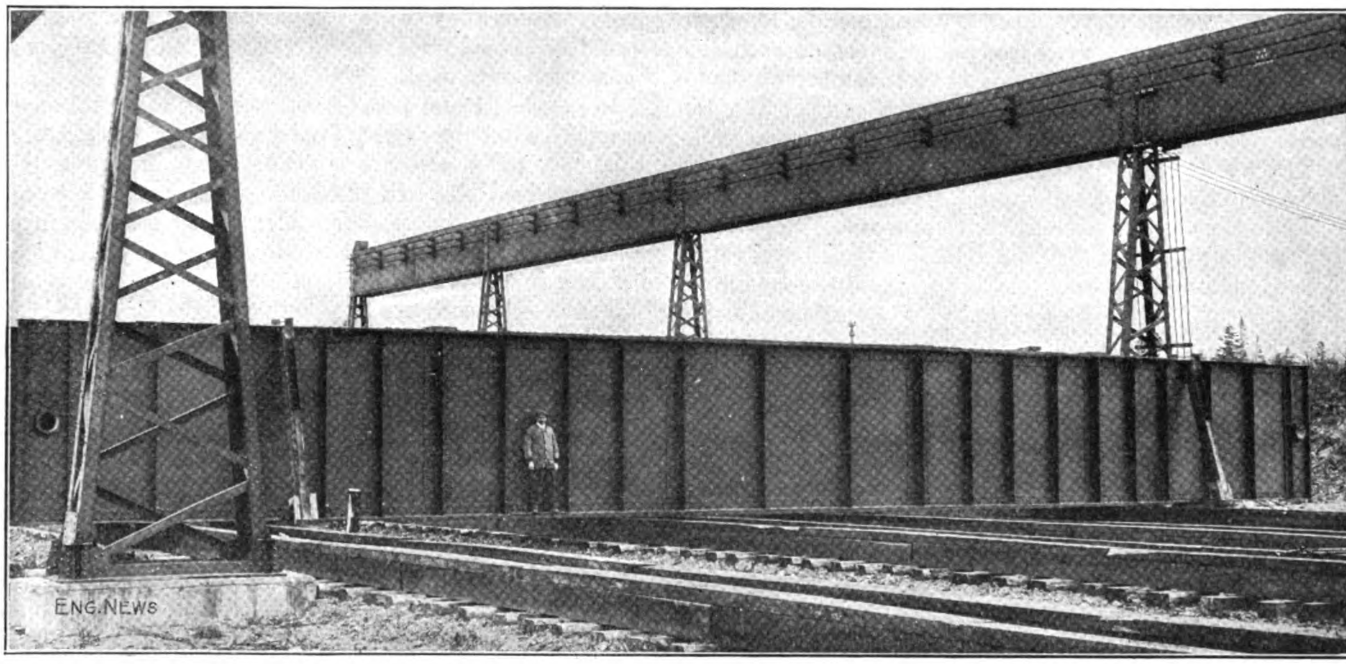


Fig. 2. Half of One of the Double-Web Floor-Beams of Quebec Bridge

(These two I-section halves are shipped separately and connected together in the field. Weight complete, 60 tons. Note the hole for the pin connecting floor-beam to vertical member of truss.)

Makeup of Main Members

The Tension Chord—Carbon-steel eye-bars are used for the top chords of the cantilever and anchor arms. Owing to the large number of bars required, it was found necessary to arrange them in *double rows* (upper and lower) for the entire length of the chord. The heavy section at the main post requires 32 bars $16x2\frac{3}{16}$ in., with a total sectional area of 1120 sq.in. On account of the length between main panel-points, it was found necessary to use *two lengths of eye-bars* between these points. In order to facilitate the work of erection, and also keep the eye-bars properly aligned after erection, these bars are carried on a supporting lattice-truss between panel-points, the trusses remaining permanently in the structure.

All other tension members in the cantilever and anchor arms of the bridge are built-up members. As the eye-bars which would be used for such members were so entirely out of proportion to the very large compression members in their vicinity, the use of the built-up tension members was justified from an esthetic point of view. In the case of the very long diagonal members, where at least three lengths of eye-bars would be required, the built-up tension members were eminently more desirable as they are able, by means of their flange angles, to support their own weight with little or no deflection. This would not be the case with eye-bars unless supported by auxiliary ties or other redundant members.

THE COMPRESSION CHORD—The lower-chord members are very large. All are composed of four webs tapering in depth by the amount of $\frac{1}{32}$ in. in 12 in. The largest of these chords is 7 ft. 2 in. deep and 10 ft. 4 in. wide. This member has a cross-section of 1902 sq.in., and its weight between main panel-points is approximately 400 tons. In order to facilitate shipment and erection, however, this member is spliced at the center, and further each of these two sections is divided longitudinally. This brings the weight of the member to be handled down to approximately 100 tons. The pair of webs on each side of the longitudinal center line of the member are shop-riveted together with 8½x1-in. lattice-bars at the top and bottom and a horizontal plate diaphragm at mid-height, and is also held together and squared by transverse plate diaphragms throughout the length of the member.

The webs of these members are, as a rule, composed of four plates to each of the four webs. The maximum thickness of plate used on the webs is 1 in., and the maximum thickness of one web is 3¾ in. The question of using thicker plates was considered, but it was felt that, as a rule, better material could be obtained in the thinner gages, and, as stitch-riveting would have to be used in any case, little or no benefit could be derived from the use of the thicker plates. It was also possible to get wider and longer plates in the thinner sizes, and thus reduce the number of shop splices.

SPLICES—The splices of all compression members are designed with 100% of rivets. This was felt to be preferable in these very large members, taking into consideration the possible uncertainties of getting a true planed surface for the abutting ends.

MAIN POST—The main post, over the piers, is probably the largest single member of this type ever built into a bridge. It is 310 ft. long center to center of pins and has an unsupported length of 145 ft. It is composed of four separate columns latticed together, its outside dimensions being approximately 9x10 ft. Its greatest cross-section is 1903 sq.in. Each post weighs 1500 tons. For erection purposes it is required to be shipped in 26 sections.

The detail of the top of this post is the heaviest on the bridge, it being required to distribute the stress of five very heavy members. In order to keep the rivet grip within the specification, it was necessary to use nickel steel. This connection is shipped in two pieces, each of which weighs 65 tons.

MAIN SHOE AND CONNECTIONS—The shoe, under the main posts, has a bearing surface on the masonry of 22 ft. by 26 ft. and is 19 ft. high over all. It is constructed to be shipped and erected in seven pieces, and weighs complete approximately 400 tons. The lower section is composed of four steel castings 4 ft. high, each of which weighs approximately 40 tons. The next section is constructed of plates and shapes and is erected in two pieces. This section takes the thrust from the lower chords of cantilever and anchor arms. The upper section is erected in one piece and carries the two diagonal compression members and the main post.

In order to keep down the thickness of bearing plates in the chord adjoining the shoe, a sleeve is used to distribute the bearing of the pin. This is used for the two chords and the main post. These sleeves have an outside diameter of 45 in., the pins themselves being 30 in. in diameter.

In order to facilitate erection, all pins through members with four webs are in two lengths, there being a separate pin for each two-web section. The 30-in. pins at the shoe and post weigh a little over 6 tons each.

FLOOR-BEAM CONNECTION—Contrary to usual practice, the floor-beams (with one or two exceptions) are not riveted to the vertical posts of the bridge, but are connected by an 11-in. pin passing through the post and the web of the floor-beam. This method of connection was adopted partly to facilitate erection, but principally to do away with the bending in the post that would be caused under full live- and dead-load if a rigid connection were used.

In panels L1 and L2 of both the cantilever and anchor arms it has been necessary to make a rigid connection. In this case, the floor-beam is connected to the post under full dead-load plus half live-load. This means that under dead-load alone the post is under initial bending in one direction, while under full live-load it is subject to equal bending in the other direction.

FLOOR—At each main panel-point double floor-beams are used, whose webs extend past either side of the tension vertical. At the intermediate panel-points, however, single-web floor-beams are used, supported by means of hangers or posts attached to the intersection of the diagonals.

The floor-beams usually are 10 ft. deep back to back of angles, and weigh complete from 50 to 60 tons. The main stringers are through plate-girders framed with sub-floor-beams and I-beam stringers. The top flange is stiffened laterally with a heavy channel, and gusset plates connect the sub-floor-beams to the webs. By this arrangement it was thought to give an added sense of security to those using the bridge as well as an actual factor of safety in case of derailment.

WIND SYSTEM—All the wind stresses are transferred to the piers through the bottom lateral bracing, there being no lateral bracing at the top chord. The wind stresses in the web members are carried down the compression members to the bottom chord. There is a separate system of lateral bracing for the floor, which at each main panel-point transfers its stress down through the swaybracing to the bottom lateral system.

The lateral reaction from the wind at the end of the anchor arm is taken care of by means of a steel wind-anchorage embedded in the pier which allows a longitudinal and vertical movement of the truss but no lateral motion.

TRACTION AND EXPANSION—Traction forces are carried to the bottom laterals through specially designed traction trusses at panel points *AL*1, *CL*1, *AF*9 and *CF*11, at which points there are expansion joints in the floor system.

The expansion of the cantilever arms and suspended span from temperature is taken up at the two ends of the suspended span. This is effected by means of a specially designed expansion brake. This brake is adjusted to stand a force of 250,000 lb. in either direction.

RIVETS—The size of the rivets varies from 1⅛ in. to ⅞ in. All rivets are carbon steel. The ⅞-in. size is allowed up to 3½-in. grip, the 1-in. size from 3½- to 5½-in. grip, and the 1⅛-in. size for 5½-in. grip and over. The greatest grip of rivets occurs at the field splice of the bottom chord next the shoe, where 9⅞ in. is required. When the grip of the rivet exceeds four diameters, the allowable unit-stress of the rivet is reduced by 1% for each $\frac{1}{16}$ in. of additional grip. This does not apply, however, to compression members having butt joints.

All rivets over 5 in. long have a taper of $\frac{1}{32}$ in. in 12 in. The size under the head is $\frac{1}{32}$ in. smaller than the diameter of the hole.

No material less than ½ in. in thickness is allowed in main members. Material ⅜ in. in thickness is allowed, however, in details, such as lattice bars and tie-plates of the lateral and sway bracing, provided the requirements of the specification as to unsupported length, etc., are fulfilled.

Design Loading

The bridge is designed for 5000 lb. per lin.ft., covering both entire tracks, with two E60 engines, the engines and train loads being placed to give the maximum condition of loading.

Wind load was assumed at 30 lb. per square foot of exposed surface of the two trusses and 1½ times the elevation of the floor; and 300 lb. per lineal foot as a moving load on exposed surface of the train. A wind load of 30 lb. per sq.ft. *parallel with the bridge* was also assumed acting on one-half the area assumed for normal wind pressure.

In considering temperature stresses, the following conditions were assumed:

Variation of 150° F., on the uniform temperature of the whole structure. A difference of 50° F. between the temperature of steel and masonry. A difference of 25° F. between the temperature of a shaded chord and the average temperature of a chord exposed to the sun. A difference of 25° F. between the outer webs exposed to the sun and the inner webs of compression members.

The substructure was constructed by M. P. and J. T. Davis, of Quebec, under the direction of S. H. Woodard.

The St. Lawrence Bridge Co. is the Contractor for the superstructure; Phelps Johnson is President, G. H. Duggan is Chief Engineer, and G. F. Porter is Engineer of Construction. The work is being supervised by the Board of Engineers Quebec Bridge, composed of C. N. Monsarrat (Chairman and Chief Engineer), C. C. Schneider, and R. Modjeski.

An Early Experimental Pavement--An Interesting Bit of Pavement History

It has been customary to refer to the experimental pavements on Nelson Ave., Columbus, Ohio, those of the United States Office of Public Roads at Ithaca, N. Y., the Second Ave. pavement, New York City, and the Byberry Turnpike, Philadelphia, as pioneer attempts to study the pavement problem "scientifically" by comparing the effects of the same traffic on adjoining strips of different kinds of pavement surfaces. Read the item reprinted below, from the *Civil Engineers and Architects Journal,* London, England, October, 1839, and the old old proverb that "there is nothing new under the sun" inevitably occurs to mind.

THE EXPERIMENTAL PAVING OF OXFORD STREET

The extended time allowed by the Marylebone vestry for testing the durability of the various specimens of experimental paving laid down in Oxford street, having expired on the 3rd ultimo, a large body of members of the Experimental Paving Committee proceeded to Oxford street, for the purpose of entering into a minute examination of the specimens, prior to completing their final report and recommendation to the vestry as to the plan which it would be most advisable to adopt. The blocks of granite laid down, and the interstices of which were filled up with Claridge's Asphalte, were found to be in excellent condition, as was also the granite laid down by the parish and grouted together. The Bastenne Guajac bitumen had stood the test of wear occasioned by the great number of vehicles passing through this extensive thoroughfare in a surprising manner, but at parts where the traffic is most severe, here and there, slight ruts are perceptible. On arriving at the wooden blocks, the surface was found as smooth and even as when first laid down. Five of the blocks were taken up and minutely examined by the Committee and one of them split to pieces for the purpose of discovering if any symptoms of decay had made its appearance, but the wood was found to be perfectly sound, and the diminution of the length of the blocks [evidently means height] (12 in.), notwithstanding the immense weight of vehicles continuously passing over them, was scarcely perceptible. Having completed their survey of the road, the Committee adjourned to the Court-house for the purpose of deliberating as to the best mode to be adopted, when a long discussion ensued upon the subject. Mr. Kensett supported the adoption of the wood, and Mr. Harbutt and several others opposed it on the ground that the material was of too slippery a nature for horses. After a variety of arguments in the course of which three or four amendments were put and negatived, the following resolution was put and carried—viz. "That it appears to the Committee that wooden block paving has proved itself equal to the traffic and paving of the whole of Oxford street, and it is, therefore, resolved to recommend to the vestry to adopt the wooden block paving for the thoroughfare, subject to certain conditions and regulations." The greatest interest is manifest on this subject in Marylebone.

This item is of much interest for another reason. It is, so far as we can determine, the first mention of a bituminous pavement laid in London. Tillson in his "Street Pavements and Paving Materials" (1912) states (p. 8):

A sample of asphalt macadam was laid on the road between Bordeaux and Rouen in 1840. This was a mixture of asphaltic rock and ordinary stone, and was probably the first bituminous roadway laid on a public highway, although about the same time asphaltic rock was used for sidewalks on some of the streets of Paris.

We are unable to determine the nature of the "Bastenne guajac bitumen" pavement, but the context shows clearly that it was a kind of asphaltic pavement.

A subsequent issue of the same periodical contains a note to the effect that wood block was ultimately adopted for the whole of Oxford St. after an acrimonious controversy between the advocates of the wood block and the advocates of the granite block. Moreover, this was the first real triumph of wood-block pavement in London. Its adoption for Oxford St. led to the formation of a local "pavement improvement" society, whose sole object was to make an impartial study of different pavements to discover that phantom, for which the general public still seeks—the universal pavement.

Fifty-Thousand-Dollar Verdict for Injured Miner—A premature explosion in one of the Delaware, Lackawanna & Western R.R. coal mines at Luzerne, Penn., severely injured Matt Yarkanis, a miner. The railroad was charged with negligence; and the verdict of the jury in the Federal District Court of Brooklyn, N. Y., on Apr. 4, granted Yarkanis $50,000 damages. The plaintiff asked for $75,000.

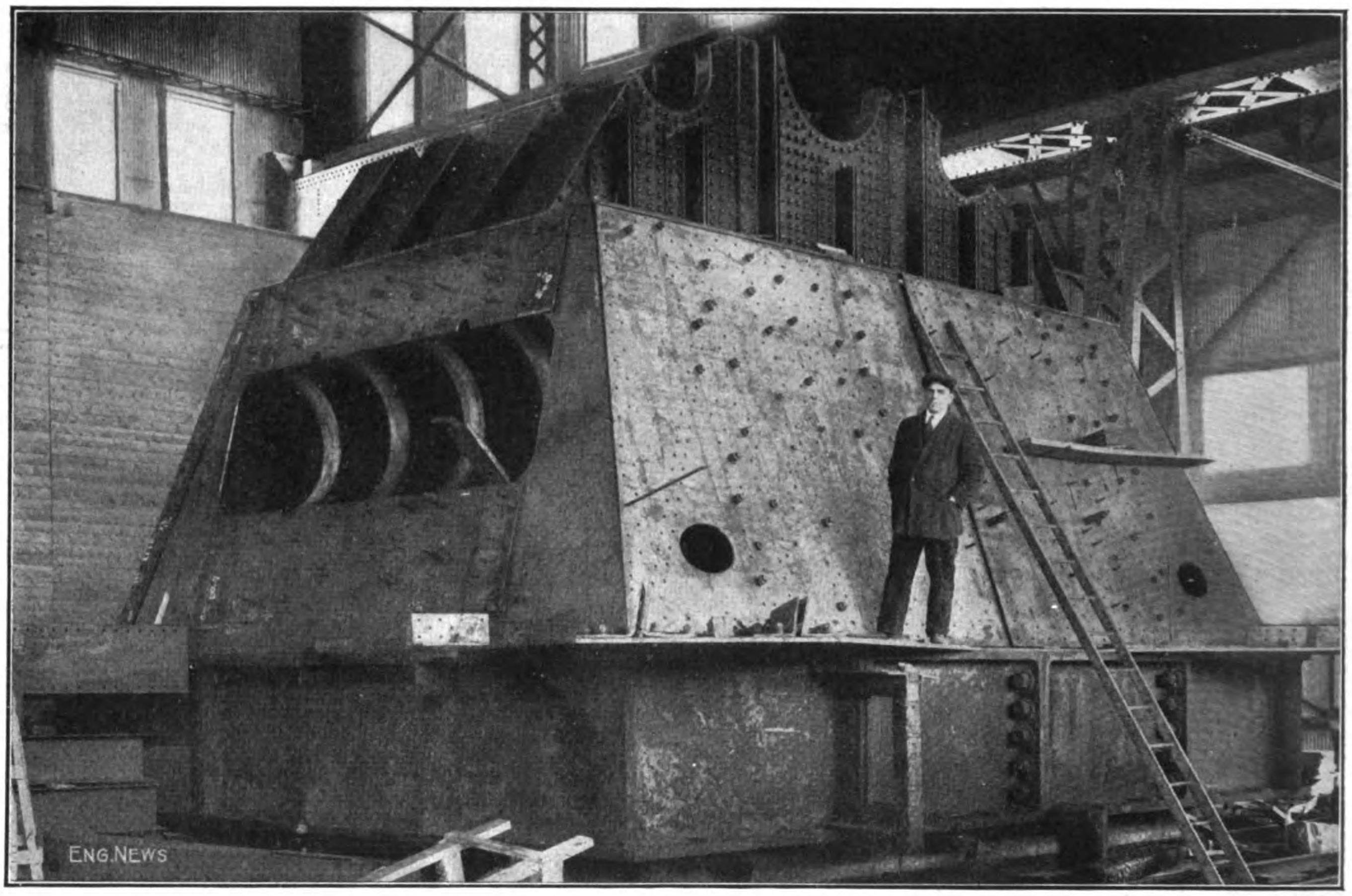
Special Shopwork on the Heavy Members of the Quebec Bridge

By H. P. Borden*

SYNOPSIS—Steel fabrication of unprecedented kind is involved in the construction of the new Quebec Bridge. Among the factors are: Metal thicknesses over 9 in., 1⅛-in. rivets, nickel steel, great weight of parts and of completed members, drilling from solid for nearly all rivet-holes, planed faces up to 10x20 ft., 45-in. pin-holes. The equipment and shop methods are described. Large detail photographs illustrate the account.

During the past fourteen months, the fabrication of members for the new Quebec Bridge has been steadily going ahead at the shops of the St. Lawrence Bridge Co. near Montreal. As an entirely new plant was built and new machines constructed for this work, the contractors were compelled to face the problem of initiating the construction of a bridge whose members have no equal in bridge construction at the present day, with an untried equipment and a brand new organization. The difficulties of constructing ordinary bridge or structural work under similar conditions would be apparent, but when there is coupled to these conditions the unprecedented size of the members and the unusually stringent specification governing the work, it can be seen that the difficulties were very considerably augmented.

Fortunately, the contractors were able to first put through the shops the floor system, which is considerably simpler than the truss members and therefore served to get the organization in running shape. The floor system for the anchor and cantilever arms, for both sides of the river, has all been manufactured and shipped to the site, and since the first of the year a definite start has been made on the fabrication of the huge members which go to make up the North Anchor Arm.



One of the Great Shoes of the Quebec Bridge, Assembled in the Shop for Fitting and

Plant and Machinery

In order to adequately cope with the difficulties involved in the manufacture of this steelwork, it was necessary to equip these shops with machinery and handling facilities of, in many cases, unprecedented size, capacity and accuracy. This includes machine tools for precise work in finishing bearing surfaces up to 10 ft. square, boring machines capable of boring pin-holes (in compression members) up to 48 in. in diameter, and a planer capable of surfacing castings or bed plates 30 ft. long and 10 ft. wide.

The shop itself (Fig. 4) is 660 ft. long, 160 ft. wide for 440 ft. of its length and 190 ft. wide for the remainder.

*Assistant to Chief Engineer, Board of Engineers, Quebec Bridge, Montreal.

The main shop has one row of columns down the center, the remainder of the space being clear of posts in order that material could be handled with the least amount of difficulty. Material is handled in the receiving yard by two 7½-ton trolley hoists on a 15-ton electric traveling crane with 90-ft. span on runways 500 ft. long. Material entering the shop passes consecutively through the edge planers, shears, drilling tables, etc., to the punching and riveting machines. From this point it branches to the assembling skids, the heavier members going to one side of the shop, which is equipped with two 70-ton electric traveling cranes, while the lighter material goes to the other side, equipped with two 35-ton electric cranes. The latter is intended for members weighing 70 tons or less; the former is capable of handling the largest members to be fabricated, some of which weigh about 190 tons. This means that the two 70-ton cranes are at times compelled to take considerable overload, but it is found that they are capable of handling these heavier members with little or no difficulty.

Many members in this bridge must, on account of their size, be shipped in a number of pieces. This means in most cases that the various parts of one member have to be fabricated and machined with entirely separate operations and riveted together in the field. In order to guarantee that these several parts will go together as designed, extreme precautions are necessary in their fabrication. With this in view, the specification for this bridge has been prepared with unusual care with the idea of obtaining results consistent with the best known shop practice.

Planing and Facing of Material

According to the specification *all sheared edges shall be planed,* at least ⅛ in. being removed.

Long plates are planed on two heavy machines with a capacity of 46 ft. in length. The planing of sketch plates is done on a similar but considerably smaller machine. The plates are held in position on the table by pneumatic clamps, which operate very quickly. The tool carriage travels the length of the planer, carrying the operator, and cuts in both directions. The ends of universal plates, and angles are planed on twin milling machines by which both ends are faced simultaneously, finishing to the exact length. In this machine, it is possible to fasten six or eight plates or angles together and mill them at the one time.

The ends of stiffener angles are faced on a special machine which grinds one edge to fit the fillet of the flange angles. Small material, such as connection angles and small lattice bars, is usually ground on an emery wheel. Heavy lattice bars whose ends are cut to different angles, go to a special machine which mills these two faces simultaneously.

Punching, Reaming and Drilling

According to the specification, all material $\frac{11}{16}$ in. and under may be subpunched and reamed, while thicker material must be subdrilled and reamed or else drilled from the solid. As the minimum thickness of material allowed in main members is ½ in., and a large proportion of the material considerably exceeds that thickness, most of the work is drilled. The shop has the usual equipment of punches as well as an automatic multiple punch, yet chief attention naturally was paid to the equipment of drills and reamers.

The drilling plant includes 16 heavy stationary radial drills mounted on a long foundation at the center of the shop and parallel to its axis, and 24 similar portable drills each mounted on an individual truck traveling on a portable track which can be clamped to the concrete floor. All the drills have 6-ft. arms with vertical adjustment and have locking devices for attaching them to the track. Variable-speed motors furnish the power. There are also 12 horizontal drills mounted on trucks to work in connection with the radial drills and for use in drilling field splices in main members.

According to the specification all reaming and drilling shall be done *dry,* without the use of water or other lubricant.

A long series of tests has been made by the shop on high-speed tools without lubricant, and while it was found there is no serious difficulty in drilling plates of moderate thickness, yet considerable difficulty has been met when thicknesses of 4 in. or more are to be drilled. This is especially true in the case of nickel steel. At the present time, the shop is using a thick red-lead paste with which the tool is painted during the drilling of thick plates, and it is found that this works very efficiently and has not the objectionable features that are found with either water or oil.

As a rule, three sizes of rivets are used, namely, ⅞ in., 1-in., 1⅛ in. With these machines it is found possible to drill a $\frac{15}{16}$-in. hole at a feed of 3 in. per minute; $1\frac{1}{16}$- and $1\frac{3}{16}$-in. holes are drilled at about the same rate.

A large proportion of the drilling is through exceedingly thick material, made up of a number of plates or angles. On the bottom chords, the thickest material is through the splice-plates of the chord next the shoe, where the rivet-grip amounts to 9⅞ in. The connection at the top of the main post has a maximum thickness of 7 in. of nickel steel.

In assembling the webs of the chords, tack holes are subdrilled in the plates and angles for the purpose of assembling, after which the full-size holes for shop rivets are drilled from the solid. Field connections are either subdrilled, and reamed after the member has been assembled in the shop, or are drilled full size to a steel template.

As misdrilling or mispunching on these very big members may prove very costly, all center-punch marks for drilling are *encircled in white paint or chalk* so that the chance of error is reduced to a minimum.

Where a steel template is used the specification calls for a plate at least 1 in. thick. In practice, however, it has been found that a template ¼ in. thick, having the holes fitted with hardened steel bushings 1 in. high, gives better results, as the hardened steel tends to prevent any drifting of the tool should the subpunched or drilled hole be somewhat irregular.

Special attention is paid to keeping the several parts of a member thoroughly bolted up while being drilled in order to prevent filings entering between the various thicknesses of plates. This is satisfactorily effected by staggering the holes when drilling and following up immediately with bolts.

Facing the Ends of Complete Members

Probably the most carefully watched work in the shop is the facing of the compression members and the boring of the pin-holes.

The main bottom-chord members, measuring 10 ft. by

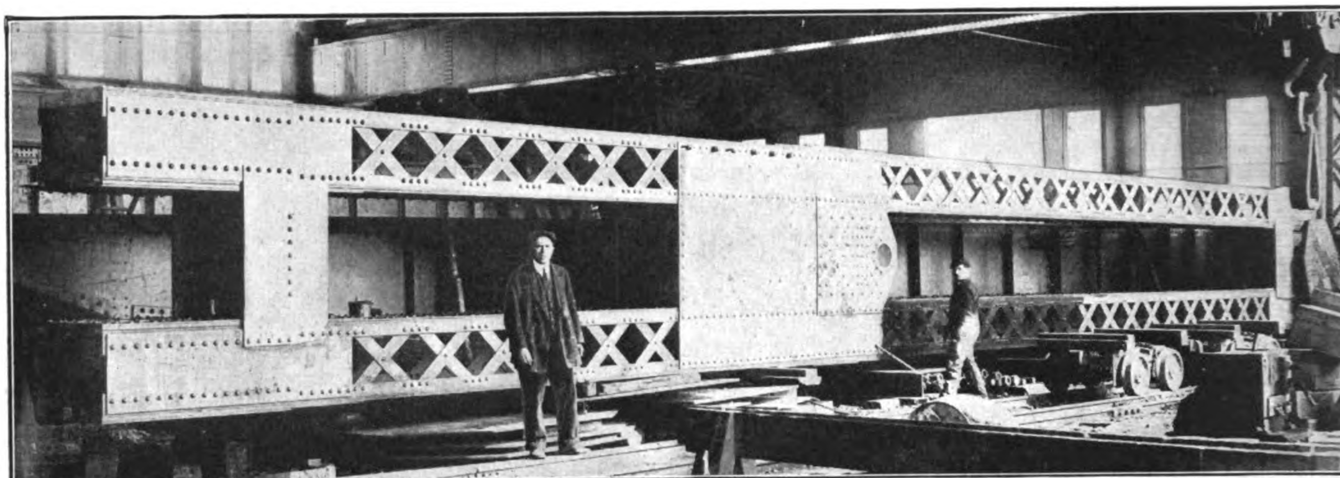


FIG. 1. VERTICAL TENSION-MEMBER AM6-L6, READY FOR SHIPMENT

(The pin-hole at the middle takes the pin which supports the floor-beam, whose two webs extend past the post either side.)

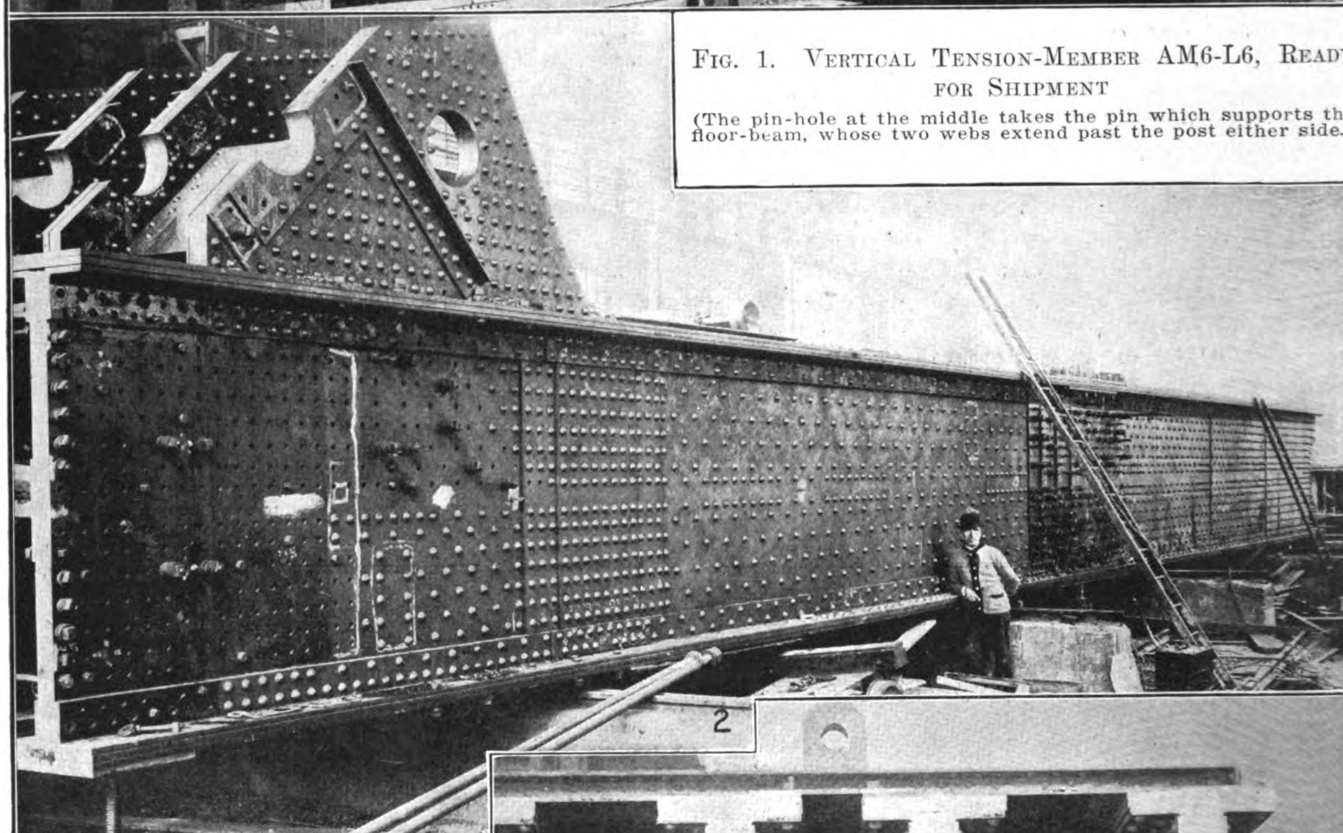


FIG. 2. BOTTOM CHORD AL12-AL14 ASSEMBLED FOR REAMING OF FIELD SPLICE

(This is the full panel-length. It is shipped and erected in four pieces, spliced longitudinally between the two inner ribs and transversely at mid-length.)

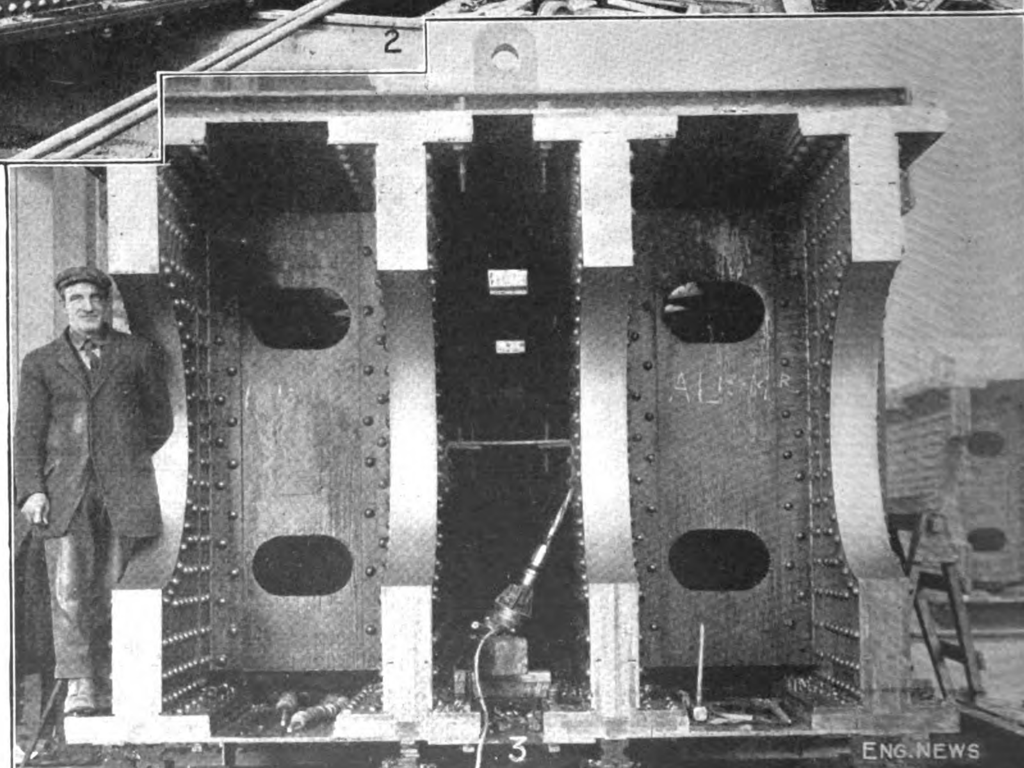


FIG. 3. FORTY-FIVE INCH PIN-HOLE IN PIER END OF BOTTOM CHORD AL12-AL14

(The 45-in. hole takes a cast-steel bushing, in which a 30-in. pin is seated. This view shows the far end of the member pictured in Fig. 2.)

THE QUEBEC BRIDGE IN THE SHOP

7 ft. in outside dimensions, are faced in a duplex vertical and horizontal planing machine which is capable of facing both ends of the chord simultaneously. One of the machines is stationary, the other is movable to any position on a 25x75-ft. bed. The heads of these machines can make a 10-ft. cut in either a vertical or horizontal direction and are equipped with patent tool holders for cutting on both direct and return strokes.

The operation of setting a chord member in the correct position in the machine is a somewhat delicate and laborious one, taking from one to two days to accomplish. As the chords are tapering in a vertical plane, the difficulty of setting their center line true is increased. The practice is to determine this center line on the web of the member by numerous measurements and drive a tapered pin in 1/8-in. holes bored exactly on this center line; on these pins, used as benchmarks, the horizontal position of the piece can be ascertained by means of a field level. The greatest care is taken by numerous checks and rechecks to make absolutely certain that the member is in its true theoretical position before any work is done.

On the first chords that were milled, the planing was effected by making the roughing cut on the downward stroke of the machine and the finishing cut on the upward, thus finishing the milling when the machine had traveled completely across the member. This method was found to be not entirely satisfactory, as it took five days to make one cut across the member, and changes of temperature in consecutive days would be liable to affect the length materially. In one case, the length was so seriously affected that the member had to be refaced. The present practice is to face the end first with the roughing cut, the machine feeding both on the up and down stroke, and then going completely across the second time with the finishing cut with a little faster feed. By this means, each operation can be completed in from 15 to 20 hours. In order to ascertain whether there has been any wear or give of the tool, the machine is run across the face after it has been finished and the distance between the tool and the end of the member determined by means of "feelers." From 0.001-in. to 0.004-in. variation from a true surface is the maximum that is generally detected in this way.

Smaller members, such as the tension verticals and other members which do not take end bearing, have their ends milled on the rotary facing machines.

Planing the Shoes

One of the most laborious planing operations is the facing of the enormous steel castings which form the first story of the shoe. These castings are 20 ft. 10 in. by 6 ft. 8 in., and 4 ft. high, and weigh approximately 40 tons each. There are four of these castings under each shoe. Their webs, flanges and interior diaphragms are from 2½ in. to 3 in. thick. These castings are planed on a large planing machine designed especially for this work, in which the cutting tool has a travel of 10 ft. transversely and 30 ft. longitudinally. The castings are faced top and bottom and all four sides. There are two cutting tools fitted on the horizontal crosshead, so that the operator is able to start one tool from the center of the casting and the other one at the edge, reducing the time of the total operation by one-half. The finishing cut, however, is done with one tool across the entire face of the casting. It requires about 20 hours for a single tool to make one entire cutting.

The sides are planed without turning the casting on edge, by a cutting head on each side attached to the vertical head of the machine.

Under fair conditions, it takes about two weeks to completely plane one of these castings. As there are sixteen required, this operation alone requires from eight to nine months' continuous work.

On the first one or two castings that were finished it was found, on checking them up, that while the ends were exactly to gage there seemed to be a slight concavity near the center, in one case amounting to nearly $\frac{1}{32}$ in. Exhaustive tests were made on the machine to determine how this could have occurred, but no explanation could be arrived at except release of internal stresses in the casting when the outer crust had been removed by the cutting tool, thus causing a slight warp. To guard against such contingency, it is now the practice to finish one face, leaving about $\frac{1}{16}$ in. for the final cut, then turn the casting over and completely finish the opposite face. The casting is then again turned and the final cut made on the original face. It has been found that much more satisfactory results have been obtained by following this program.

Boring Pin-Holes

The boring of the pin-holes and half pin-holes in the gusset-plates of the chords and in the shoes is performed on a large boring mill which has a boring capacity up to 48-in. diameter, with a longitudinal motion of 23½ ft. and a vertical motion of 15½ ft. The vertical arms carrying the crosshead and boring bar travel on a steel bed embedded in concrete and are operated by a rack and pinion geared to an electric motor. When the member is once assembled in its proper position on the bed of the machine, it is therefore not necessary to move it until all the holes are bored. In such an operation as boring the shoe, where there are five distinct pin-holes, these holes can be drilled without moving the member, thus assuring that all holes are absolutely parallel and at right angles to the vertical and longitudinal axes of the member.

The largest half pin-holes to be bored are those at the end of the bottom chord and in the shoe, which receive the 45-in. cast-steel bushing which in turn takes a 30-in. pin. There is also the same size of pin and bushing on the top of the shoe which takes the reaction from the main post.

The pin-holes in the smaller tension and compression members are bored on the twin boring machine on the other side of the shop. Pin-holes in members up to 100 ft. long center to center, can be bored simultaneously on these machines. When it occurs that members exceed this length, as is the case with a number of the members, it is necessary to bore each end separately, making a separate adjustment of the member itself for each operation.

The boring of the pin-holes in eye-bars is performed on another special eye-bar-boring machine, which bores the holes in both ends of the eye-bar simultaneously. The pin-hole is $\frac{1}{32}$ in. larger than the normal diameter of the pin, plus 0.01 in., which is allowed for wear of tool. In order to minimize the chance of error from undue wear of the tool, only one eye-bar is bored at a time. The specification requires (as customary in other bridgework) that when the bars for one full panel have been bored they shall be placed one upon the other and the pin shall be required to pass through the holes at both ends at the same time without difficulty.

In order to facilitate erection and offset the possible effects of settlement of the falsework and the deformation of the web members under their own weight, pinholes in both ends of the top-chord eye-bars, and in the upper ends of tension diagonals of the anchor arms, are bored oblong to the extent of ½ in. As there are two lengths of eye-bars in each panel, this means there is a possible adjustment of 2 in. in each main panel of the top chord. The back-to-back spacing of these oblong pin-holes corresponds in each case to the calculated length of member as designed to take care of the final camber, the elongation of the hole being toward the center of the member.

As this detail is somewhat unusual in bridge practice, a series of tests was made by the St. Lawrence Bridge Co., at Ambridge, Penn., to determine the relative elongation of eye-bar heads with oblong and round holes. Tests were made on six bars having a round hole at one end and an oblong hole at the other. Readings were taken on these bars for a distance of 3 ft. from the back of each pin-hole. It was found that under a load of 20,000 lb. per sq.in. the elongation and permanent set in the oblong heads was on an average greater than that on the other end by about 0.004 in. and 0.002 in., respectively. Under a load of 28,000 lb. per sq.in., the elongation and permanent set at the oblong heads were greater by 0.016 in. and 0.0111 in. respectively. As all the bars broke in the body, it is considered that the results were satisfactory, the difference in results found in the two heads being practically negligible.

Assembling

The specification requires that all field splices in chord and web members shall be assembled in the shops and reamed in place. The operation of assembling the sections of the main chord is a very long, laborious task. As these members have four webs, with horizontal diaphragms between outer pairs, it means there are twelve separate sets of splice plates to be attached to the webs in addition to four sets to be attached to the horizontal diaphragms (which are figured as cross-section in the member). In order to facilitate handling, these splice plates, which are made up of several plates, are tack-riveted together with a countersunk head in the inner face in order that each may be handled in the field as one piece. In addition to these, there are two heavy tie-plates top and bottom.

Although these joints have been designed with 100% riveting, every effort is made to bring the two faces of the chords to a bearing fit before the joints are reamed. It is the practice to drill the splice-plate full size from a template, and to do the same with the webs on one side of the joint. The splice-plates can then be bolted securely up to one half-section of the chord while the other half is held in place by about 10% of tack bolts. When the joint has been satisfactorily assembled, the remaining holes in the webs are drilled from the solid, using the splice-plate on one side as a template. All splice-plates and other loose plates are then matchmarked with steel dies before being taken apart, to insure their being put back in the field in exactly the same location.

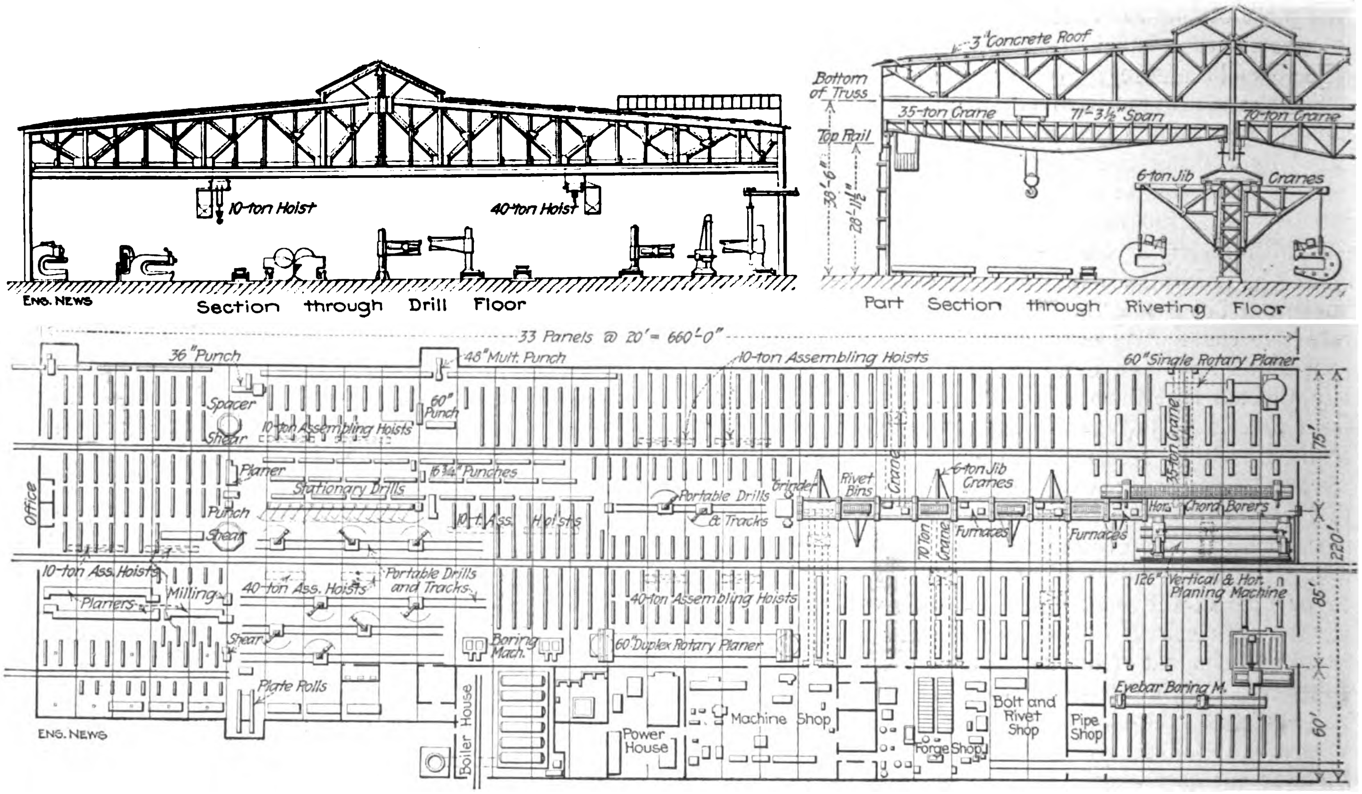
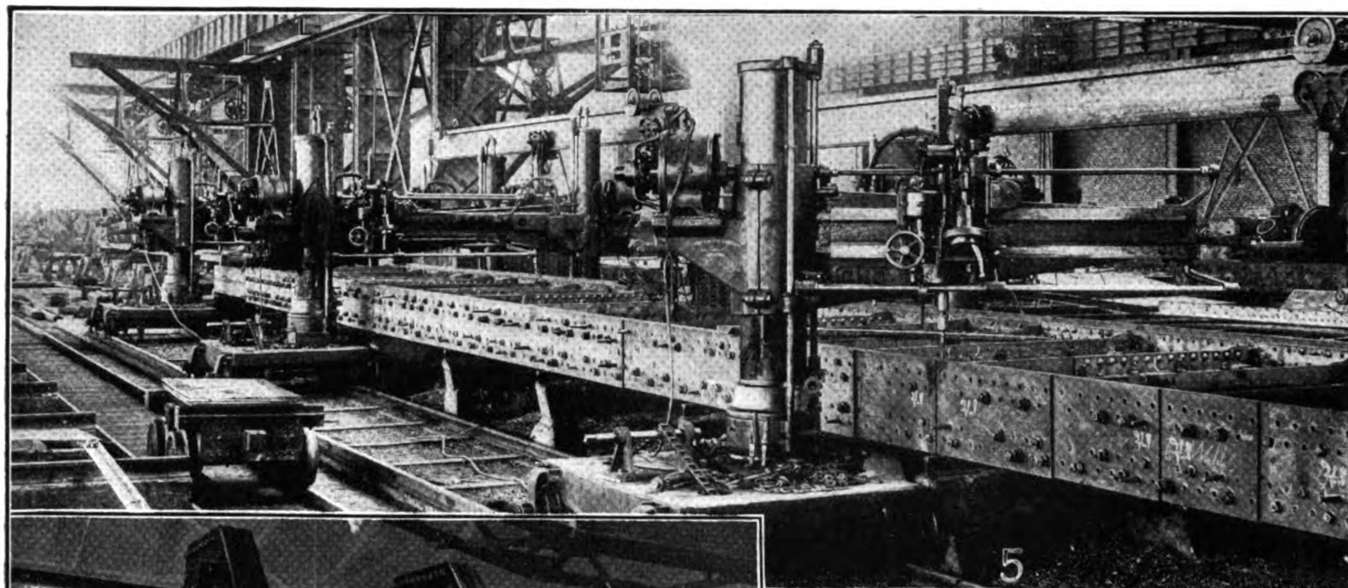


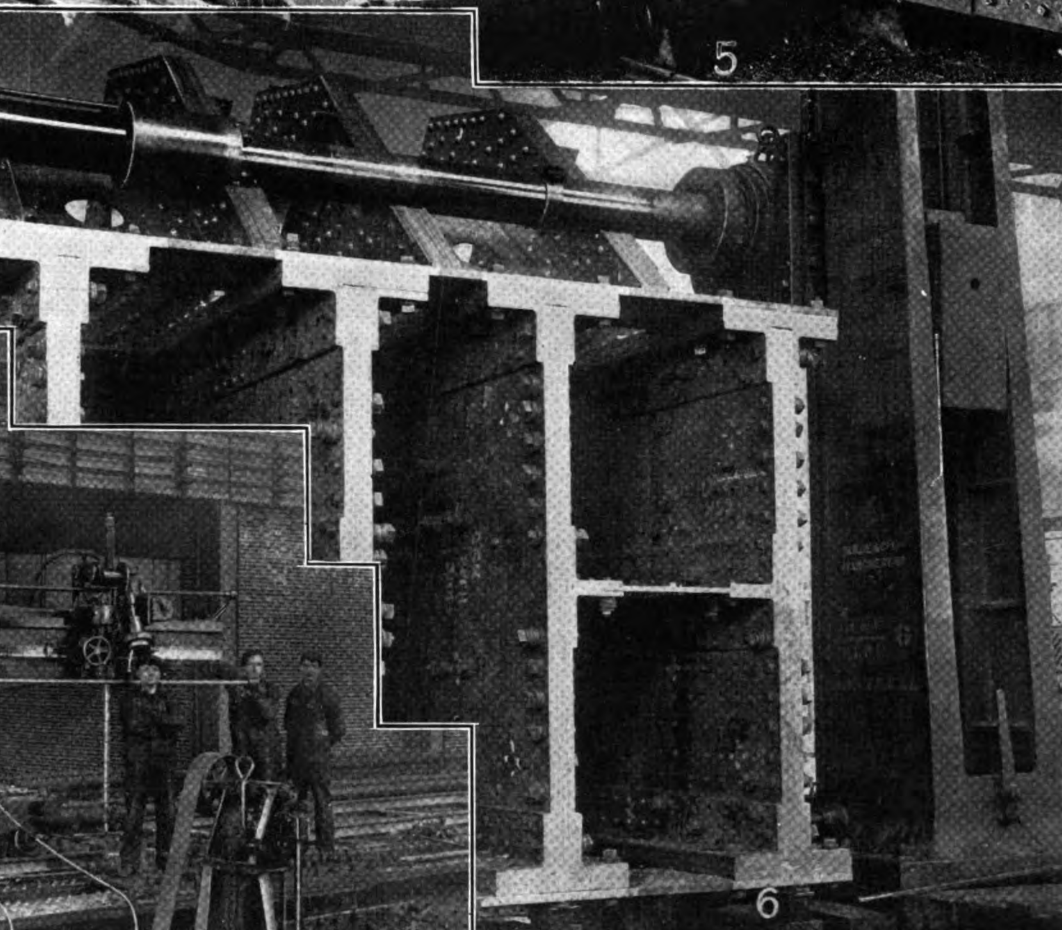
Fig. 4. Plan and Sections of Shop for Fabrication of Quebec Bridge

Riveting

All rivets for both field and shop are of carbon steel. As already mentioned, three sizes are employed, viz.: ⅞-in., 1-in. and 1⅛-in., the diameter being determined by the grip.

Both in shop and field, all rivets are required to be heated in oil furnaces, and special precautions are taken to prevent burning and scaling. Rivets are driven by pneumatic yoke machines with gaps of from 24 to 72 in.,





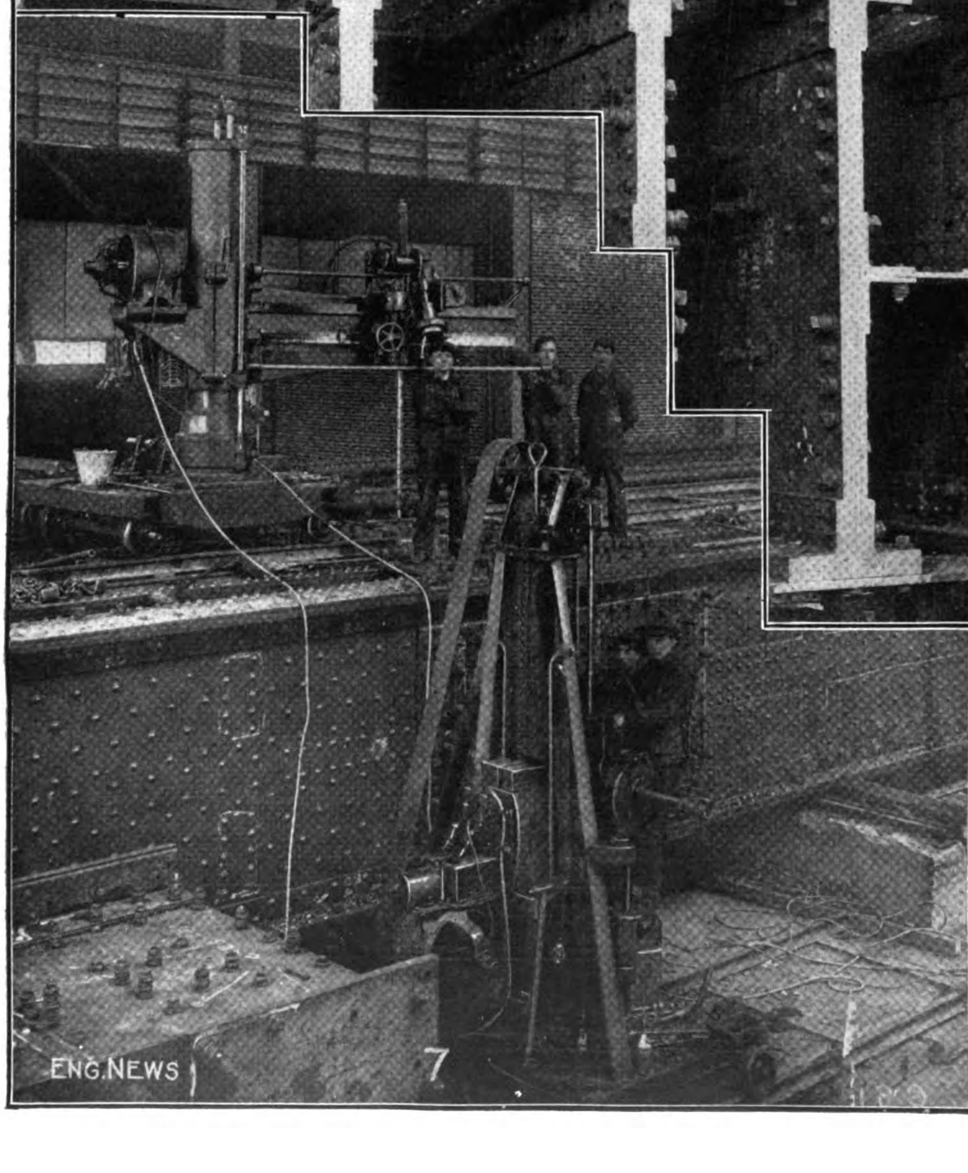


FIG. 5. REAMING AND DRILLING THE WEB OF A FLOOR-BEAM WITH THE PORTABLE DRILLS

FIG. 6. BORING THE PIN-HOLE IN GUSSET-PLATES OF A BOTTOM-CHORD SECTION

(Faced end of chord, for splice.)

FIG. 7. FITTING THE SPLICE OF BOTTOM CHORD

(Two drills are at work drilling the splice rivet holes.)

THE QUEBEC BRIDGE IN THE SHOP

handled by 6-ton jib-cranes traveling on runways 180 ft. long on both sides of the center row of columns. For riveting in restricted spaces, hand riveters and pneumatic buckers-up are used. Air is delivered to the riveters under a pressure of 80 lb. per sq.in. The large yoke machines have a capacity of 100 tons.

When very thick material is to be riveted, it is found that the best results are obtained by dipping the end of the rivet in water for a few seconds, thus hardening the end slightly. This causes the rivet to be thoroughly upset in the rivet-hole before the head is formed. All rivets over 5 in. long are tapered $\frac{1}{16}$ in. in 12 in., which makes practically a driving fit at the shank. The result is that, as a rule, remarkably tight rivets are obtained. When loose rivets are found, the heads are not sheared off, as is the usual practice, but are drilled off by means of a flat cutting-tool having a hollow core slightly smaller than the shank of the rivet. The upset portion of the head is consequently cut entirely away, leaving the shaft, which is then driven out. By this method, the hole is not distorted in any way.

Pins

About 1480 tons of pins are employed on the whole bridge. Their sizes vary from 8 in. to 30 in. In order to facilitate erection, double pins are used at the connections of all members having four webs, as well as for the top-chord eye-bars, each pin extending over half the width of the member. Each of the 30-in. pins weighs approximately 6 tons. Both nickel and carbon-steel pins are used depending upon the location. These pins are being forged and turned at the plant of the Bethlehem Steel Co. The specification requires that the finished pin shall not vary more than 0.001 in. from the true diameter.

Painting

The specification requires that all plates riveted together in the shop shall be given one coat of iron oxide and shall be allowed to dry before they are essembled. The shop coat is pure red lead to which is added 4 oz. lampblack to 30 lb. red lead, and mixed with pure boiled linseed oil to the proper consistency.

Each member is weighed individually before being stored, and the weight painted on in plain figures.

Mr. W. P. Ladd is works manager for the St. Lawrence Bridge Co. in charge of this work.

The Board of Engineers have their own staff of inspectors both in the mills and shop, and every stage of manufacture is very carefully followed.

Tests of the Electrolysis of Sewage at Toronto

By I. H. Nevitt*

During the fall of 1913, experiments were conducted at Toronto to determine the action of electrolysis on the sewage of that city. Although the results obtained were not such as to warrant the adoption of the process at Toronto, they are worthy of public record, especially in view of the limited available data on the electrolytic treatment of sewage.

The flume used at Toronto was copied from that in use at Oklahoma City,† with the exception that the width was made 12 in. instead of 22. This affected the number of plates in each grid, which had to be reduced to 13.

The plates were 10x24x$\frac{3}{16}$ in., bound on top by a copper strip riveted to the iron, and were spaced 5/8 in. apart. They were held together by four 1/2-in. rods, the bottom two of which were insulated from the plates by rubber tubing; the other two acted as conductors. The 10 grids were connected in parallel across two copper busbars, running along the sides of the flume. One of these bars had disconnecting switches which permitted two grids being thrown in at once.

Current was obtained from a Westinghouse motor-generator set designed for 250 amperes, at 125 volts. It was connected through a water rheostat to the terminals of the busbars. In the experiments, the amperage was kept at about 250 amperes, while the voltage at the generator was about 9. The voltage at the flume varied from 3 to 6.

The experiments were run at various rates of flow, and for different lengths of time, one being for 24 hr. at the rate of 120,000 U. S. gal. per day. The minimum rate was 84,000 U. S. gal.; the maximum, 156,000 U. S. gal.

The chlorinated sewage was pumped from the high-level intercepting sewer, after having passed through 1/2-in. screens. This sewer carries about four-fifths of the sewage of the city but does not contain very much trade-waste, although its reaction is slightly acid.

The discharge from the pump flowed through a baffled U-shaped forebay and on through the flume, and was measured at the discharge end of the flume over a weir without end contractions. A bypass weir was provided at the forebay to take care of the surplus sewage pumped.

In the first experiments, it was found that a surface film of sewage flowed through the flume without treatment. This was overcome by placing baffles between the grids, which reached about 2 in. below their top.

Samples of the raw sewage and of the effluent were taken about every five minutes and were sent to the laboratories of the Department of Health to be examined for reduction in solids and bacteria.

On starting an experiment, the bubbling action began slowly and gradually increased. In the course of a few minutes, a scum began to form on the surface of the sewage which got thicker as time passed. It was composed of a frothy mass containing a good deal of iron. Sludge also settled out in the bottom of the flume.

The flowing-through period was about five minutes. The effluent had a more or less rusty appearance, which was very noticeable if the flow stopped at all. The milky appearance which is described as typical of this method was not noticed until a trough was placed at the discharge end of the flume, and the effluent allowed to run about 30 ft. further. It then formed about 10 ft. below the weir.

The examination of the samples was for total solids, suspended solids and bacteria in the raw sewage and effluent. A further examination was made for solids after a sedimentation period of two and one-half hours. A table of typical results follows:

BACTERIAL REMOVAL PER C. C. AT 37° C. AFTER 24 HOURS

Dec. 1	Total bacteria agar	Red colonies bile salt agar
Raw sewage chlorinated	6,600	325
Electrolyzed	5,900	780
Electrolyzed 8 minutes	790	71
Dec. 3		
Raw sewage chlorinated	685,000	21,000
Electrolyzed	77,000	900

*Assistant Sewer Engineer, City Hall, Toronto, Ont.

†See "Eng. News," Mar. 21, 1912.

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Progress of Work on the New Quebec Bridge during the First Erection Season

By H. P. Borden*

During the past season substantial progress has been made toward the erection of the new Quebec Bridge. In spite of the fact that the actual start on the work of erecting the main trusses of the anchor arm was not made until the middle of July, 1914, over 80% of the north anchor arm, amounting to some 15,000 tons, has been entirely erected, and for the most part riveted.

During the winter of 1913-1914, the traveler for this work was erected on the north shore, just clear of the abutment. On May 18, 1914, the traveler was completed and moved out over the approach span, which had been put in place the season before.

Two Sets of Falsework—From a position over the north anchor pier a start was then made on the erection of the steel falsework extending between this pier and the north main pier. The erection of two systems of falsework was required at this point: (1) An inside false-

*Assistant to Chief Engineer, Board of Engineers, Quebec Bridge; Montreal, Que.

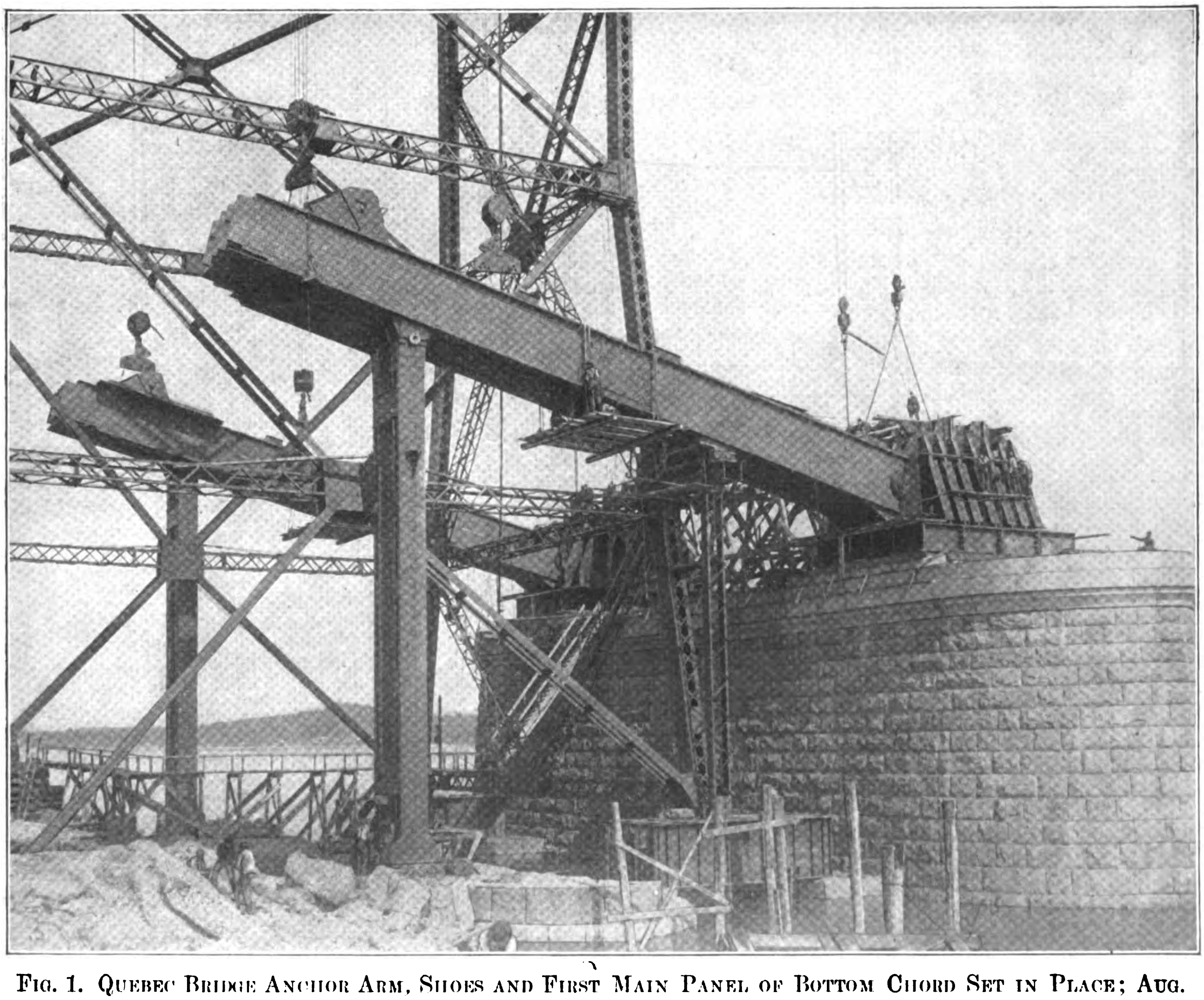
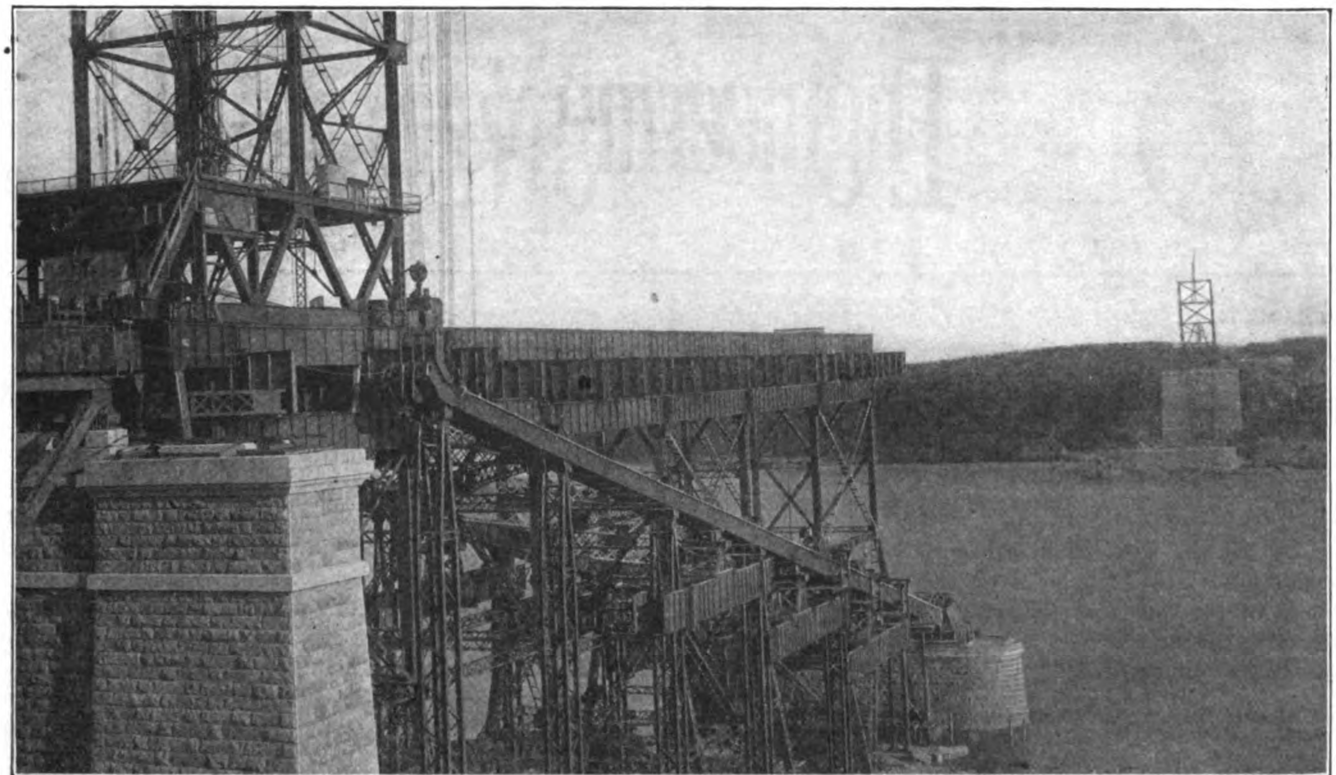
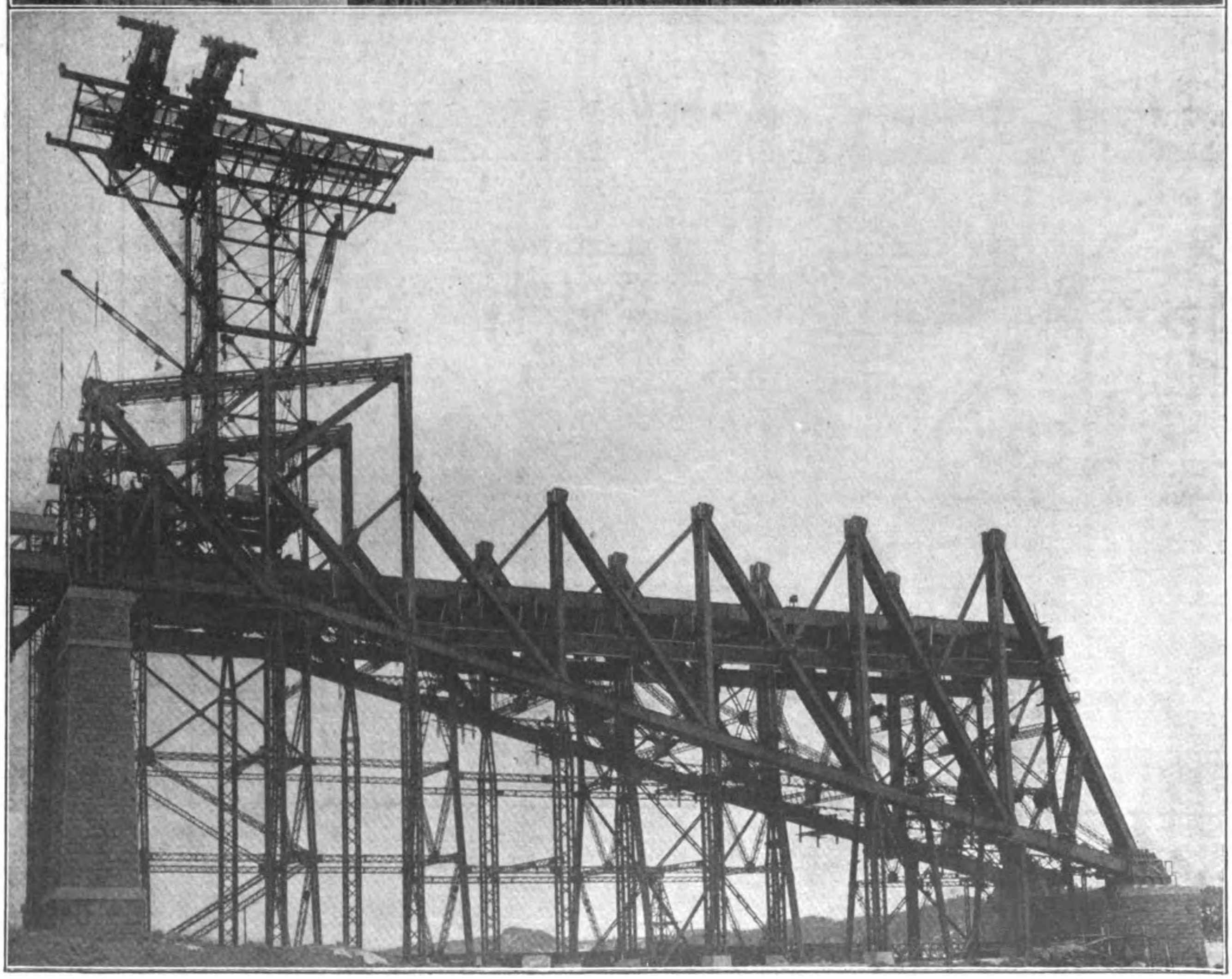


Fig. 1. Quebec Bridge Anchor Arm, Shoes and First Main Panel of Bottom Chord Set in Place; Aug. 4, 1914

(Note steel falsework for carrying the chord. Floor and traveler are carried by the independent inner falsework. Chord and shoe are connected by a 30-in. pin in a 45-in. bushing. The chord panel was erected in four pieces, spliced longitudinally and transversely; the transverse splice is just over the riveter staging.)





FIGS. 2 AND 3. QUEBEC BRIDGE ANCHOR ARM: BOTTOM CHORD COMPLETED (UPPER VIEW, SEPT. 3), AND LOWER HALF OF WEB SYSTEM ERECTED (LOWER VIEW, NOV. 17, 1914)

(The upper view shows girders and blocking supporting the chord between the panel-point falsework towers, for making the field splices.)

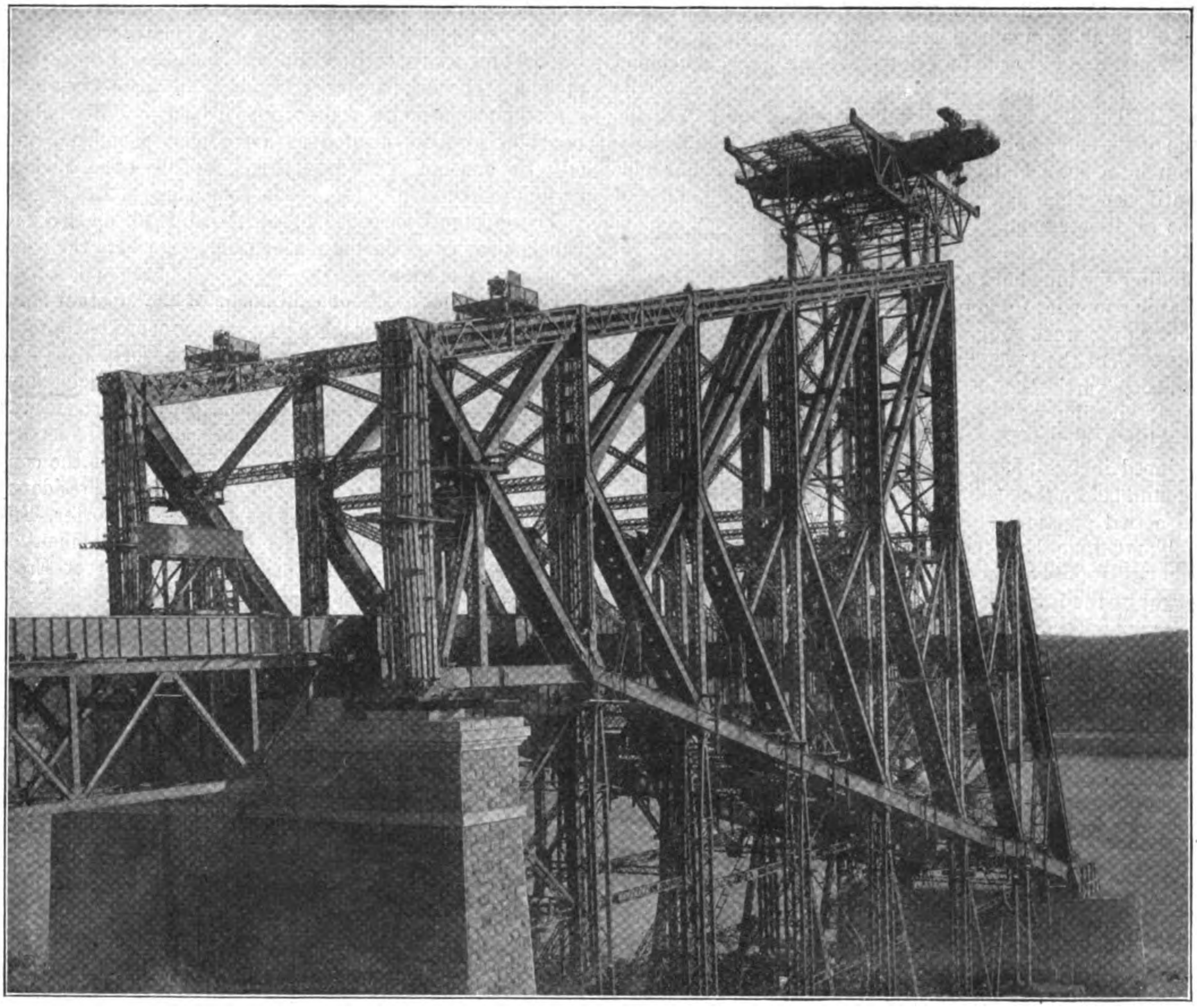


FIG. 4. THE QUEBEC BRIDGE AT THE CLOSE OF THE 1914 SEASON, DEC. 5

(In this view the double bank of eye-bars in the top chord, and their supporting trusses, are seen very clearly.)

work, which was required to support the *floor* of the bridge during erection and upon which the traveler and all erection equipment and plant were handled; (2) an outside falsework, entirely independent of the first, which supported the *trusses* of the anchor arm during erection.

The inside falsework consisted of seven bents, braced laterally and horizontally, the traveler moving ahead panel by panel as each bent was erected. These seven bents, covering a distance of 500 ft., were erected in practically two months, or at the rate of one bent a week.

On July 15, 1914, a start was made on the erection of the shoes on the north main pier. These shoes, weighing 400 tons each, had been completely erected in the shop before shipment and all holes reamed out either in place or to template, and all parts accurately matchmarked and stamped, so that the assembling at the site went ahead very quickly. By Aug. 1 were entirely assembled and sufficient rivets driven to allow a start to be made on the erection of the bottom chords.

PLACING THE BOTTOM CHORDS—According to the scheme of erection, the bottom chords as a whole were erected on the outside staging from main to anchor pier before any of the web members were erected in place. The average main panel length being 86 ft., it was necessary to erect these lower chord members in four pieces for each panel, there being a transverse as well as a longitudinal field splice (both vertical). A full panel of bottom chord near the shoe (Fig. 1) weighs approximately 400 tons per truss. By Sept. 28 these chords were erected, connected up with the bottom lateral system, and the web splices riveted. (Fig. 2 shows the work a few weeks earlier.) The traveler moved back toward the anchor pier as the work progressed.

ERECTING THE WEB MEMBERS—When the traveler had finished the erection of the bottom chord, it was again moved forward to the main pier and the erection of the lower half of the web members (up to the point where the diagonals and the verticals intersect) was started. These diagonals, also, on account of their weight, had to be erected in four pieces between main panel points, having a vertical as well as a longitudinal field splice. As their ends are pin-connected, the erection of this portion of the web system proceeded rapidly. Each diagonal was accurately trued up and all the rivets in the vertical web splices were driven for the connection before it was connected to the vertical. No difficulty whatever was met with in the erection of these members.

The lower half of this web system was fully erected back to the north anchor pier by Nov. 9, 1914 (see Fig. 3). The anchorage bars were then put in place in the anchor pier and connected up to the eye-bar heads, which had been left extending above the masonry at the foot of the well. These bars were carried up and connected to the top of the end compression diagonal, which is held in position by a special steel strut resting on the anchor pier until such time as it receives stress from the weight of the cantilever arm.

The start on the upper portion of the web system, including the top-chord eye-bars, was made on Nov. 12, the traveler moving forward panel by panel toward the main pier as the work progressed. This work went ahead even faster than the lower half of the web system, as the compression verticals were shipped in one length, and very little riveting was required.

Although the longer of the tension diagonals were shipped in two pieces, they were riveted together on the ground before they were erected in place, thus saving time in actual erection.

Top-Chord Eye-Bar Erection—The top chords are composed of two banks of eye-bars (see Fig. 4). On account of the impossibility of getting eye-bars of sufficient length to span a full panel, they are erected in two lengths, supported on the center by a small lattice truss, thus doing away with redundant members in the web system. The eye-bars are assembled in these trusses in the storage yard, the center pins driven, and the whole panel of eye-bars erected as a complete member; thus only the end pins need to be driven to fully erect a whole section of the chord.

The erection proceeded to panel point 10, or two full panels away from the main pier (Fig. 4), where the work ceased for the season on Dec. 5.

Owing to the excellent equipment of the erection traveler, the members for both trusses were erected simultaneously, which materially expedited the work.

Best Day's Work—The erection of one complete panel of the upper section of the web, with all pins driven, was the best single day's work during the season.

The St. Lawrence Bridge Co., of Montreal, is the contractor for this work. Phelps Johnson is President; G. H. Duggan, Chief Engineer; George F. Porter, Engineer of Construction; W. B. Fortune, Superintendent of Erection; S. P. Mitchell, Consulting Engineer of Erection.

Cleveland West Side Water-Supply Tunnel

SYNOPSIS—A 10-ft. diameter, 3-mi. tunnel now being driven under Lake Erie from existing crib 1½ mi. offshore to new crib nearly 4 mi. offshore. The tunnel is to be used for additional water-supply of the city. Tunnel being driven with boring tunnel machine in stiff clay. Lining of large concrete blocks placed with special block erector.

The city of Cleveland is now engaged in the construction of a three-mile water-supply tunnel reaching from an existing crib a mile and a half out in Lake Erie to a new crib nearly four miles off shore. While the tunnel work has only just been started, the details of the construction are so novel as to warrant a description of the methods employed.

The first water-supply tunnel for Cleveland was driven from the west side of the city to a crib a mile and a half from shore (see Fig. 1), known as Crib 4. This tunnel was 5 ft. in diameter and was later paralleled by a tunnel 7 ft. in diameter reaching the same crib. Crib 4 and the two tunnels connecting it were abandoned in 1904 for the city water service when the East Side tunnel, from the Kirtland St. pumping station, was completed, although they were continued for power stations requiring only a small supply. The East Side tunnel is five miles long and 9 ft. in diameter, and reaches on a diagonal line to Crib 3. The new tunnel, 10 ft. in inside diameter, is to start from Crib 4 and will extend 16,088 ft. outshore to Crib 5, now under construction. When the system is completed, water from the new intake will pass through the 10-ft. tunnel to Crib 4, whence it will be diverted through the 7-ft. tunnel and through the original tunnel (which is to be enlarged from 5 to 8 ft. in diameter); both of these tunnels come to shore near the new West Side filtration plant now nearing completion, and water from them will be passed through that plant before being delivered to the city mains.

The successive operations in the construction of the tunnel are: The construction of Crib 5; the clearing out of the old 5-ft. and 7-ft. tunnels; the reconstruction of the shafts at Crib 4 to permit the temporary shutting off of the 5-ft. and 7-ft. tunnels, and the construction of a working chamber below for the beginning of operations of driving the 10-ft. tunnel; simultaneous construction of

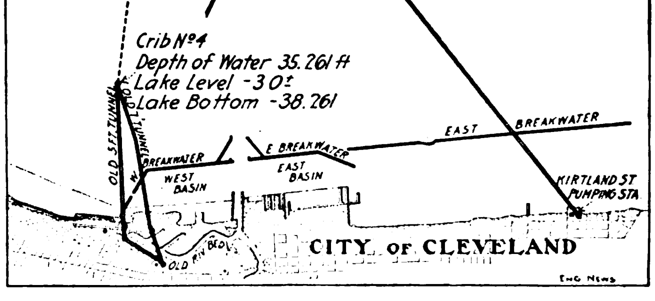


Fig. 1. Location of Water-Supply Tunnels under Lake Erie at Cleveland, Ohio

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The Erection Traveler, New Quebec Bridge

By H. P. Borden*

In the erection of large bridges, the rapidity and economy of operations are to a very large extent governed by the efficiency of the erection traveler employed on the work. These travelers may be either steel or wood, but on bridges of first importance from an engineering standpoint, steel travelers are of necessity employed.

Available Types of Traveler

On two of the most recent cantilever bridges to be erected, namely, the Blackwell's Island bridge over the East River and the Beaver Bridge over the Ohio, entirely different types of traveler were used. On the former an inside traveler was employed, running on the floor of the bridge and extending above and beyond the trusses on either side. On the Beaver Bridge a smaller traveler was employed which ran on the top chords of the bridge itself. Owing to the fact that the top chords of this bridge were sloping as well as curved, considerable adjustment was necessary at each movement of the traveler to keep it in a horizontal position.

In the case of the old Quebec Bridge two types were used. For the anchor and cantilever arms a high traveler was used, but operating outside of the trusses instead of inside. The tracks for this traveler were supported on special floorbeams attached to and extending beneath and beyond the bottom chords of the main trusses. This type has some advantages to its credit, the most important being the fact that the erection of lateral and sway bracing is not interfered with by the traveler.

For the erection of the smaller members at the end of the cantilever arm and those of the suspended span, a smaller top-chord traveler was used. By this means erection stresses were kept at a minimum when working toward the center of the span.

Fig. 1. The Quebec Bridge Traveler in Working Position

(The traveler track is built on the bridge floor, which rests on inside falsework independent of the truss falsework. Part of lower chord near pier is in place on the truss falsework.)

Inside Deck Traveler Adopted

In the case of the new Quebec Bridge the chief objection to a top-chord traveler is that the panel lengths of the cantilever and anchor arms vary, which would necessitate very elaborate details for distributing the reactions to the panel-points at each movement of the traveler. After thoroughly studying the question, it was decided that a traveler operating from the bridge floor would give better results, and this type was adopted. The difficulty in putting in the bracing between the trusses with this type of traveler was overcome in the anchor arm by the program of erection mapped out.

General Program of Erection—In accordance with this program, the entire floor of the anchor arm was erected first on special falsework and the traveler was moved over this floor to the main pier. The main shoes were then erected on the pier and both chords with their lateral system were supported on another set of special falsework from the main pier to the anchor pier. The traveler was then moved back to the main pier and the web members, with their sway bracing, were erected up to the point where the verticals and diagonals intersect, or to a point slightly above the line of the floor. This operation will be carried right back to the anchor pier, the traveler moving back as the work progresses.

The portal and the upper half of the diagonals and verticals, together with the

*Assistant to Chief Engineer, Board of Engineers, Quebec Bridge, Montreal, Que.

top-chord eyebars, are then erected, the traveler moving toward the river and keeping in advance of all erection work. By this means all bracing can be put in at the time the member itself is erected, all parts of the traveler being clear of this work.

When the traveler reaches the main pier it will naturally reverse the operation and erect the cantilever-arm material in front of it. The bracing in the panel being erected will therefore have to remain out until the traveler is moved one panel ahead. The bracing under the floor, however, can always be put in.

The traveler was erected on special supporting girders on shore, just clear of the abutment, the top of these girders being in line with the girders carrying the traveler tracks on the bridge.

Design of Traveler

The traveler structure consists of a main tower 37 ft. long by 54 ft. wide and 200 ft. high. At the foot the supporting trusses extend to the rear 52 ft. 9 in. in order to give an increased length of base for distributing the load on the trucks. The upper trusses have an

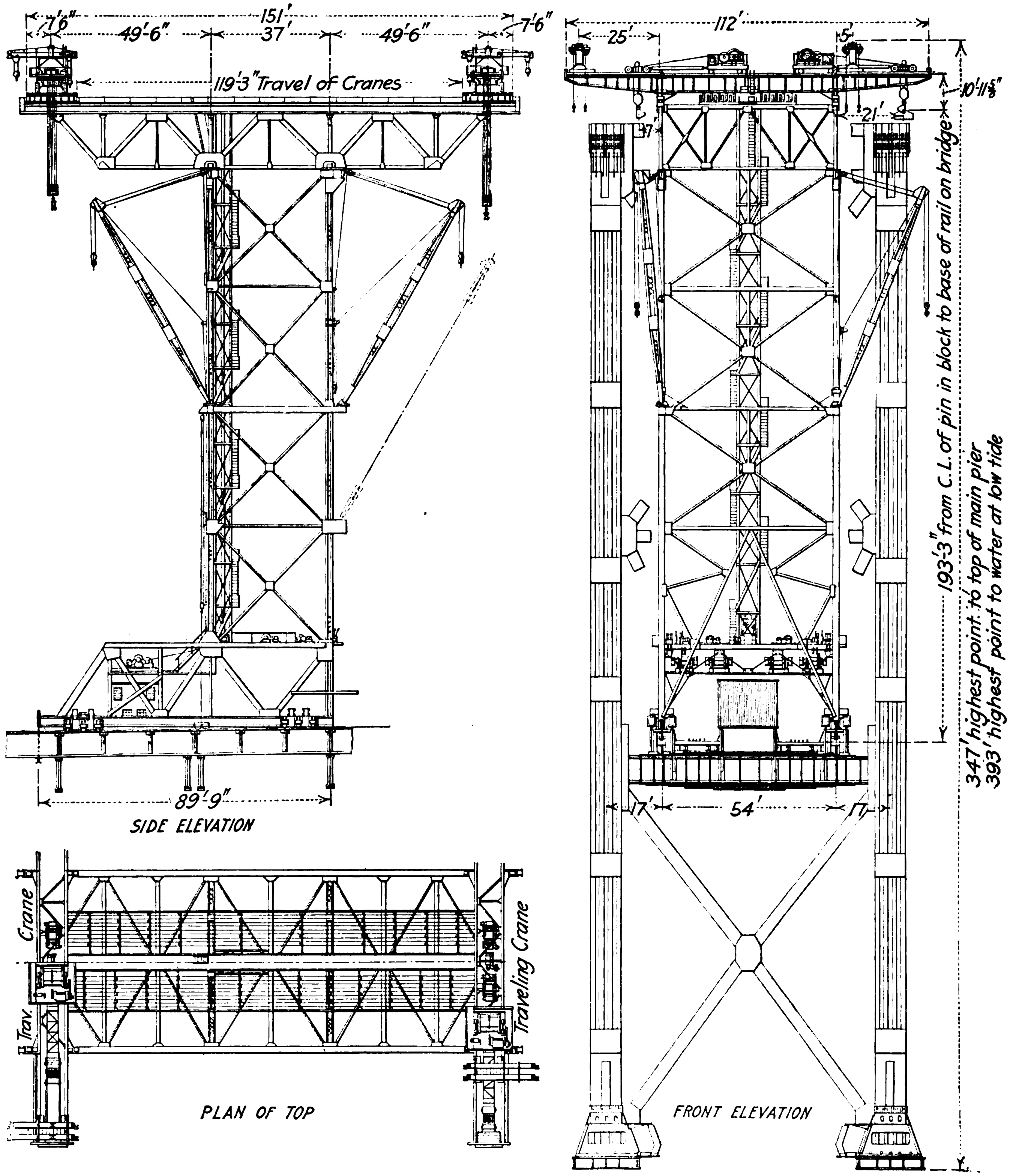

Fig. 2. Erection Traveler for the Quebec Bridge

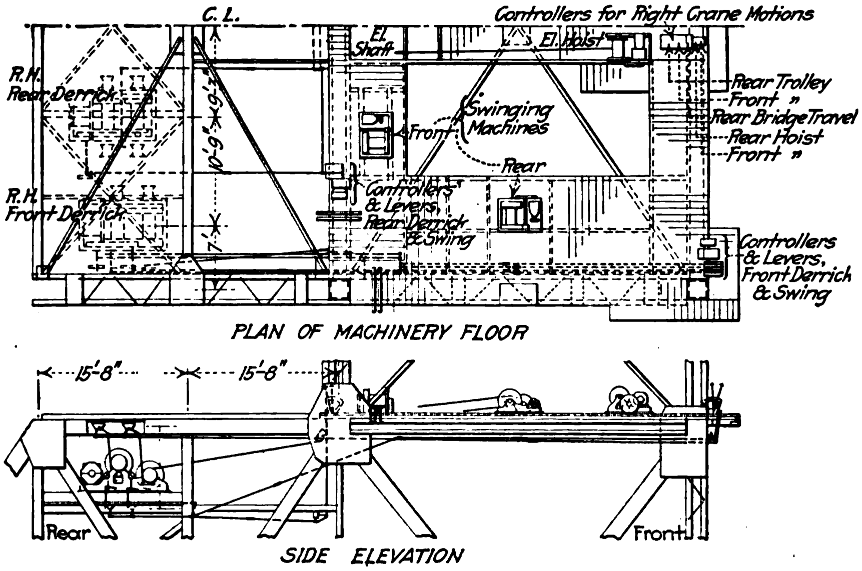

FIG. 3. PLAN OF MACHINERY PLATFORM

overhang fore and aft of 57 ft., upon which run the two heavy cranes which are used in handling the heavier pieces; the main members of these trusses are constructed of nickel-steel. The traveler moves on four tracks, one at each corner, each truck having six wheels distributed in sets of three over two rails placed 6 ft. 6 in. c. to c. In order to distribute the load on the three wheels equally over the rail, the reactions are taken by heavy spiral springs directly over the axle of each wheel. By this means any inequality of the track will be taken up by the springs and the load kept uniformly distributed. On the forward trucks the reactions from these six wheels are distributed by means of needle beams to one 6-in. pin through the webs of the bottom chord of the lower trusses. By

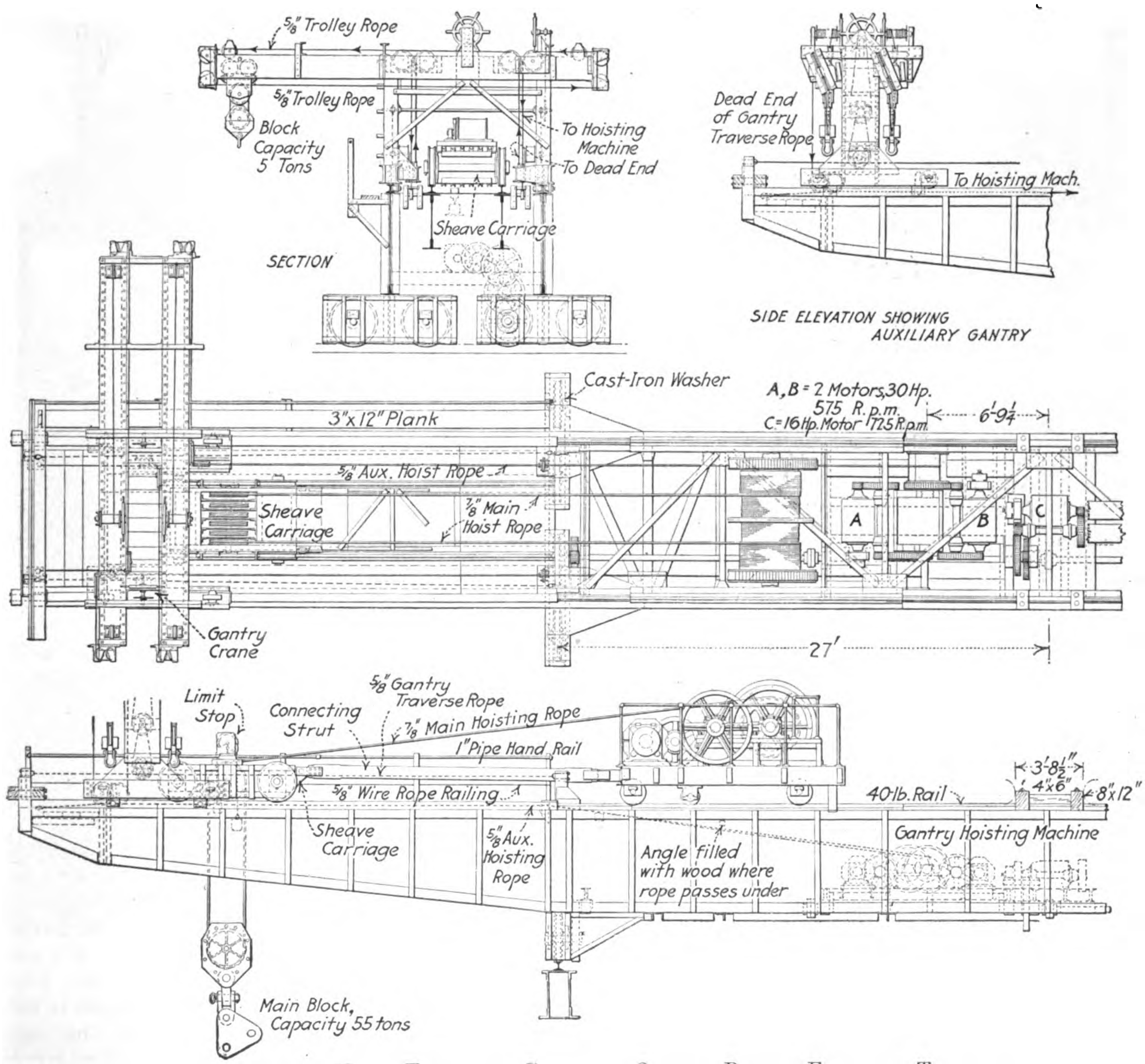

FIG. 4. MAIN-HOIST TRAVELING CRANE ON QUEBEC BRIDGE ERECTION TRAVELER
(Traveler carries two of these cranes.)

this means the entire reaction is concentrated at one point and is absolutely determinate.

As there is a grade on the bridge, some means had to be found for adjustment in order to keep the traveler level when operating. If this grade remained uniform it would be a simple matter, but on account of the deformations of the various members of the cantilever arm the grade is materially accentuated during the early stages of erection and it will also change at each panel. To take care of this, reactions from the rear of the traveler are transferred to the rear trucks by means of a vertical rod 5 in. in diameter, which takes the place of the pin in the forward trucks. When it is necessary to adjust the level of the traveler, the cranes on the top are moved ahead, thus materially decreasing the rear reaction, and then by tightening or releasing the nuts on these rods the necessary adjustment is made.

The traveler is carried on four lines of rails, two on each side of the bridge. The two inside rails are carried on the outer permanent track girders carrying the bridge floor. Outside of these girders is placed another row of special erection girders, 6½ ft. away, supported on falsework or special erection brackets and thoroughly braced to the permanent stringers.

The traveler is not designed to be self-propelling. When it is necessary to move ahead or back, a tackle is attached ahead to the track girders on each side and the cable is overhauled on the spools of the hoisting machines on the first floor. This has been found to work very satisfactorily, the traveler being moved at the rate of about 10 ft. per minute. When it has reached its new position the wheels are locked and shims placed under each corner. By this arrangement the cast-steel wheels of the traveler carry only the dead-load of the traveler itself and not the live-load under lifting operations. When the traveler is moved, the traveling cranes are always moved to the extreme rear of the top trusses, thus giving an almost equal distribution on the forward and rear trucks. When the cranes are engaged in lifting in their forward position, the rear of the traveler is held down by special anchorage tackle to the floor of the bridge.

Machinery and Hoisting Equipment

On the first floor of the traveler, about 18 ft. above the base of rail, are situated the four electric hoists which operate the four steel derrick booms at each corner of the traveler. These hoists are designed for a rope pull of 10,000 lb. with eight layers of ¾-in. wire cable on the drum. The mean rope speed is 150 ft. per min. The drums are 15 in. diameter and 31 in. long. Each hoist has four spools 12 in. diameter and 15 in. long, each spool being loose on the shaft and provided with sliding steel jaw clutch operated by hand lever and quadrant. The motor* is 51 hp. (half-hour rating, 55° rise of temperature), running at 295 r.p.m., with drum controller and brake.

The booms are 70 ft. long and can be extended to 90 ft. if desired. They operate from brackets at two different levels, according to the location of the members they are required to handle. The single ¾-in. wire-rope lead from the drum is converted into a four-part tackle at the boom, giving a hoisting capacity of 20 tons in any posi-

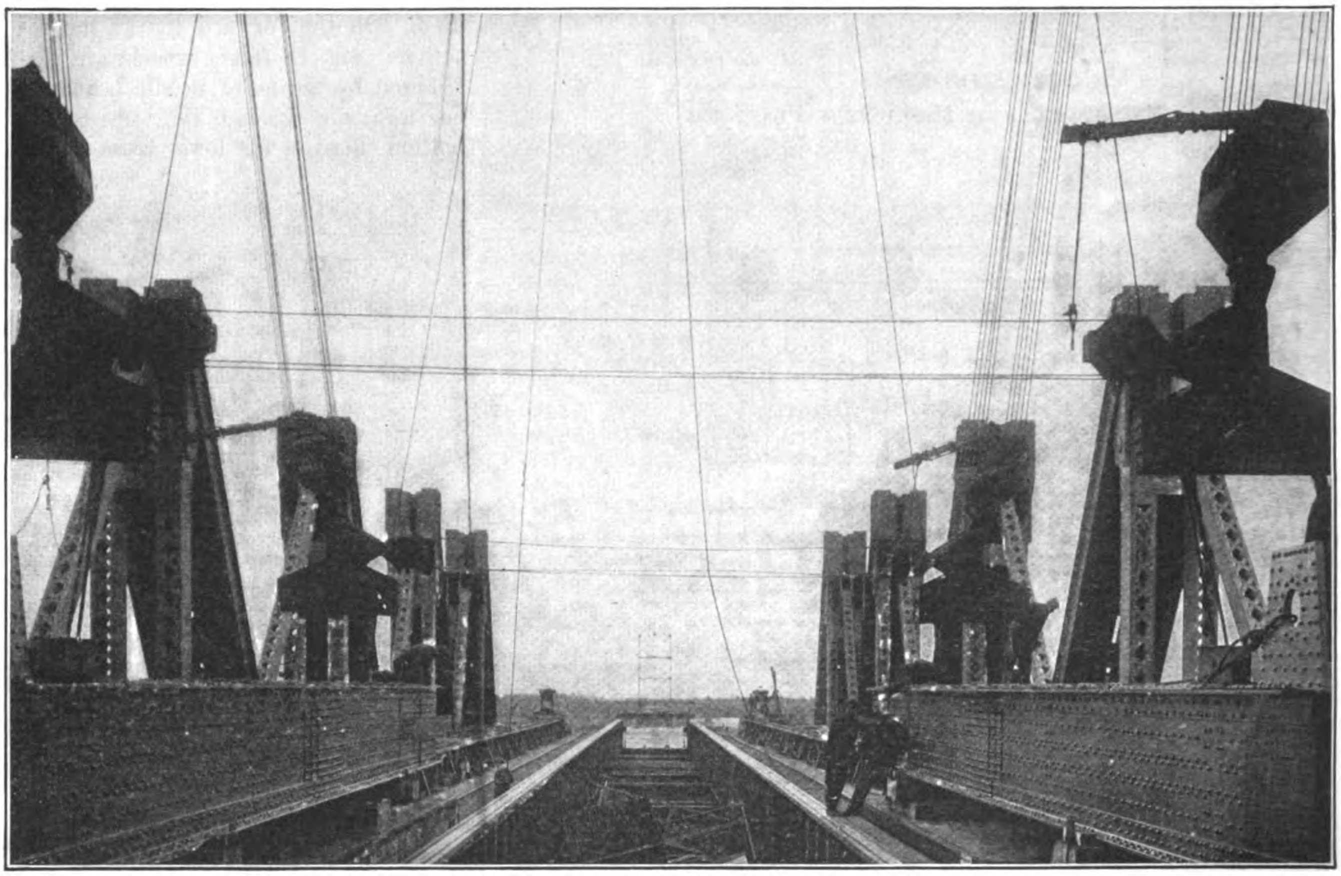

Fig. 5. Erecting the Base of the Quebec Bridge Traveler

*The electric supply is 220-volt direct current. The motors used on the traveler are all of General Electric make.

tion. These booms are used for handling smaller material, such as the lighter web members, bracing and floor material.

On the second floor, 25 ft. above base of rail, are situated the four machines required for swinging the booms.

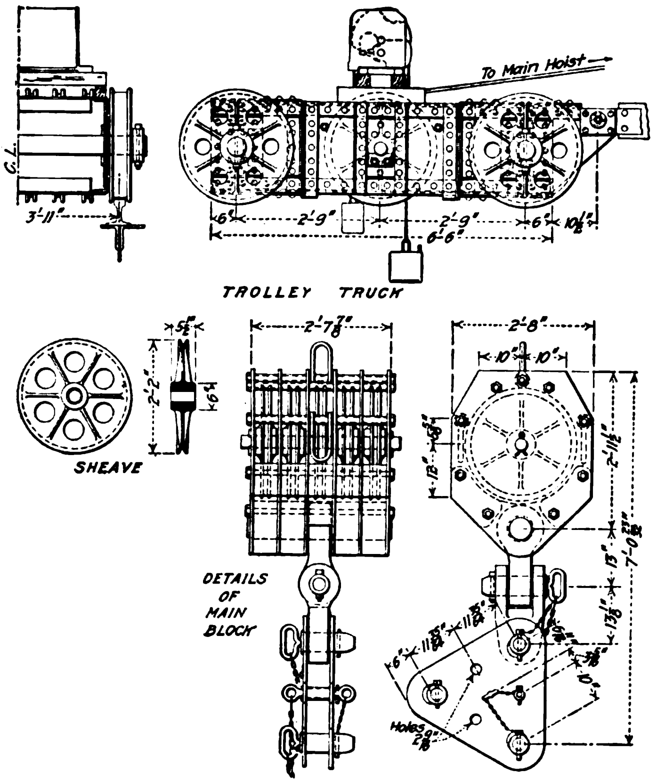

Fig. 6. Main-Hoist Sheave Details

This machine has a drum 13 in. in diameter and 14 in. long, which is operated by a 5-hp. motor having a capacity of 6000 lb. at 25 ft. per min. The motor is fitted with a solenoid brake, which is used principally to block the machine at night and when not in use.

Main Hoist Cranes

On the extreme top of the traveler are located the two traveling cranes which handle the heavier members of the bridge. These cranes, of the usual twin plate-girder type, are 112 ft. long; their girders are 5 ft. 5 in. deep at the center and are spaced 8½ ft. apart. They travel on the top chord of the upper trusses and are each carried on eight 30-in. wheels (in four 2-wheel trucks), but with only one driven wheel at each end of the crane. The crane is operated by a 16-hp. mill-type motor with a working speed of 725 r.p.m. and a capacity of 25 ft. per min. at full load. The cranes are required to climb a 1% grade with a maximum load of 500,000 lb.

Each crane is provided with two electric hoists, the working range of each hoist being necessarily outside the line of the upper trusses of the traveler. On account of the weight of the hoisting machine, it is desirable both from static and economical reasons that this machine be kept as near the center of the traveler tower as possible. As a consequence, the hoisting cable is carried over a sheave carriage placed some 20 ft. in front of the hoisting machine and connected to it by a strut. The hoisting motor is mill type, rated at 80 hp., 500 r.p.m. The controller system is of the magnetic-switch type, and is arranged for dynamic braking with a minimum lowering speed of 12 ft. per minute. The armature has an automatic disk brake capable of holding the maximum load. It is therefore impossible for the load to run away, as, in lowering, the motor works through a resistance greater than the load carried by the crane. The burning out of a fuse applies the full force of the electrical resistance, preventing all movement. In case of failure of the automatic brakes on the armature shaft or in case pinion should break or gear strip, an emergency hand brake is provided. This brake has a sufficient capacity to stop and hold the maximum load lifted. In order to guard against attempting to lift loads greater than those for which the traveler has been designed, special fuses are provided which will blow out when the load reaches a maximum.

Each machine has a lifting capacity of 120,000 lb. at a speed of 12 ft. per min., through a 10-part ⅞-in. wire cable, both ends of which are wound on the drum, thus obtaining a rope speed of 60 ft. per min. The maximum lift of these hoists is about 330 ft., requiring a total length of cable on each machine of approximately 3500 ft. Specially designed self-lubricating blocks are used which, with their shackle, are over 7 ft. high and weigh approximately 1½ tons each.

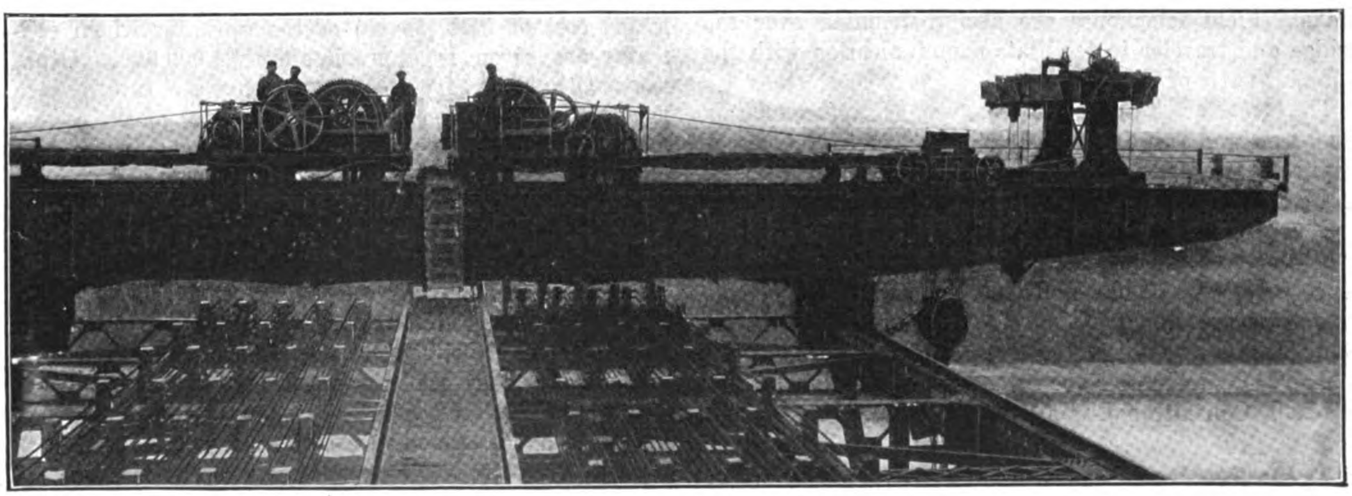

Fig. 7. Top of the Quebec Bridge Traveler, with Main-Hoist Traveling Crane

The carriage has a traversing motion of 14 ft., the motive power being a 5-hp. motor, 800 r.p.m. It is driven by means of a rack rail capable of holding the carriage against an end thrust of 30,000 lb. (due to fleeting loads), but the carriage is not required to move against this load.

When handling heavy members off the cars, duplicate members for each truss are lifted simultaneously, thus tending to maintain a more uniform state of equilibrium in the traveler and floor. As the cars are brought in under the traveler on tracks spaced 32 ft., and the hoists operate at a minimum distance of 68 ft. c. to c., the members require to be fleeted out. This is provided for by special fleeting tackle which holds the blocks together until the member is clear of the cars and any obstructions.

At the extreme end of each of the crane girders is an auxiliary gantry hoist of 5 tons capacity. The transverse and longitudinal movements of these gantries are operated by hand power, the hoisting power being, however, supplied by a 30-hp. motor situated at the top floor. These gantry hoists have a limited travel, being used principally for the handling of pins, riveting cages and other equipment in line with the main trusses beneath.

With the exception of the four hoisting engines operating the boom derricks, the electric controls of all machines are located on the second floor or bridge situated 25 ft. above base of rail, or about 175 ft. below the crane girders. One man is thus able to operate the two switchboards which control the travel of both crane girders and the traversing movements and hoisting operations of the four hoisting machines. The hoisting operations of the four auxiliary gantries are also controlled from this floor, by two other sets of control levers, situated one on each side of the floor. By this arrangement the operator is in close touch and has an unobstructed view of all work being done and can anticipate and promptly execute all orders given by the foreman in charge. One mechanician is always at the upper level to oil and keep a general supervision over the mechanical operations of the machines and is at hand to apply the emergency brakes should occasion arise.

Communication between the top of the traveler and the lower floors is effected by means of an electrically operated elevator as well as by wooden stairs encircling the elevator shaft. The motive power is supplied by a 15-hp. motor which gives a rope speed of 150 ft. per min. The controller is located in the car and is arranged for dynamic control. The motor is provided with a solenoid brake. Field telephones are also distributed over the bridge and traveler to facilitate communication with the different parts of the work.

The traveler, fully equipped, weighs approximately 1000 tons and cost in the neighborhood of $175,000. A duplicate traveler is being erected on the south shore, so that, next season, work can be proceeded with simultaneously on both sides of the river.

All the erection equipment was designed by the St. Lawrence Bridge Co., Ltd., Montreal, the contractors for the superstructure: Phelps Johnson, President; G. H. Duggan, Chief Engineer; George F. Porter, Engineer of Construction; W. B. Fortune, Superintendent of Erection; S. P. Mitchell, Consulting Engineer of Erection.

To Illustrate the Lasting Qualities of Cypress, when buried in the earth, the Southern Cypress Manufacturers' Association is sending out pieces of bored cypress logs laid for use as water pipe in New Orleans in 1798, or 116 years ago. One sample seen by a representative of this journal had an exterior diameter of about 12 in. and a 4-in. hole in the center of the heartwood. For about 1½ to 2 in. around the hole, the wood appeared to be as sound as when new. Outside this the wood showed a little decay and marked radial checks.

Factors in Grade Separation

In November, 1914, E. N. Layfield, Consulting Engineer, of Chicago, presented a report to the city and railway authorities of Houston, Tex., upon the local problem of grade-crossing elimination. Certain features of the report, dealing briefly with the Houston project and more extensively with the general subject of grade separation, are given below.

Grade Separation at Houston

There are 12 railways entering the city, having 50 miles of line, 300 miles of track and four stations within the city limits.

In the estimates of cost for Houston, the fill or embankment is taken at $1 per cu.yd. in place under the tracks. In the extensive work at Chicago a price of 50c. is generally used, but there is a vast supply of sand available from along the south shore of Lake Michigan, and this can be loaded and unloaded more cheaply than clay or ordinary earth filling. The cost of street excavation for lowering street grades is taken at 75c. per cu.yd., this work being expensive on account of interference with traffic and difficulty of drainage during the progress of the work. If this material could be used economically in the fill, the prices for both excavation and fill might be reduced. Concrete abutments and retaining walls are estimated at $10 per cu.yd., this price being arrived at after conference with the engineering departments of the city and the railways.

The bridge superstructures are assumed to be of steel at a cost of 4c. per lb. erected, to which is added the cost of covering with concrete and waterproofing. Under the conditions at Houston, reinforced-concrete structures would cost somewhat less and have advantages which are leading to their general use in work of this kind. The street paving is taken at $18 per lin.ft. of 65-ft. street, with brick paving on concrete base, concrete sidewalks and concrete combined curb and gutter. In most cases the damages to adjacent property would be small and would be covered by the 10% allowance for engineering and miscellaneous expense. Special allowances are made for such work as the elevation of the tracks through the Grand Central Station. The estimated cost of that portion of the work for which estimates are given is approximately $4,000,000. Other work within the city is suggested for the future, as a part of the general scheme, but on account of the remote date of its construction this is not included in the estimate.

Grade Separation Problems

Grades in Streets—In track elevation projects it is usual to allow depression of the streets at the crossings, so as to give a minimum height of raise of the tracks. This involves consideration of the approach grades on the streets. Easy grades mean longer and more expensive approaches and greater property damage, but, on the other hand, economy in cost must not be carried to the extreme of providing grades so steep as to affect the hauling of normal loads in the districts where heavy hauling is done. For horse traction, occasional short grades of 2 to 3% have very little effect on the loads, but steeper grades could be adopted if only modern motor-truck traction had to be considered.

The report states that in the Chicago track elevation

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Quebec Bridge Work in 1915

At the beginning of the 1915 working season the condition of the St. Lawrence Bridge Co.'s work on the Quebec Bridge was as follows: On the north side the anchor arm was only partly erected, nearly two panels not yet having been set; on the south side the approach spans were up, but nothing had been done on the anchor arm, not even the falsework being started.

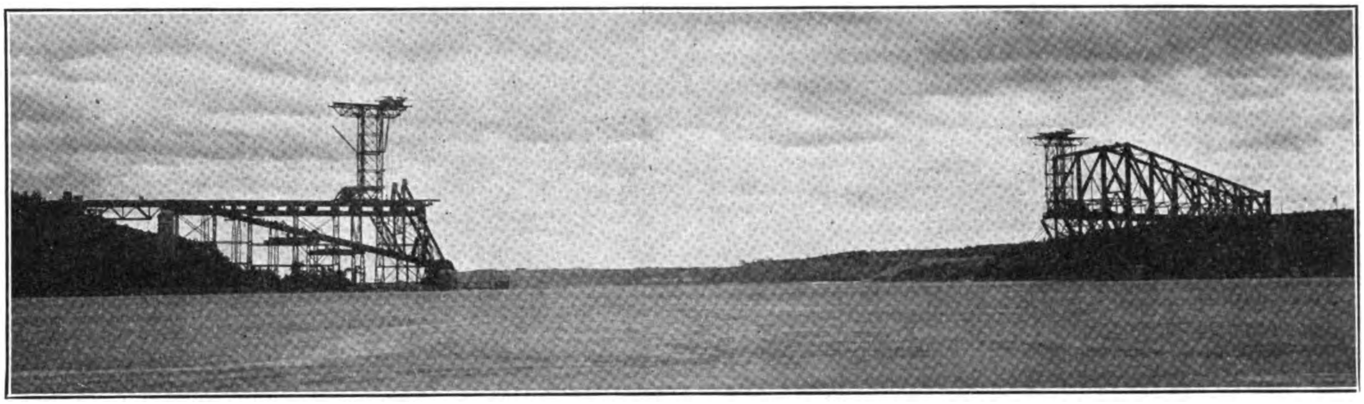


FIG. 1. PRESENT STAGE OF ERECTION OF QUEBEC BRIDGE

North cantilever arm three panels out; south anchor arm about half erected, two months ahead of last year's work on north anchor arm

By the latter part of August the north anchor arm had been completed and three panels of the cantilever arm erected, as Fig. 1 shows. On the south side the anchor arm falsework had been put up, the floor placed, the lower chords completed, and three panels of bottom posts and lower diagonals placed.

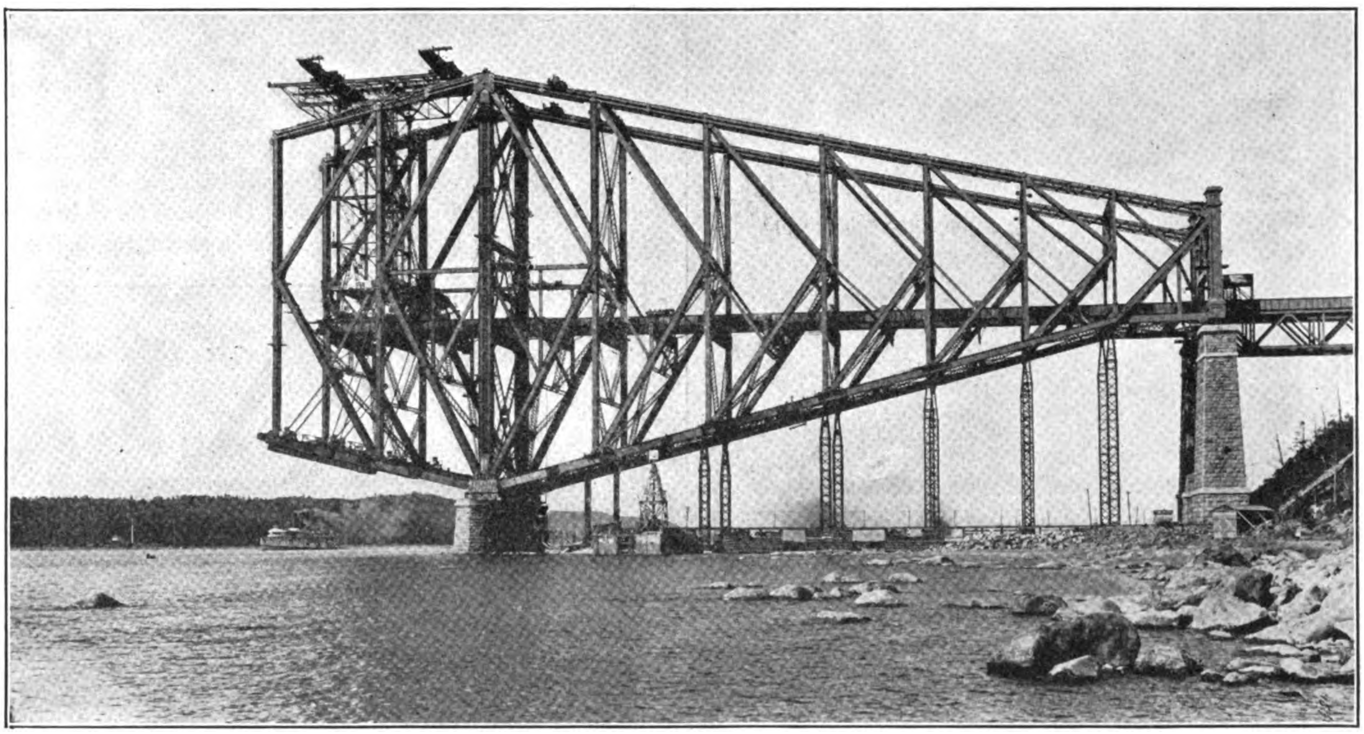


FIG. 2. NORTH CANTILEVER ARM OF QUEBEC BRIDGE ON AUG. 3, 1915

Progress on the north side is slightly ahead of schedule. Comparing this year's work on the south side with last year's work on the north, the 1915 work is eight weeks ahead.

The north cantilever should be completed this season, whereupon the traveler will be taken down, transferred three miles downstream to Sillery, and reërected (in part) for the construction of the suspended span. The foundations for the falsework on which the latter is to be erected are now going in. By the time the south cantilever is finished, in 1916, the suspended span ought to be fully erected and ready to be moved and hoisted into place in the bridge. Thus the bridge may be in service at the close of 1916—a season ahead of what was thought probable.

In speed of erection the contractors have made a notable record. A specially high performance was the erection on the south shore of 1,240,000 lb. (bottom chords and laterals) in one week in July, and a total of 10,040,000 lb. for the month, in addition to transferring the interior falsework and setting the main-post shoes.

Precision of Setting Members

This speed is attributed by the contractors partly to the general design and largely to the excellence of the shopwork. Every connection is fully assembled in the shop. The bottom chord members (each panel of the bottom chord is in four parts, being split longitudinally and divided once transversely between panel points) are machine-faced when fully assembled, and special attention has been paid to securing uniform temperature for all four webs. By the scheme of field erection for the chord, perfect bearing has been secured in every joint—not even a "feeler" being admitted.

The north shore main posts were set to a measured length only 0.002 ft. greater than the 292 ft. height between pin centers top and bottom, although of course such accuracy was accidental. This post was in 24 pieces and weighed 3,000,000 lb.

Particular effort has been made to have in the shop only work demanding accuracy. Rougher work, such as some of the falsework fabrication, was sublet.

Joining Large Compression Members

The bottom-chord erection scheme of G. F. Porter, Construction Engineer of the St. Lawrence Bridge Co., has worked perfectly. This is illustrated by the views

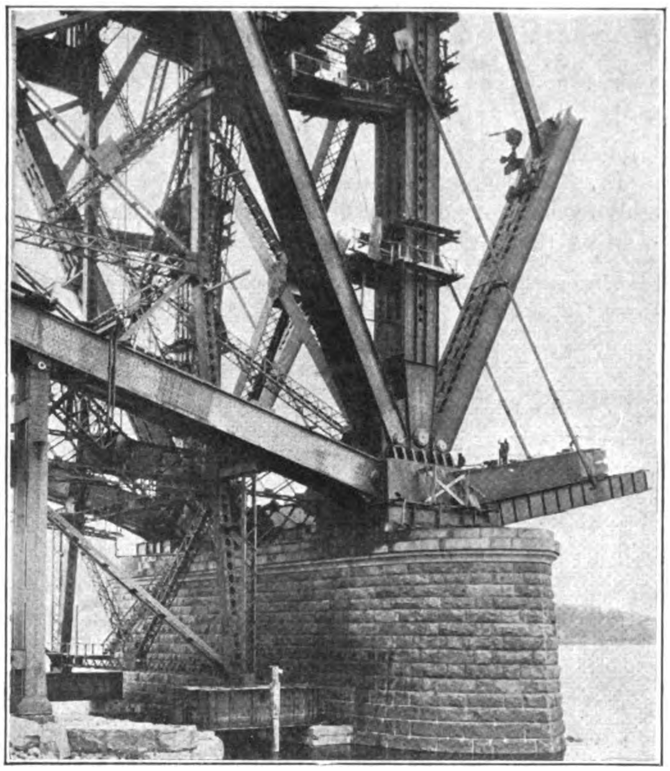


FIG. 3. ERECTING THE FOUR-PIECE MEMBERS OF THE BOTTOM CHORD ON FLYING BRIDGE

Figs. 3 and 5, taken on the north cantilever arm. A flying bridge projecting forward from the finished work is set just under the location of the chord to be set; it is hung by suspension rods from pins in temporary brackets attached to the web members. The quarter sections of the bottom-chord member are lowered into place on jacks, and the top flanges of the abutting sections are connected by temporary brackets, pins and links. Then lowering with the jacks throws the whole joint into full bearing, so that the bolting and riveting can be done.

Sunshine on the outer webs has frequently made a difference of a few thousandths of an inch in the bearing of these chord joints, but the riveting has been started if the difference did not exceed 0.012 in. In all cases the discrepancy disappeared as the riveting proceeded.

On the anchor-arm, instead of using the flying bridge, floor-beams are set longitudinally on the outer falsework to support the sections of the bottom chord. The whole anchor-arm chord is erected straight, from anchorage to main post, and after riveting all splices the falsework column jacks are lowered to give the proper camber

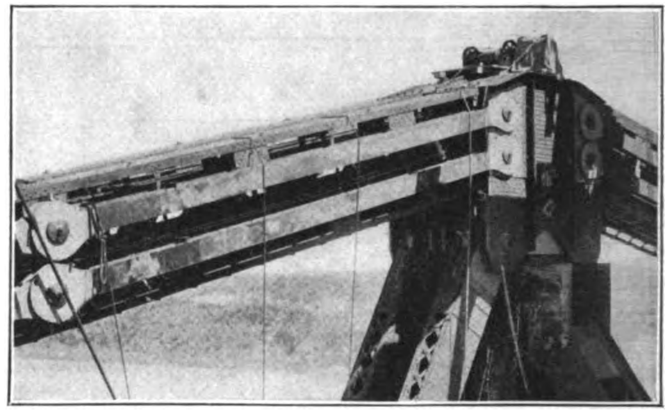


FIG. 4. THE DOUBLE EYE-BAR CHORD AT MAIN POST TOP

(about 6 in.). The cantilever arm is erected with the proper slight camber in each panel, and this flattens out under the increased load as the erection proceeds.

With three cantilever panels nearly completed, the effect of lifting on the anchor arm just begins to be appreciated. A rise of about $\frac{1}{16}$ in. is apparent at the first four panel points. This is not a lifting off the falsework, but merely a decrease of the compressive shortening of the falsework columns.

Moving the Traveler

Moving the traveler forward is accomplished as follows: The traveler cranes are run back, which eases up on the front truck springs. The erection brackets (bolted to the ends of the track girders to distribute

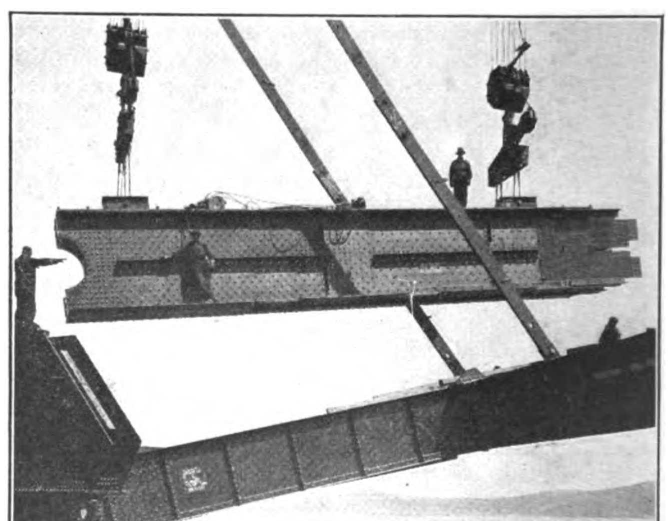
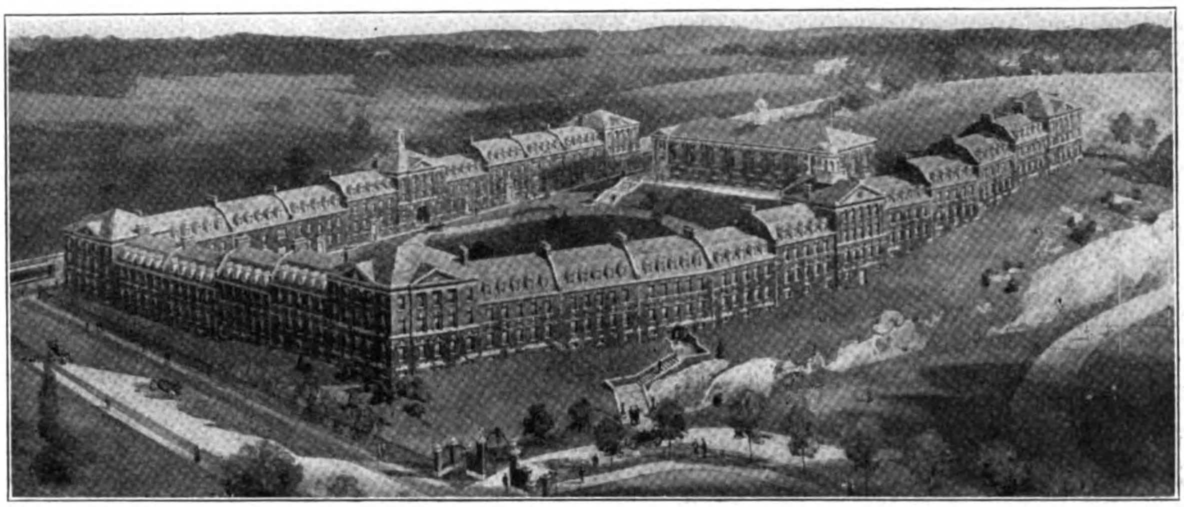


FIG. 5. SWINGING ONE OF THE BOTTOM CHORD PIECES INTO PLACE

the traveler load on two floor-beams—these being designed for only half the traveler load) are then taken off, which allows the setting of the new forward stringers track. The traveler is moved forward on the new length of track, the cranes being moved forward as the traveler advances so as to distribute the load over the front and rear trucks. When the front of the traveler comes above the next pair of floor-beams ahead, pedestals are mounted on these floor-beams, and the traveler shimmed up on the pedestals by manipulating the cranes, thus cutting the truck springs out of action and getting a rigid bearing.

New Buildings in Rensselaer Polytechnic Institute

Construction of a dining-hall and six initial units of a dormitory quadrangle for the Rensselaer Polytechnic Institute at Troy, N. Y., is now under way. As shown in the accompanying view, there will be 27 separate dormitory units, built about three sides of a court, east of the gymnasium and athletic field and adjacent to 15th St. and Ave. B of the City of Troy. Across the end of the court will be a dining-hall accommodating about 400 men. All the buildings will be of Harvard brick, with limestone trimmings, and of fireproof construction, the floors and partitions being of brick and concrete.



PROPOSED DORMITORIES AND DINING HALL; RENSSELAER POLYTECHNIC INSTITUTE, TROY, N. Y.

Each separate unit of the dormitory system will house twelve students, four on each floor. Two students will use one study and each man will have a separate bedroom. There will be a bathroom on each floor.

Six of the units will be larger than the other 21, the larger ones costing $16,000 apiece and the others $12,500.

A. T. White, of Brooklyn, N. Y., has given the sum of $50,000 for four units of the dormitory, and Capt. Robt. W. Hunt, of Chicago, $16,000 for one of the larger units, which will contain the main entrance to the group. Mrs. Russell Sage has given $100,000 for the dining-hall.

Air Analyzer for Determining Fineness of Cement

Experiments during the past three years at the United States Bureau of Standards have resulted in the development of an air-analyzer which is claimed to give results superior in accuracy to the sieve methods prescribed for the determination of cement fineness. For some time it has been felt that the variation in sieve results obtained under the standard methods is entirely too great for practical purposes. The mechanical construction of the sieves does not seem to be constant, and the method of operation can be varied to give different results.

The device is described in detail in "Technologic Paper No. 48," which may be obtained from the Bureau of Standards, Washington, D. C. It is described as follows:

This apparatus consists of a vertical brass pipe about 3 in. in diameter and 5 ft. long, at the lower end of which is attached a glass bulb in which the cement to be tested is placed. Air at constant pressure is blown into the cement through a glass tube or nozzle in the side of the bulb, and as the air can escape only through the vertical stack, it carries with it the cement dust, which is caught in a flannel hood surmounting the stack. The air flow in the stack is very uniform, and in a short time all the dust will be removed from the cement, leaving a granular residue in the glass bulb. This residue is weighed, and the amount of dust is determined by subtracting the weight of the residue from that of the original sample of cement.

Different grades are obtained by using different-sized nozzles, and thus a number of separations can be made in the very fine portion of the cement. With the aid of the microscope the size of the largest particles in any given separation can be readily determined, and in this manner the apparatus is standardized without reference to the size of the nozzles and other parts of the apparatus or the air pressure used.

It has been found that the cement "flour," that is, the portion of cement which contains no perceptible grit when rubbed between the fingers, consists of particles less than 0.0007 in. in diameter.

The apparatus may be used for separating and grading any hard-grained materials, such as ground quartz, emery and other abrasives. The air-analyzer in modified form is also capable of separating many other powders; for example, paint pigments, plasters, clays and similar materials.

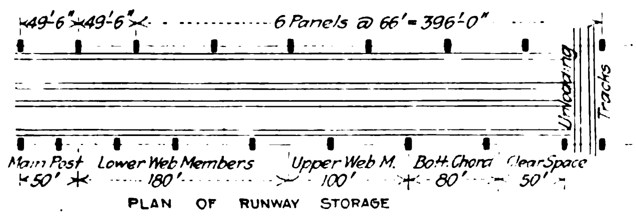
The Cement Trade of Australia has not interested the United States greatly, on account of the high freight rates compared with those from England and Scandinavia. The Interstate Commerce Commission has been investigating this industry in Australia, and finds that in 1913, the last year for which full statistics are available, 200,000 tons of cement were produced in Australia, and 124,100 tons were imported. The average wholesale price per cask of 400 lb. was $3.04 with duty paid, the duty during that period being 18c. per 112 lb. for cement from England, and 24c. per 112 lb. from other countries. The imports were distributed as follows: England, 31,000 tons; New Zealand, 2700; Germany, Austria and Belgium, 81,000; Scandinavia, 9000; United States, 400. The local production was divided among five factories—two in New South Wales, with a capacity of about 145,000 tons; two in Victoria, with a capacity of about 38,000 tons; one in South Australia, about 17,000 tons. According to Commercial Attaché William C. Downs, Melbourne, the effect of the war on the cement situation was to raise the price to $5.48 per cask; it then dropped to $4.38, varying thereafter from this figure to $4.87. Contracts for cement at the last figure have been placed in Scandinavia and Japan. Freight rates from Norway and Sweden are reported to be from $8.52 to $9.73 per ton.

Field and Office

Quebec-Bridge Camp and Yards

The arrangements for storing materials and housing workmen and engineers on a contract as great as the Quebec Bridge are always of interest. A single camp was built for the men working on both sides of the river, although, as it seems now, sufficient accommodations could have been secured without any camp whatever.

The arrangement of bunkhouses, dining hall, office, bungalows, etc., is shown in Fig. 3. Water-supply lines (2 to 4 in.) and sewers (6 to 10 in.) were run to the various buildings. The camp layout was shown in detail in the issue of Mar. 5, 1914, p. 499.



The crane runways for the storage yards are at right angles to the railroad tracks. A single crane is used on each. (This is quite different from the arrangement of the Phoenix company during the work on the old bridge—two cranes on a very long runway parallel to the railway.) The columns of the runway are staggered, Fig. 1, so that members longer than the crane span can be shifted along the runway by traversing the trolleys back and forth as the crane runs along.

Each yard was designed for storing all the members of an anchor arm at the same time, to provide for the chance of delay in erection. The principal piling arrangements are shown in the typical yard sections. To minimize rehandling, only those members which are to be used at about the same time are piled on one another. All members are placed with regard to their ultimate position on the bridge, so that they do not need turning end for end. At least 1 ft. clearance is maintained between adjacent pieces.

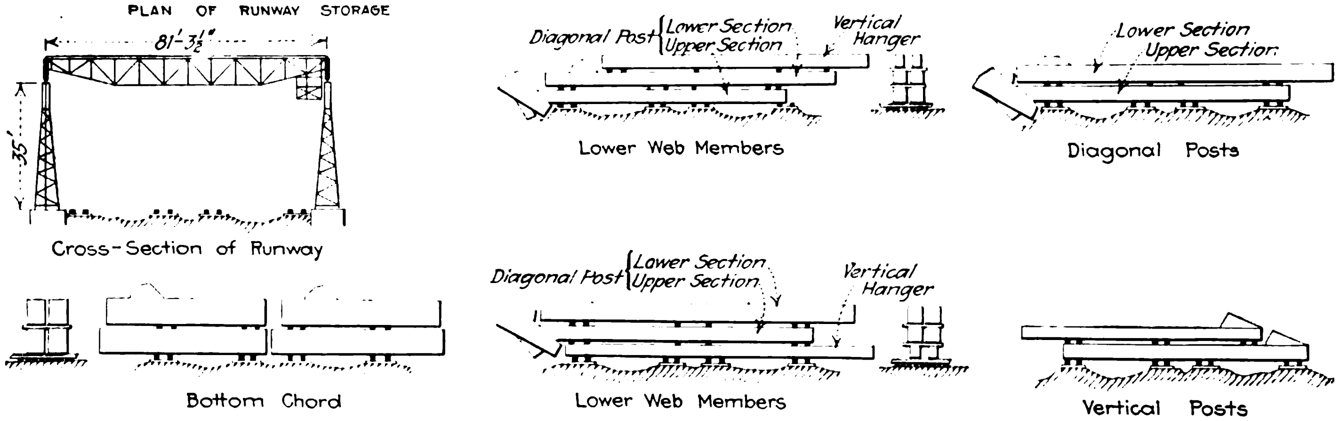


FIG. 1. STORAGE YARD AND TYPICAL PILING, NORTH SIDE, QUEBEC BRIDGE

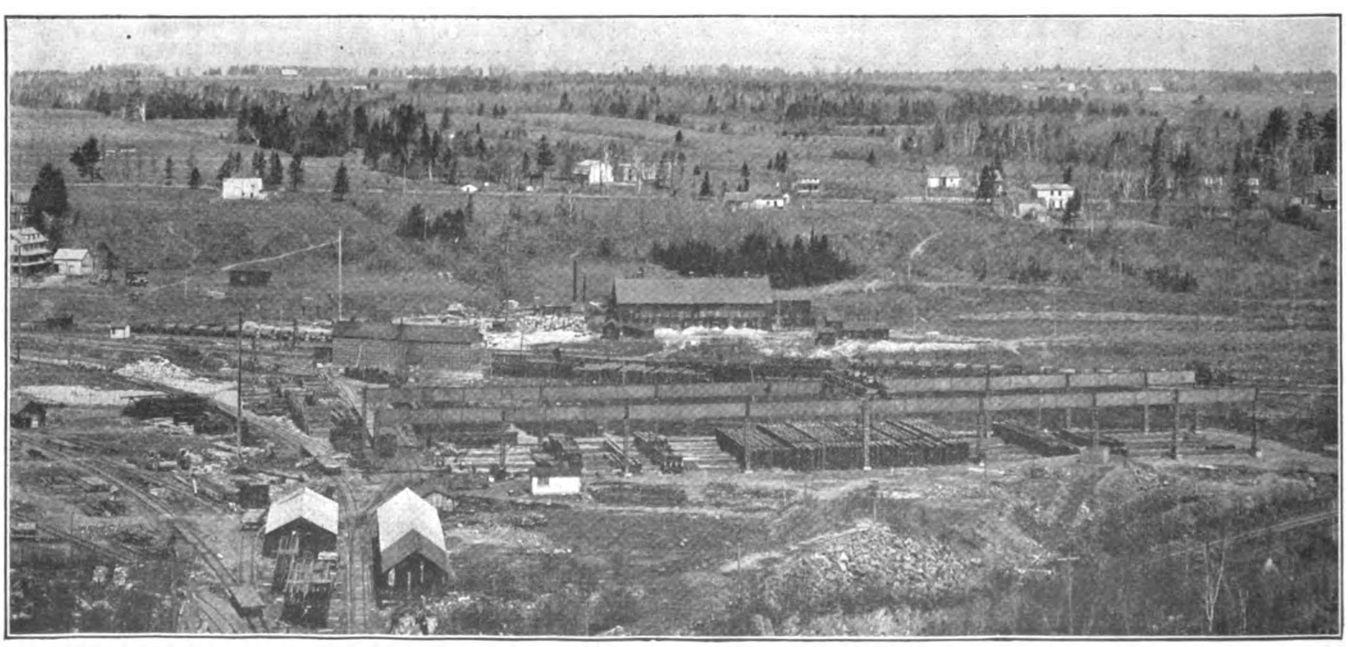


FIG. 2. NORTH SHORE MATERIAL YARD, QUEBEC BRIDGE

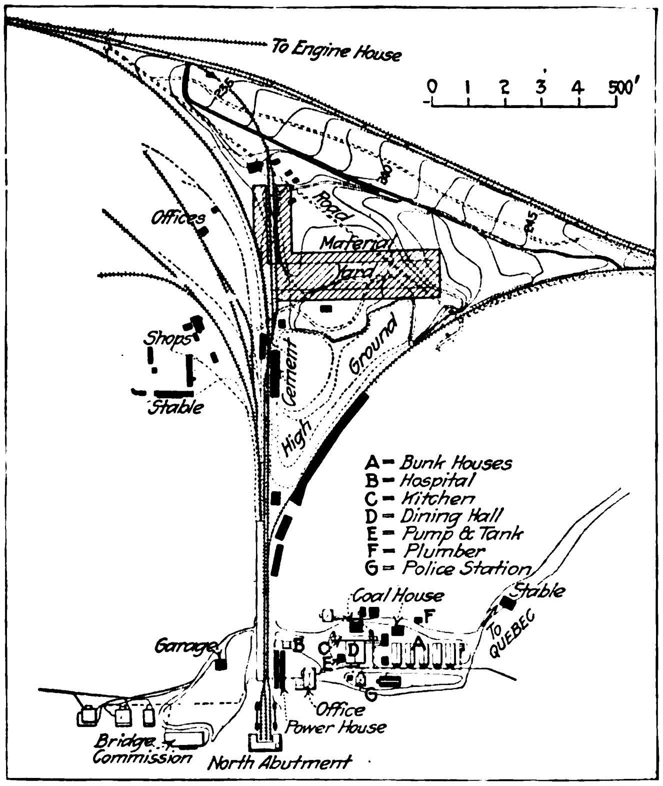


FIG. 3. CAMP AND NORTH SIDE YARD

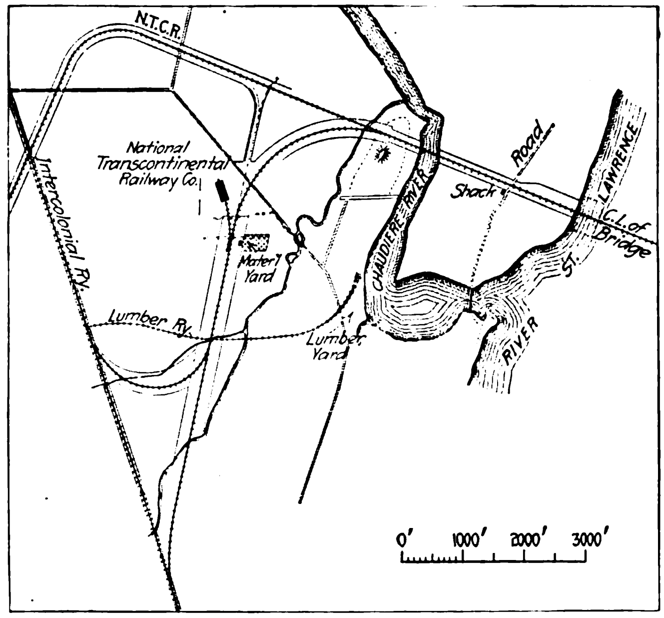


FIG. 4. SOUTH SIDE YARD, QUEBEC BRIDGE

Pieces with outstanding angles are generally handled with hooks, all others with wire-cable slings. For many parts special handling devices were built. The maximum amount stored at one time on the north side has been 10,000 tons and on the south side 12,000 tons.

The crane runway on the north side, patterned after one in the shops of the St. Lawrence Bridge Co., has box girders to carry the crane track. The runway for the south side has plate girders whose top flange is stiffened by a horizontal lattice-girder bracketed out from the girder web.

Each side of the river has its own electric-power station (or rather, substation; the two are fed from separate sources of energy) and compressor plant. The north-side station contains two 250-kw. motor-generator sets (one regular, one spare) to convert 2,200-volt alternating current to 220-volt direct current for the traveler. The primary supply is received at 22,000 volts. There are also four 100-hp. motor-driven air compressors, delivering air at 100 lb. gage pressure. The south-side plant is similar, but has only three 100-hp. compressors. The incoming current here is received at 11,000 volts.

Mirrors at Road Intersections

The use of mirrors for the protection of highway traffic at road intersections where the view is obstructed was noted in *Engineering News*, Aug. 5, and reference was made to an installation at Glencoe, Ill. The situation at this point is shown clearly by the accompanying plan and views, for which we are indebted to Robert A. Allton, Assistant Engineer of the Sanitary District of Chicago.

It will be seen that there is a jog in the street lines at the intersection, while the view is obstructed by the railway fills and bridge abutments.

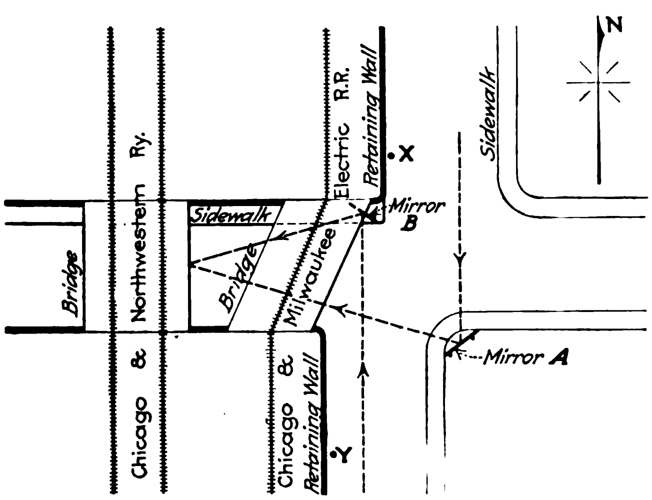


FIG. 1. PLAN OF ROAD CROSSING FITTED WITH MIRRORS, GLENCOE, ILL.

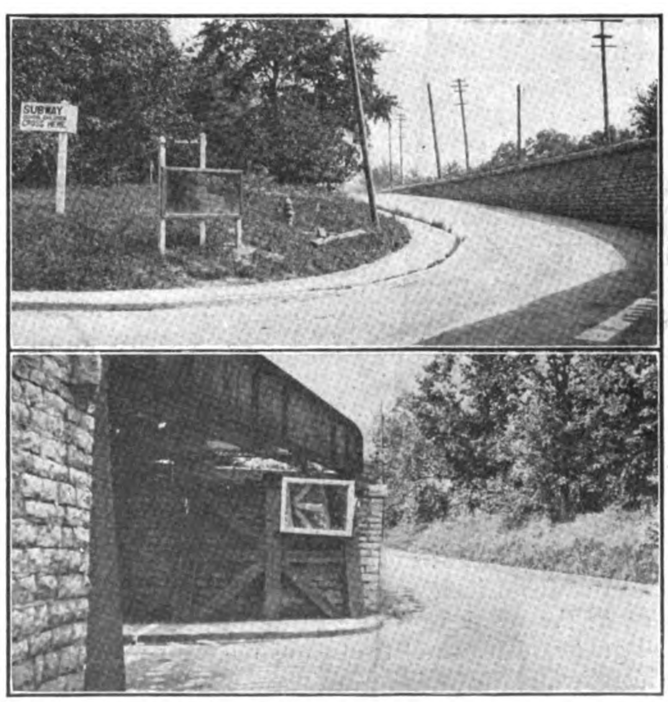


FIG. 2. MIRRORS AT ROAD CROSSING AT GLENCOE, ILL.

Upper view looks south from point X on the plan, showing mirror A. Lower view looks north from point Y, showing mirror B under the railway bridge

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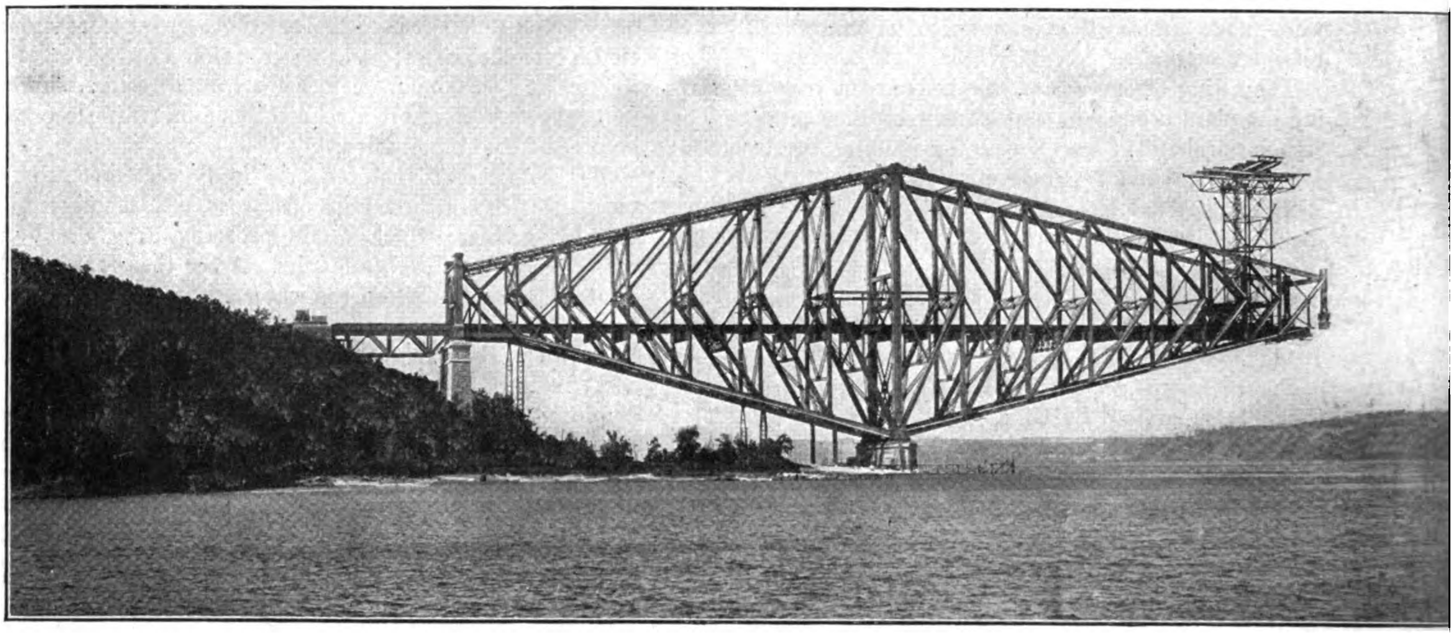


South Cantilever Arm of Quebec Bridge Completed

By A. J. Meyers*

SYNOPSIS—The south cantilever arm of the new Quebec Bridge was completed on July 28. About 13,000 tons of steel was placed in 92 working days—over a month ahead of schedule. Description of erection work and equipment.

During the season of 1915 the material of the new Quebec Bridge erected on the south shore of the St. Lawrence River consisted of the inside falsework carrying the anchor-arm floor and traveler tracks, the outside staging carrying the anchor-arm truss material and bracing, and all material of the south anchor arm complete, including the main post and the links at the top of the main post connecting the top chords of the cantilever and anchor arms. The total weight of steel erected by the one traveler on the south shore during the season of 1915 amounted to approximately 20,000 tons, including the weight of the falsework. At the close of the working period the traveler was standing over the south main pier, prepared to begin the erection of the south cantilever arm as soon as the working season for 1916 opened.

The erection of the 13,000 tons of steel in the south cantilever arm was started about Apr. 1, 1916, by which time the traveler machinery and tackle had been thoroughly overhauled. The members of the cantilever arm placed by the traveler while standing over the main pier are shown in heavy lines in Fig. 2.

Use of Flying Bridge

The first half of the flying erection bridge which was used, panel after panel, to support the bottom chords and laterals, as well as the main tension verticals, until these members were properly aligned, their splices riveted completely and the final connections to the upper truss members made, was first placed in position, connected up to its bearings on the main shoes and the front end tied back by yoking girders and links to the main post.

This erection bridge is illustrated in Fig. 5. It was made up of four plate girders of a length equal, approximately, to the longest main panel of the cantilever arms. These girders were placed in pairs under the chords of the trusses on each side of the bridge. The two girders forming a pair were spaced far enough apart to allow ample working space for the men on each side of the bottom-chord members as they were temporarily supported on the bridge. The girders of each pair were braced together by cross-girders which transferred the load of the truss members to the girders and also provided seats for the hydraulic jacks which were used for aligning and connecting these truss members. Each pair of girders had a complete bottom lateral system and a wooden floor supported along the bottom flange. This wooden floor between the girders completed a commodious working apartment for the assemblers and riveters who were working on the chords. The pairs of girders were braced together by two transverse plate girders which had a separate lateral system of their own. These transverse girders had to support the bottom-chord laterals until the end connections were made.

The erection bridge was thus a complete erection unit and was handled as such. It was supported, while in use, at the end nearest to the main pier by pins connecting to brackets which were bolted to the bottom flanges of the chord members already placed. The other end was held up by links at the center and end of the erection bridge. These links were attached to yoking girders which straddled the compression diagonals already erected overhead and in the panel to the rear.

The method of handling by the traveler cranes of the erection bridge while being moved forward into a new position is illustrated in Fig. 6. The links to the

*Chief Draftsman, Board of Engineers, Quebec Bridge, Montreal, P. Q.

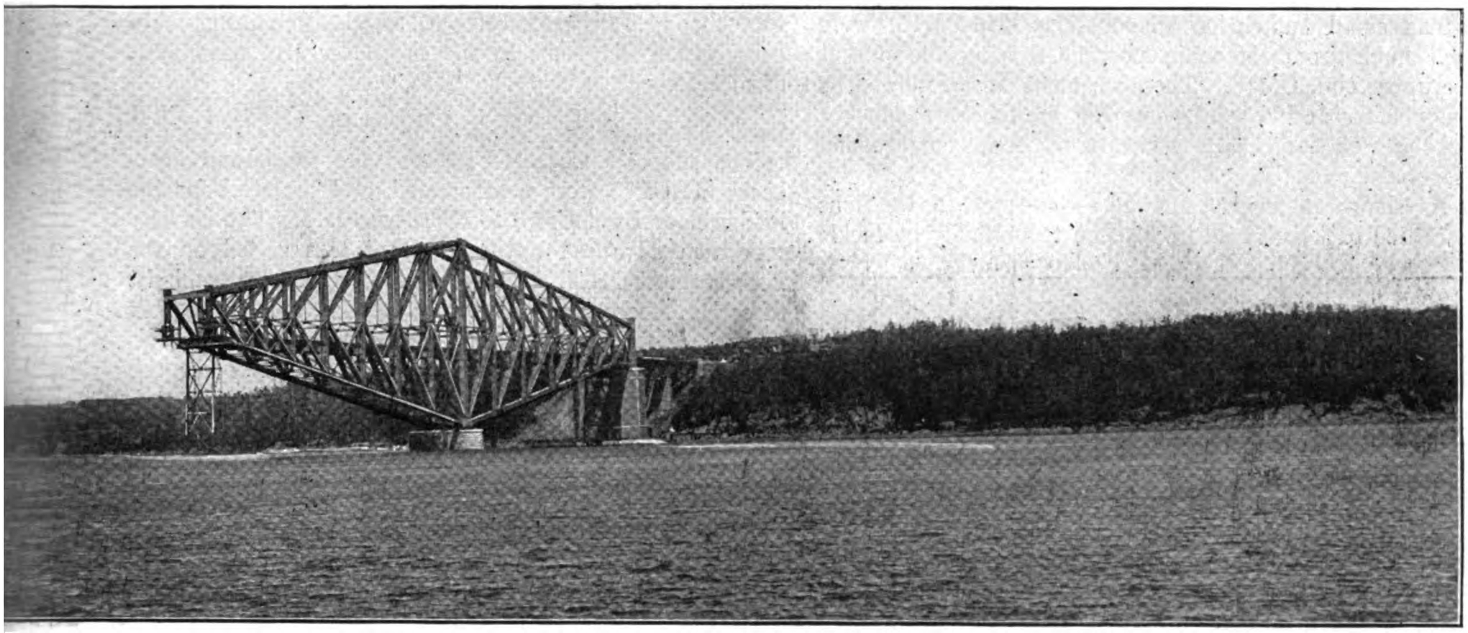


FIG. 1. THE QUEBEC BRIDGE ON JULY 31, 1916, WITH BOTH CANTILEVERS COMPLETED. ON THE SOUTH ARM, TO THE LEFT, THE ERECTION TRAVELER IS IN ITS LAST POSITION. THE NORTH ARM, TO RIGHT, ALREADY HAS IN PLACE HANGING TRUSS, FOR GUIDING THE SUSPENDED SPAN DURING ITS MOORING

forward end were first disconnected. The load of the erection bridge was then taken by the rear cranes, links L1 were disconnected, and the pins connecting the bridge to the chord brackets were removed. The bridge was then lowered and moved forward until stopped by the bottom laterals of the chord panel just completed. At this point the bridge was connected up and the load transferred to the hoisting tackle of the front cranes. The bridge was then moved forward and lifted up into its new position.

Cantilever Bottom Chords

In the first main panel CL16-CL14, as soon as the first section of the erection bridge was in position, the pair of 30-in. pins, with their 45-in. semicircular sleeves, for the cantilever arm bottom chords, and the 20-in. pins, with 30-in. sleeves, for the main compression diagonal CL16-CM14, were first placed on the shoes at CL16.

The first sections, CL16-CL15, of the cantilever-arm bottom chord were then placed on the erection bridge and carefully centered and aligned from the shoe. These members, built entirely of nickel steel, having a cross-sectional area of 1,630 sq.in., with outside dimensions of cross-section 84 in. deep by 124 in. wide, are the heaviest pieces in the cantilever arms. Each main-panel chord, made up of four vertical webs laced longitudinally in three horizontal planes, was divided into two half-panel sections, the members being fully spliced at this half-panel point in material and rivets, as well as being accurately faced to as nearly a perfect bearing as modern equipment and machinery could make possible. Each half-panel section was again divided vertically along its longitudinal center line. The member was shipped and handled in these sections, the heaviest of the sections weighing 160,000 lb. Each section was handled by means of specially designed and tested hitches bolted to the top flanges, two sets of hitches to each section. The sections were lifted from the cars in pairs, at the same time, one section for the east truss and the corresponding section for the west truss, all four of the 55-ton hoists of the two traveling cranes at the top of the traveler being used, one hoist of each crane to a section. The sections after leaving the cars were "fleeted" apart until

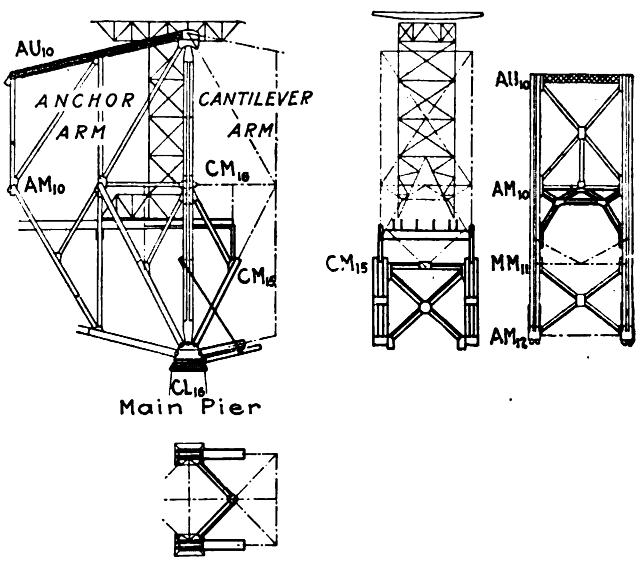


FIG. 2. CANTILEVER MEMBERS PLACED BY TRAVELER OVER MAIN PIER

they hung vertically over their positions on the erection bridge, when they were lowered into place.

The erection of the half-panel of the bottom chords was followed by the placing of a half-panel of bottom laterals, panel CL16-CL15, from the shoes to the middle intersection plate. The middle intersection plate and the lateral members themselves rested on the transverse girders of the erection bridge. The bottom lateral system is a double intersection system with a strut at each main-panel point. Each of the diagonals in a panel of the system is designed to take compression as well as tension. The lateral members are of nickel steel, the largest cross-sectional area being 98.7 sq.in., made up of eight 8x8x$\frac{13}{16}$-in. angles, built into a box section with a depth equal to that of the bottom chord and latticed in the horizontal and vertical planes.

The small subtension diagonals, CM16-CM15, were next hung from the main post by driving the pin at the upper end, CM16. These members were followed by the placing and careful centering on the shoe pins of the lower section, CL16-CM15, of the main compression diagonals.

As shown in Fig. 7, the upper ends of the sections were tied back and held in position temporarily by tackle to the main post. As soon as the two halves of the lower sections were placed, the pins connecting the subtension diagonal, CM16-CM15, to the main compression diagonal at CM15, were driven; and the sway bracing for these lower sections was placed in position. These compression diagonals are built members with an arrangement of cross-section similar to that of the bottom chord, but of much smaller area. They are spliced, shipped and handled in a manner similar to the bottom-chord members. The diagonals in the panel next to the main pier, where the weight of the member itself has practically no influence on the remainder of the bridge, were made of carbon steel. All of the remaining main compression diagonals in the cantilever arm were built of nickel steel.

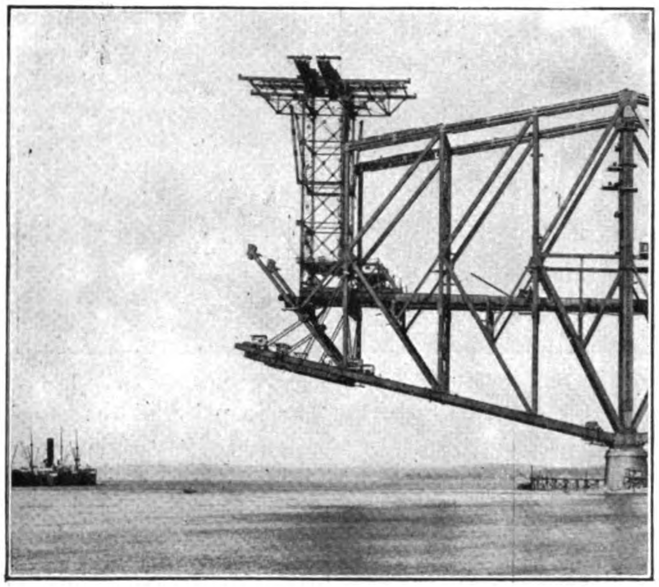
Bridge Floor-Beams

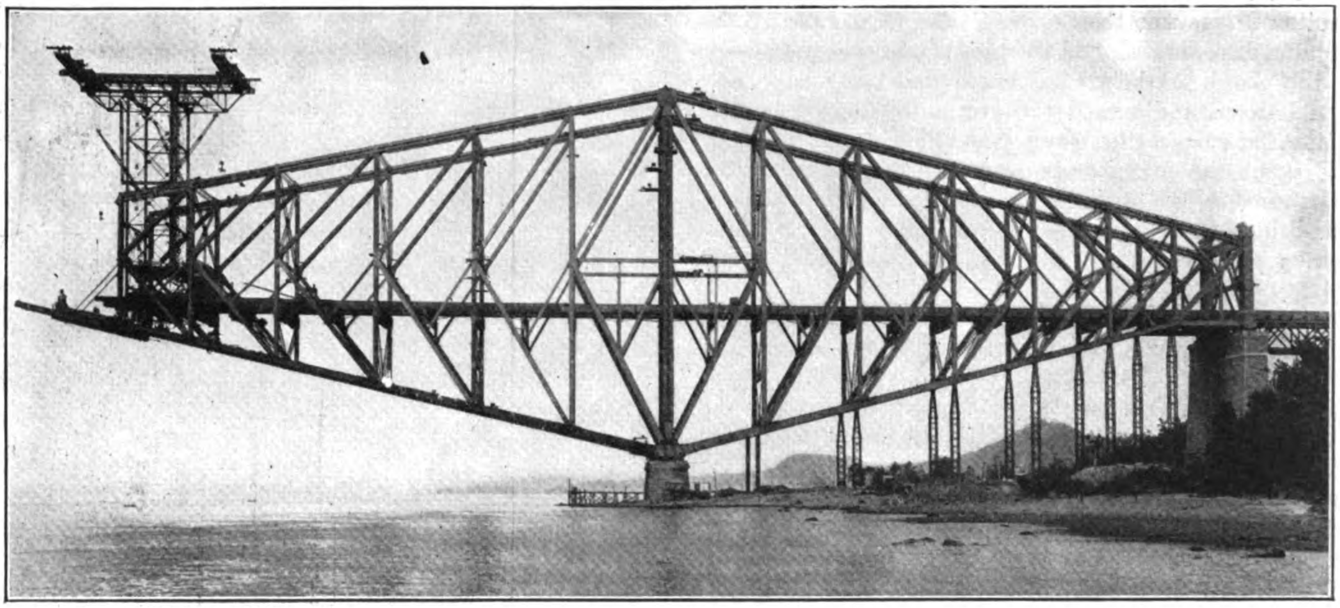
The subposts CM15-CF15, and the single-web floor-beams at CF15 were next erected, and the steel for the bridge floor from CF16 to CF15 was laid. The traveler was then advanced until the front legs rested on the floor-beam at CF15. The floor-beams of the bridge, built of carbon steel, are 88 ft. c. to c. of pins, and 10 ft. back to back of flange angles, the heaviest weighing approximately 120,000 lb. In the majority of cases for the cantilever arm they had additional material in the web and flange to take care of the shears and moments from the dead weight of the traveler and the lifted loads.

To lift these floor-beams from the cars and place them in their position in the bridge necessitated the design of a special lifting connection shown in Fig. 8. This lifting connection had two sets of cast vanadium-steel hooks which engaged the top flanges of the floor-beams. These hooks were pin-connected to hanger plates that were in turn riveted to a crosshead composed of two 10-in. 25-lb. channels. A specially designed swivel pin, with an eye at the upper end and threaded at the lower end to engage a cast-steel bearing nut, was used to transfer the load from the hoisting cables to the channel crosshead. A short cast-steel guide pin was bolted to the top of the floor-beams and served the purpose of insuring that the floor-beam would rotate about a vertical axis. The floor-beams were lifted from the cars by the two hoists of the front traveler crane, carried out until they cleared the steelwork of the traveler, then rotated through an angle of 90°, and lowered into position.

At each of these subpanel points, temporary bracing was put in from the floor-beam to the subpanel point, to transfer the wind load on the traveler to the bracing planes of the trusses.

The traveler, standing with its front legs resting on floor-beam CF15 (see Fig. 9), first erected the completing section of the erection bridge. The bottom-chord





FIGS. 3 AND 4. PROGRESS OF ERECTION OF SOUTH CANTILEVER

Fig. 3 (Above)—South cantilever on June 15, 1916. Fig. 4—The south cantilever on July 11, 1916. Erecting three panels in less than three weeks is a fair sample of the speed of progress on the work. In Fig. 3 the bottom chords, temporarily supported by the erection bridge, are tied back by links and yoking girders to the compression diagonals half-panel sections CL15-CL14 were then placed on the erection bridge, jacked into line, and the splice at CL15 made. Throughout the work all splices and riveted connections in the truss members were practically completely riveted panel by panel as the traveler advanced.

Erection of Main Diagonals

The upper sections, CM15-CM14, of the main compression diagonals, were next erected and the splices at CM15 riveted up, the CM14 end of the member being tied back by tackle to the main posts, as illustrated in Fig. 7. The erection of the sway bracing below the bridge floor for the upper sections of these diagonals, the upper portions, CF15-N15, of the subposts and the subchord member CM16-CM14, followed immediately, the pins at CM16 and CM14 connecting the subchord at these points being driven.

The main-panel point CM14 is typical of all the main-panel middle points or "K" joints of the cantilever arm, except that it has an extra connection for the subchord member CM16-CM14, which appears only in the first main panel of the cantilever arm. Generally there are four main truss members and one subtension diagonal connecting at these joints, all of them being pin-connected with the exception of the main compression diagonals. The main compression diagonals have the eight large main gussets riveted into their ends. These eight gusset plates are cut so that each and every gusset engages the pins of all the other main members connecting at the joint.

The main tension diagonals, CU16-CM14, are 150 ft. 6⅛ in. c. to c. of end connecting pins. They are built up of four webs, the webs being connected and riveted together in pairs by means of lattice bars and tie-plates. The pairs are then connected by means of spacer tie-plates. Each pair of webs was shipped to the bridge site in three sections, making up the total length of the completed member.

Before erection, these sections were assembled on the floor of the bridge between the traveler and the bridge trusses, and their field splices completely riveted up. Each pair of webs was then hoisted into position separately by the traveler. The inner pair was first placed and the pins at CU16 and CM14 driven. Any adjustment necessary to center the pin holes was made by means of tackle leading from the panel point CM14 to the main post. Immediately after placing the inner section, the outer section of the member was erected and its pins driven. The pin holes at the lower end of the member were slotted ¼ in. on the side remote from the bearing surface in order to facilitate the driving of the pins. The main tension diagonals in the first main panel of the cantilever arm are of carbon steel; in all the other panels they are of nickel steel. The pins

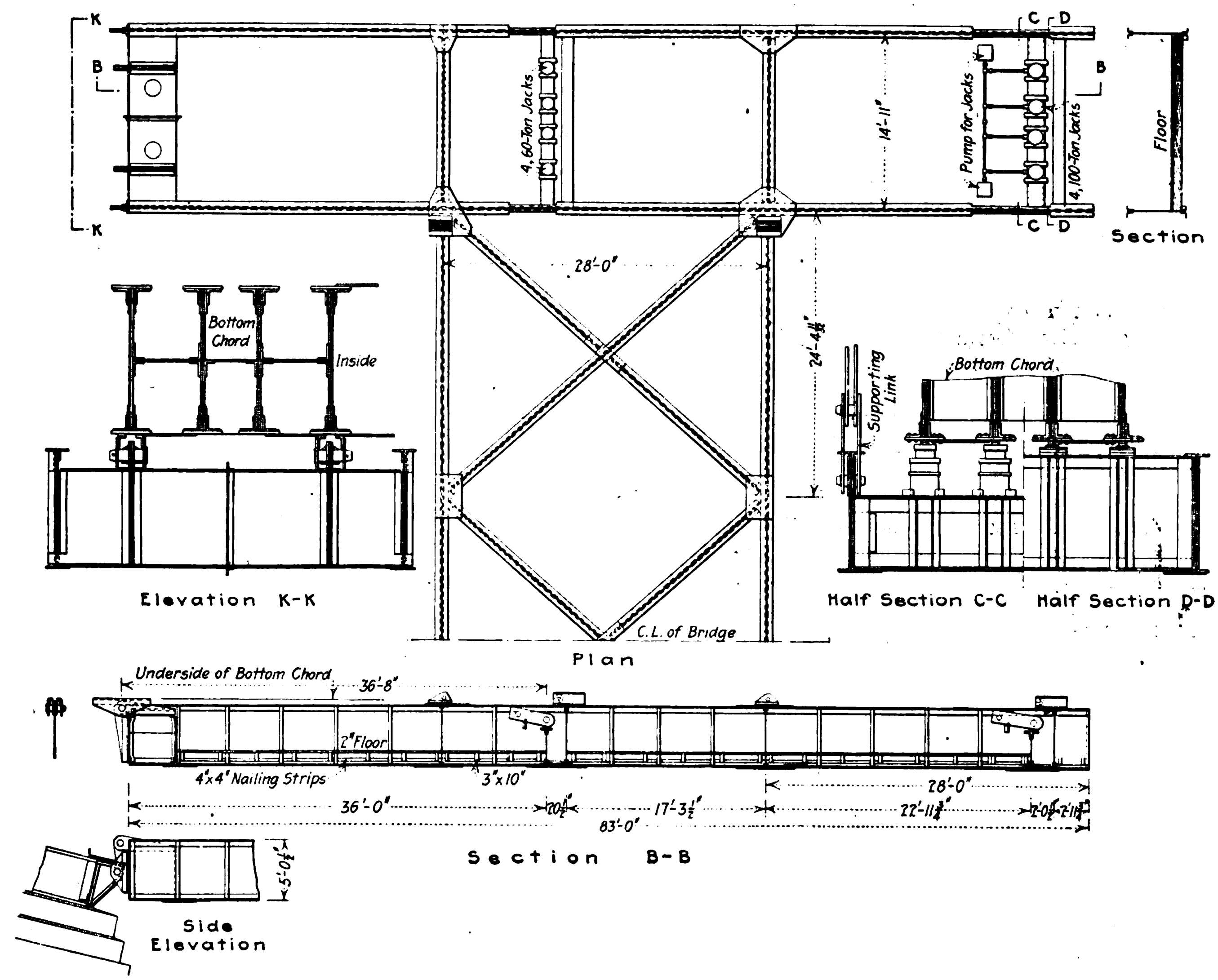


FIG. 5. ERECTION BRIDGE FOR QUEBEC BOTTOM CHORDS, LATERALS AND TENSION VERTICALS

throughout the bridge for each main-member connection are in half lengths, each half engaging two webs of the member and two gussets of the connection. This method of pin connection enabled the pins to be driven in all cases with remarkable ease.

Tension Verticals and Top Chords

The splice of the bottom chord at CL15 being completely riveted, the inside halves of the tension verticals, CM14-CL14, were next erected and the pins at CL14 driven. The bottom chords were then jacked up from the erection bridge until the pin holes at CM14 were in true line. The pins were then driven. The operation was then repeated with the inner halves of the tension verticals. These tension verticals are similar in built-up construction to the main tension diagonals previously described, and are shipped and handled in a similar manner. The upper pin holes were also slotted to allow the last driven pins to be easily entered.

The half-panel of bottom laterals, CL15-CL14, together with the bottom lateral strut at CL14, was next placed and the sway bracing from the floor-beam at CF14 to the bottom-chord panel point CL14 for the tension vertical CL14-CM14, was erected. The erection of the floor-beam at CF14 and the laying of the track floor from CF15 to CF14 followed immediately. The sway bracing for the tension verticals throughout the cantilever and anchor arms transfers a main-panel load of wind shear, due to wind on the floor and train, from the plane of the floor bracing to the plane of the bottom lateral bracing; also the cross-wind reaction on the front of the traveler during erection travels to the bottom laterals by the same path.

The compression verticals, CM14-CU14, which were the next truss members to be placed into position, were spliced transversely and longitudinally in a manner similar to the bottom chords and main compression diagonals, and the same methods were used for shipping, handling and hoisting into position in the bridge. All these compression verticals in the cantilever and anchor arms were of similar construction. They were built up of four plate-girder webs, spaced the same distance center to center of webs as the compression diagonals and bottom chords. They were similarly latticed in three longitudinal planes, and rested on pins in half pin holes at the main middle or "K" joints of the trusses. At this joint they were temporarily supported, as shown in Fig. 7, during erection by means of tension anchor bolts on the one side of the connection. These anchor bolts engaged brackets on the compression verticals and reaction brackets on the gusset plates of the main "K" joint. The anchor bolts were thrown into tension by means of the tackle which, as shown in Fig. 7, attached to the upper end of the member, and by means of which the member was tied back to and supported by the truss material already erected.

The erection of the truss material for the first main panel, C16-C15, was completed by the placing of the top chord eye-bars, CU16-CU14, with their supporting trusses (see Fig. 9), and driving the eight pins at CU16, connecting the eye-bars to the link at the top of the main post.

Each main-panel top chord in the cantilever and anchor arms is composed of two lines of eye-bars, spaced 3 ft. 6 in. c. to c. vertically, the one line above the other.

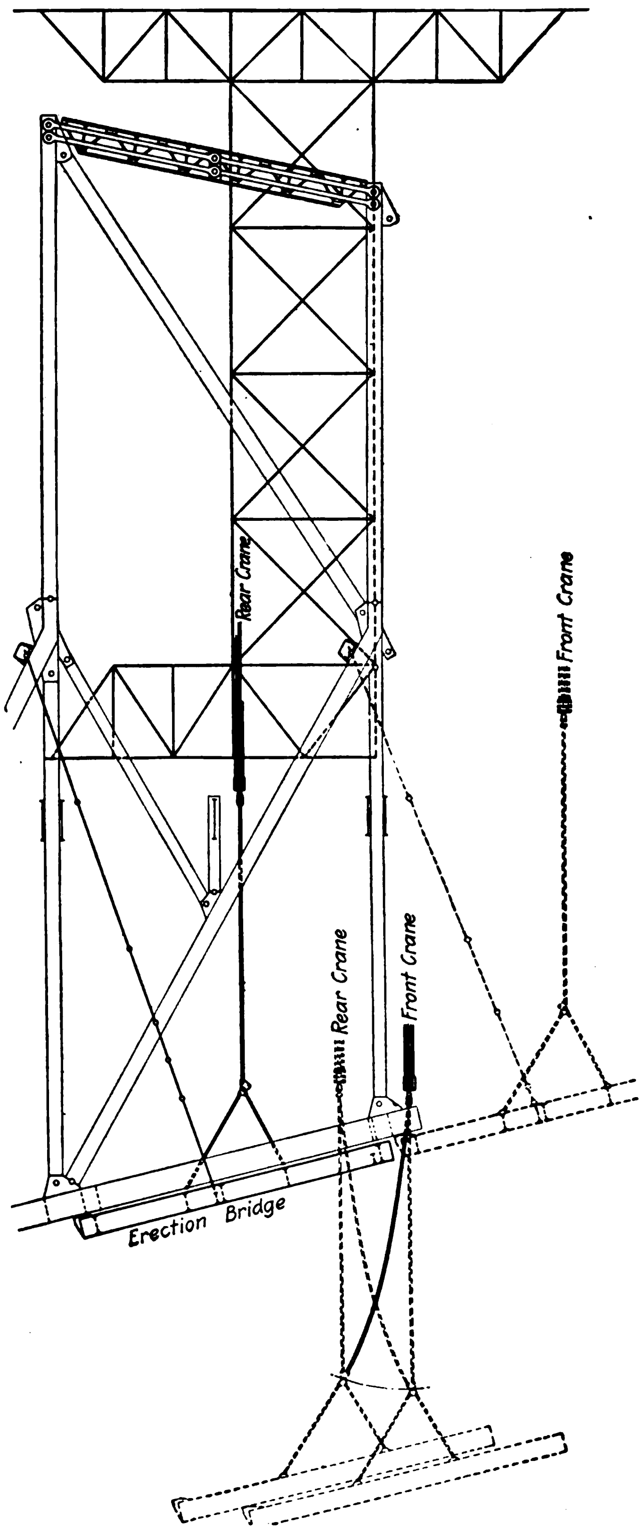


FIG. 6. TRAVELER CRANES HANDLING ERECTION BRIDGE

Each line of eye-bars in all panels over 50 ft. in length is made up of two lengths of eye-bars per panel, pin-connected to each other at the middle point of each panel, where the weight of the pins and one-half the weight of the eye-bars is taken by supporting trusses. Each supporting truss for one panel of eye-bars is made up of four single-intersection Warren trusses connected together in pairs by cover plates, tie-plates and lattice bars. The pairs of trusses are symmetrically spaced about the longitudinal center line of the chord, but are not connected. The eye-bars were assembled with the supporting trusses in the storage yard and the middle pins put in place. They were then taken out on the bridge and hoisted into position, one-half the bars in each panel being placed at one time.

As soon as the eye-bars and supporting trusses were erected, the adjusting erection links for the top chords, illustrated in Fig. 10, were placed in position. These links were made up of a series of plates pin-connected together at the end. The plates were made of such lengths that the same links could be used panel after panel by removing or inserting extra lengths of plate to suit the varying lengths of bridge main panels. Small adjustments were obtained by means of one 100-ton jack in each link reacting against sliding cross-girder heads connecting to the plates at each end of the jack. The

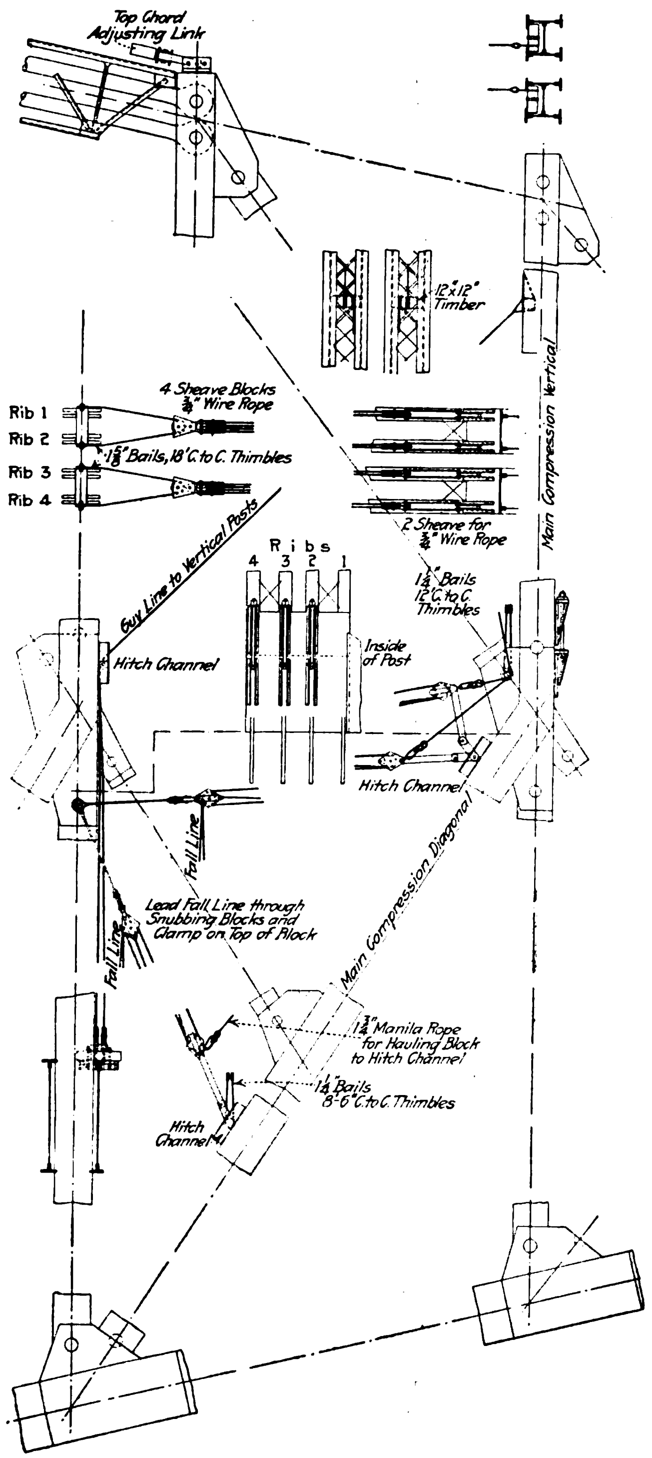


FIG. 7. ERECTION OF DIAGONALS

links, at the ends, engaged brackets which were in turn connected to the main compression vertical posts. The top-chord eye-bars had the pin holes at each end slotted ¼ in. on the side remote from the bearing surface, and these adjusting links were used to draw the top-chord main-panel points together so that the top-chord pins could be easily driven in the elongated pin holes of the eye-bars. These top-chord links also took care of any erection stress in the top-chord panel until the eye-bar pins at each end of the panel were driven. The links were placed in pairs over each chord and were put in position and connected up as soon as the panel of eye-bars with the supporting trusses was placed. They remained in this position until the eye-bars and supporting trusses in the following panel were erected. The pins at the shore end of this latter panel were then driven and the links moved forward into the next panel.

All the material in the panel C16-C14 was now completely erected except the sway bracing for the vertical post CU14-CM14 and that for the upper half of the main compression diagonals, CM14-CL16, which could not be placed until the traveler had moved forward out of the way.

Completing Sway Bracing

The traveler was next advanced until the front legs rested at panel point CF14. The erection bridge was moved forward into position to take the chord members of the following panel and the erection of this next panel and the panels following was proceeded with in a manner practically identical with that followed for the first main panel, as previously described. When the traveler was moved forward to panel point CF13, the sway bracing for the vertical compression post, CU14-CM14, was placed in position; and when the traveler was moved forward to panel point CF12 the portal bracing of the main compression diagonal, CM14-CM15, was erected by the rear booms of the traveler.

The placing of this bracing finished the erection of the complete system of sway bracing for the compression web members in the first main panel. This system of sway bracing is a double-intersection system designed to take both tension and compression in each member, and follows the line of members from CU14 to CM14 to CL16. It takes the panel wind shear from the top-chord and web members of the trusses, carrying it to the plane of the bottom lateral system. Throughout the cantilever and anchor arms there is no top-chord lateral bracing, but in each main panel there is a system of web bracing similar to that just described.

Progress Dates

As already stated, the erection of the first main panel of the south cantilever arm C16-C14 was started April 1, 1916. On Friday, Apr. 28, the traveler was moved forward to panel point CF14, the first main panel being practically completed. The number of days, 10 hr. to each day, actually worked was 22½, only 2½ days having been lost on account of high winds and rainy weather. The amount of steel erected during this time was in the neighborhood of 3,100 tons and included the placing of the largest, longest and heaviest members of the cantilever arm. An average of 200 men for each working day were employed on the work, including from six to eight gangs of riveters.

The material in the second main panel was completely erected by May 26. The steel in this panel weighed approximately 2,650 tons, and was erected in 18 working days. The 1,960 tons in the third main panel was placed by June 12, in 13 working days. The methods employed in the erection of the material in these two panels were similar to those described before, with nothing especially worthy of note except perhaps the placing of the traction truss in the third main panel.

This traction truss is placed in the plane of the sway bracing for the main compression diagonals below the floor line and forms part of the sway-bracing system for this panel. The truss is connected up to the track girders at the floor line and takes care of the traction between the expansion joints at the main post and at panel point

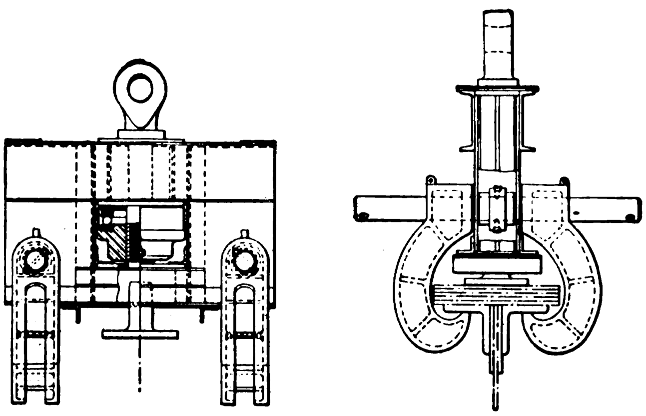


FIG. 8. LIFTING CONNECTION FOR QUEBEC BRIDGE FLOOR-BEAMS

CF7. This truss is a double-intersection latticed girder designed throughout to take both tension and compression in each member. It is 22 ft. deep and 88 ft. long, and weighed in the assembled condition 52,000 lb. It was riveted up completely on the floor of the bridge in front of the traveler and then lifted and placed in position in one piece, using the two main hoists of the front crane of the traveler and the two forward derrick booms.

As the traveler progressed toward the end of the cantilever arm, the members handled were lighter and the field splices fewer and smaller. The rate of progress was therefore greatly increased. By July 10 the traveler had completed the erection of six main panels of the bridge and was standing at panel point 4. The fourth, fifth and sixth main panels were erected in 22 working days, the total weight of steel in these panels amounting to approximately 3,600 tons.

The traveler was moved to its last position at the end of the seventh main panel on July 20. The weight of the steel in the last two main panels amounted to about 1,280 tons. This was placed in 18 working days.

The south cantilever arm was therefore completely erected between Apr. 1 and July 28, 1916. The total weight of steel placed was about 13,000 tons. This material was erected in 92 working days, about 27 days being lost from high winds, inclement weather, Sundays and legal holidays. The south cantilever arm was completed over a month ahead of schedule time and in over 25% less working time than for the north arm.

The pin holes in the eye-bars used in the anchorage chain and the top chord of the anchor arms had each been slotted $\frac{1}{2}$ in., making a total of 2 in. adjustment for each top-chord main panel. The upper pin holes of the main tension diagonals of the anchor arm had also been slotted 2 in. The anchor arms were cambered and the elevations of the tops of the falsework columns carrying the anchor-arm trusses were made of such a height as to allow the top-chord main-panel points to be brought 1 in. closer together than the manufactured length of the eye-bars demanded; slotting the holes enabled this to be done without any difficulty. The purpose of this camber was to avoid as far as possible any trouble that might arise in driving the huge pins connecting the main truss members.

Anchor-Arm Movement

As the erection of the cantilever arms proceeded, the slack in the anchor-arm top chords and main tension diagonals was taken up gradually and almost imperceptibly panel after panel. The first effect of cantilevering, in raising the bottom-chord panel points of the anchor arm, was observed after bottom chord CL11-CL10 had been placed. At this time the middle panel points AL6-8 and 10 of the anchor-arm bottom chord had lifted about $\frac{3}{16}$ of an inch. As the traveler moved forward panel after panel on the cantilever arm, these bottom-chord panel points of the anchor arm were observed to lift in amounts varying from $\frac{1}{4}$ to $\frac{3}{8}$ in. for each additional panel of cantilever-arm material erected. When the traveler was moved out to the end of the sixth main panel of the cantilever arm, the slack in the chain of anchorage bars at the anchor piers was observed to be gradually taken up. This showed that the erection of the cantilever arm had reached a point where the weight of the cantilever arm plus the weight of the traveler

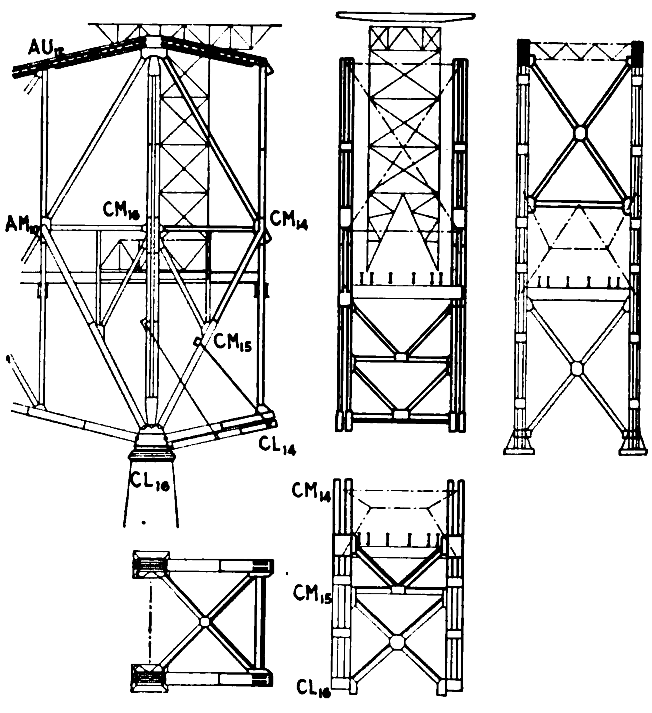


FIG. 9. TRAVELER MOVED UPON CANTILEVER

slightly overbalanced the weight of the anchor arm. Jacks were therefore placed under the falsework columns supporting panel point No. 2 of the anchor arm. The toggles in the chain of anchor bars were eased off; the jacks were then pumped up and the anchor arm was lifted until the chain of anchor bars at the anchor pier became taut. Steel shims were then driven in under the falsework columns at panel point No. 2, and the erection of the remaining panels of the cantilever arm proceeded.

The work of erection of the large compression members was greatly facilitated by the high degree of accuracy that had been required in the shop work. All members which were spliced together in the field were temporarily assembled in the shop and their splices reamed and matchmarked to insure a perfect fit in the field. All holes for field connections, other than field splices in main members, were drilled to a steel templet $\frac{5}{16}$ in. thick, with case-hardened thimbles $1\frac{5}{16}$ in. deep. Where it was necessary to take the pieces apart for shipping and handling, the respective pieces reamed together were

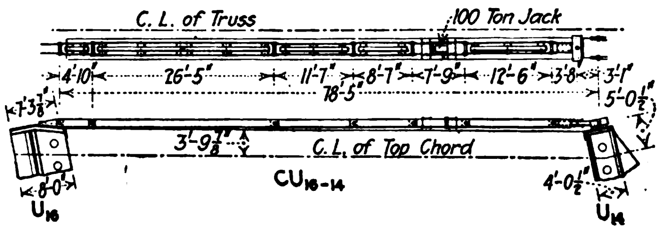


FIG. 10. ERECTION LINKS FOR TOP CHORDS

matchmarked so that they could be reassembled in the same relative position in the final setting up in the field. In the field-splicing of main members, the two portions of the members were accurately faced, assembled in the shop to exact length, and held firmly in position with rods and turnbuckles. The splice material was then placed in position and the holes finally drilled through the main and splice material.

The workmanship and finish required throughout in the shop was the best that the most suitable modern machinery and skilled labor could give. All finished measurements were made with standardized tapes lying flat and supported at frequent intervals, firmly held at one end, and under a tension of 10 lb. The policy of assembling, reaming and matchmarking all members with field splices has been found to give the best of satisfaction, even though it involved considerable extra expense in the shop. The connections in the field were made with so little trouble and loss of time that it is considered that the extra shop cost was fully warranted.

Assembling the Suspended Span

With the completion of the south cantilever arm the bridge is in readiness for the floating in and hoisting into place of the suspended span. This span is 640 ft. long, 88 ft. wide and weighs in the floating-in condition approximately 5,000 tons. The greater part of the floor steel will be left off during the floating-in operation and will be placed by means of derrick cars after the span is coupled up to the ends of the cantilever arms.

The suspended span has been erected in the shallow waters of Victoria Cove, about three miles below the bridge site. The work of erection proceeded simultaneously with that of the south cantilever arm. Foundations for the falsework bents supporting the trusses and the approach track were prepared at the periods of low tide during the season of 1915. This work was rather difficult and could not be carried on with any great speed as the time available was only from two to four hours each day. During erection, the suspended span was supported on staging placed under each panel point. The traveler which erected the north-shore cantilever and anchor arms was erected on bents immediately adjacent to the staging of the span, with the top trusses and traveling cranes left off. All the steel was handled by means of four 70-ft. 30-ton booms, placed one at each of the four corners of the traveler. After the span had been completely erected on the staging, the intermediate staging supports were removed and the span swung on the end supports.

Floating In the Span

In this condition six scows, 32 ft. wide and 160 ft. long, with 11 ft. 7 in. draft, will be floated in and placed under panel points at each end of the span. The valves in the bottom of the scows will be opened and the scows sunk until they rest on their foundation supports. The cross-girders and bracing which transfer the load of the span to the scows will then be placed. To raise the span from the end supports preparatory to floating out, the scows will be drained at low tide and the bottom valves closed; then as the tide rises the span will be lifted gradually and will be in readiness to proceed on its journey to the bridge site, if the weather and tide conditions are considered favorable. While the span is being moved to the bridge site, it will be kept under control by means of tugs of sufficient power capacity to overcome all anticipated resistances due to wind or current. Arriving at the bridge site, the span will be anchored to the ends of hanging trusses, coupled up to hanger slabs

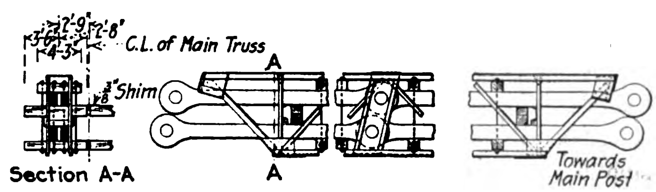


FIG. 11. PANEL OF TOP-CHORD EYE-BARS AND SUPPORTING TRUSS

provided at each of the four corners of the cantilever arms, and raised into its final position by means of movable jacking girders and eight 1,000-ton hydraulic jacks, two at each corner.

The work is being carried out under the supervision of the Board of Engineers, Quebec Bridge, consisting of C. N. Monsarrat (Chairman and Chief Engineer), Ralph Modjeski and H. P. Borden. The St. Lawrence Bridge Company is the contractor for the superstructure, George H. Duggan, being Chief Engineer, George F. Porter, Engineer of Construction, W. B. Fortune, Superintendent, and S. P. Mitchell, Consulting Engineer of Erection.

Competition of Government Engineers with engineers in private practice is agitating the engineering profession in New Zealand. At the annual meeting of the New Zealand Society of Civil Engineers a discussion on this subject is reported as follows in the "Commonwealth Engineer":

Mr. H. F. Toogood stated that the Lake Takapuna Board of Control had requisitioned the government for the services of an engineer to report on the water system. The writer expressed the opinion that the society should protest against local bodies "sponging" on the government for engineers' services. Mr. J. A. Menzies said there were county councils in Otago that employed engineers at £180 per annum and got the government to supply plans which, if the county council employed fully qualified men, would mean the employment of engineers at £300 or £400 per annum. The following resolution was carried: "That the attention of the council should be drawn to the fact that a local body has requested the government to grant the services of one of its engineers to report on a matter of engineering, and that the council be requested to take the matter into consideration with a view to taking such steps as may be necessary for the protection of engineers in private practice."

TABLE 1. RESULTS OF MECHANICAL ANALYSES OF RUBBISH IN % OF WEIGHT, 1914-15

Material	Nov.	Dec.	Jan.	Mar.	May	June	July	Aug.
Newspaper	10.4	17.8	19.5	16.1	15.6	17.2	17.0	16.0
Manila paper	6.6	12.7	7.0	9.5	8.7	7.3	12.1	13.4
Cardboard	0.8	9.4	11.7	10.5	9.7	6.9	10.9	12.8
Books, etc.	4.4	3.0	2.4	2.4	3.3	3.2	1.4	5.6
Mixed paper	16.6	4.4	3.1	5.0	4.6	2.8	2.4	2.6
Rags	5.9	3.7	4.5	5.2	7.2	4.3	5.0	5.7
Wood	1.2	7.4	1.4	2.8	2.6	4.1	3.6	4.1
Leather	1.5	0.2	0.4	0.7	1.2	0.2	1.0	1.1
Rubber	0.3	0.3	...	0.3	0.5	...	...	...
Screenings	...	4.6	18.5	13.4	10.4	18.6	12.3	11.1
Tinware	10.2	7.3	11.7	13.6	8.8	6.6	5.6	6.4
Enamelware	0.4	0.4	0.1	0.4	0.3	5.0	...	0.1
Metals	1.8	0.6	0.3	0.8	1.3	1.1	0.8	1.5
Bottles	11.0	8.1	7.7	9.0	7.7	8.3	8.4	7.2
Broken glass	3.8	3.5	4.0	4.7	5.2	4.3	4.8	3.5
Excelsior	...	0.7	0.3	0.8	0.7	0.1	0.7	...
Mattresses, etc....	5.2	1.1	...	...	0.3	...	0.7	0.1
Matting	3.1	0.8	0.1	0.3	1.6	1.5	1.4	...
Linoleum	0.7	...	...	...	0.3	0.1	...	0.1
Straw	...	...	0.4	...	...	...	...	...
Dirt	16.1	14.0	6.9	4.5	10.0	8.4	11.5	8.7
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

in the rubber resulted from the clay and similar filler, as well as the fabric contained in the rubber. The high ash value of books and magazines was attributed to the sizing of the paper, which would account for its low calorific value.

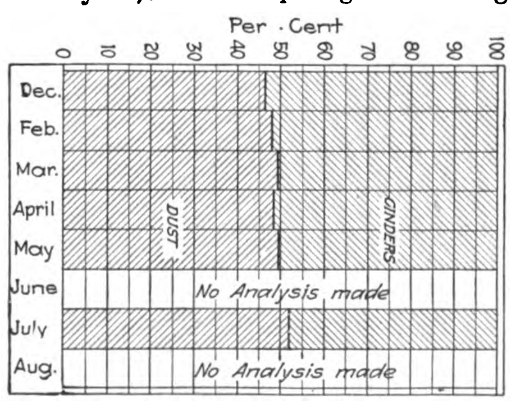
From the results obtained in making calorific tests of the various component parts of the rubbish it is possible to compute the calorific value of the rubbish as collected. The percentages of the various constituents determined by making mechanical analyses, multiplied by the calorific values obtained, give the average calorific value of the rubbish as collected.

In comparing the calorific values with the average constituents found in the rubbish as a whole, taking into account all items determined by mechanical analyses, it was found by computation that the average calorific value in samples as collected, including the noncombustibles, such as metal, dirt, etc., amounted to 5,500 B.t.u. per lb.

Analyses of Ashes

In making the analyses of the ashes it was found that there was no definite relation between the weight of the cinders and the weight per cubic yard of the ashes, although the weights show a slight range per cubic yard. In cases where the maximum weights were found, the ash contained a higher percentage of moisture, the percentage of cinders depending on the residences from which it was collected, as would be expected.

The percentage of cinders produced in the ash was approximately 50%. On comparing the results given in



PROPORTION OF CINDERS IN ASHES

tables of calorific value it was evident that considerable combustible material was left in the ash. The average of 45 analyses of the screened-out cinders showed the following: Moisture, 2.05%; volatile matter, 3.53; fixed carbon, 51.1; ash, 45.3%. The average B.t.u. per lb. in the samples as received was 7,858; and dried, 7,932. The

TABLE 2. RESULTS OF CALORIFIC TESTS ON COMPONENTS OF RUBBISH, BY BUREAU OF MINES*

		Dry Coal					
Component	Moisture	Volatile Matter	Fixed Carbon	Ash	Sulphur	B.t.u. as Received	B.t.u. Dry Coal
Newspaper	6.2	86.9	10.0	3.1	0.10		8,230
	8.67	87.41	7.84	4.75	0.11	7,485	8,196
Manila paper.....	5.1	88.9	8.6	2.5	0.25		7,840
	8.99	89.12	8.14	2.73	0.13	7,626	8,379
Mixed paper......	5.3	84.8	10.7	4.5	0.15		7,910
	9.07	86.85	9.6	3.55	0.16	6,867	7,552
Books and magazines	4.4	71.2	5.1	23.7	0.35		5,630
	7.24	74.72	4.88	20.4	0.08	5,400	5,821
Cardboard	6.0	80.3	10.4	9.3	0.15		7,430
	8.67	84.39	9.33	6.28	0.16	7,154	7,833
Screenings	8.6	78.1	11.4	10.5	0.30		6.910
	9.97	83.11	9.08	7.81	0.46	6,483	7,201
Wood	6.3	87.6	11.0	1.3	0.10		8,910
	9.55	87.78	10.22	2.0	0.19	7,721	8,536
Excelsior	7.0	88.5	10.4	1.1	0.05		8,580
	9.48	92.76	6.8	0.44	0.19	7,853	8,675
Matting	10.20	82.41	8.08	9.51	0.19	6,266	6,978
Rags	3.7	89.9	8.8	1.3	0.10		7,410
	6.47	94.39	2.45	3.16	0.16	7,312	7,818
Rubber	1.2	...	...	47.6	0.65		6,620
Leather	5.9	72.3	17.7	10.0	0.45		8,530
	10.33	78.64	11.18	10.18	0.45	8,240	9,189
Dirt	2.7	20.2	9.1	70.7	...		4,261
	3.7	22.27	3.87	73.86	...	3,661	3,802
Linoleum	2.1	65.8	6.8	27.4	0.40		8,310
Mattress	6.4	75.4	18.5	6.1	0.25		7,430

*The average calorific value per pound of rubbish as collected, including dirt, metals and other noncombustible material, amounts to 5,500 B.t.u.

calorific value of this ash, if recovered each year, would amount to that produced by approximately 24,000 tons of coal. The highest calorific value of ashes was found in the collections from the residential sections, which contained a higher percentage of unburned coal.

Quebec Bridge in Picture

Four striking views of the Quebec Bridge are presented on two following pages. They immediately recall to all who have followed this, the world's most notable piece of bridge work, some of the outstanding features of design and erection which it is well to review as the day of completion draws near.

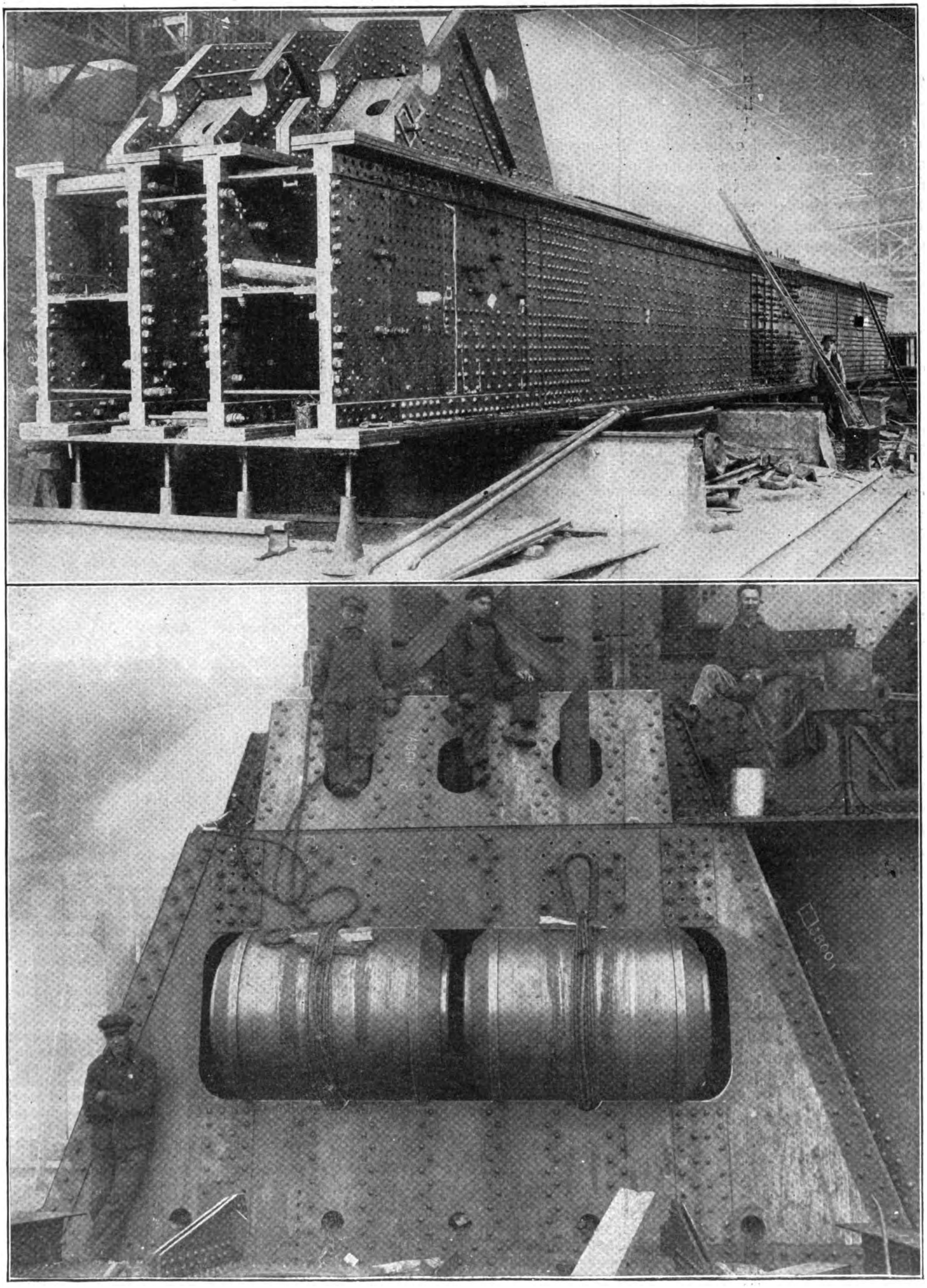
One view shows the famous K-truss in the anchor and cantilever arms, the first use of an epoch-making departure in design. With this goes the novelty of a bridge within a bridge to carry the floor, the extensive use of nickel-steel and larger and longer rivets than ever before. A second view shows the 640-ft. suspended span, in itself the longest single-span bridge yet completed.

The views of two great bottom-chord sections and a pair of the huge main-shoe multiple pins are reminders not only of a pioneer design but also of the great accuracy to which the builders' shop practice was developed. So determined was the bridge company on this last point that the shop handled nothing on which coarser work was permissible; even the falsework was sublet to get it outside. The benefits of a new shop, new machines and new methods were reflected in the smooth machine-like progress of erection and in elimination of field corrections.

Taken together any collection of such views must emphasize the boldness of the erection schemes, as for the elimination of secondary stresses in bottom chords, the lavish expenditure on construction adjuncts like the 200-ft. traveler, the anchor-arm falsework, the flying bridge, the top-chord jacking links and eye-bar trusses. Yet with all this completeness of preparation, the amount of junk has been kept down to a probable 5,000 tons by studied uses of various pieces—even of the bridge itself—in different parts of the work. And finally any collection of Quebec Bridge pictures must always bring fresh to mind the names of a few men closest connected with the design and erection—Johnson, Duggan, Porter, Mitchell, Fortune; Monsarrat, Modjeski, Schneider.

COMPONENT PARTS OF THE GREAT QUEBEC

At the top, suspended span as erected on falsework near shore at Sillery, ready for transfer to six scows for floating and hoisting; view taken with erection traveler in last position. Below, the bridge seen nearly endwise from camp on north shore about two months ago. North cantilever arm completed; the south arm was finished July 28, 1916



BRIDGE ACROSS THE ST. LAWRENCE RIVER

At the top, two of the great bottom-chord quarter sections laid together in the St. Lawrence Bridge Co.'s shop for drilling, facing and matchmarking the splice. Below, the 45-in. sleeves on 30-in. pins, in the main shoe ready to receive the first section of the great bottom chord

Quebec Suspended-Span Hoisting Details Completed

By A. J. Meyers*

SYNOPSIS—Outline of program and details of equipment for floating to position and hoisting into place between cantilever arms the 640-ft. 5,000-ton suspended span of the Quebec Bridge.

The detailed arrangements for floating the great suspended span of the Quebec Bridge are now practically finished, and the completion of this task rests on favorable meteorological conditions. This span is 640 ft. long, 88 ft. wide and weighs, in floating-in condition, about 5,000 tons. It was erected at Sillery, about 3 mi. below the bridge site, over the shallow waters of Victoria Cove. During erection it was supported on staging under each panel point; afterward the intermediate supports were removed. As shown in Fig. 1, the suspended span, after it had been completely assembled and riveted up, rested on the end staging bents at L0 and L18. The scows for floating the span to the main bridge site, about 3 mi. up the river, were floated into the positions shown in the diagram under panel points L1, L2, L3, L15, L16 and L17, and as the tide lowered they came to a bearing on their concrete and timber beds. In the bottom of these scows, valves are provided which were opened and will be left open until the span is to be floated, so that the scows and the span will not be disturbed by the daily rise and fall of the tide.

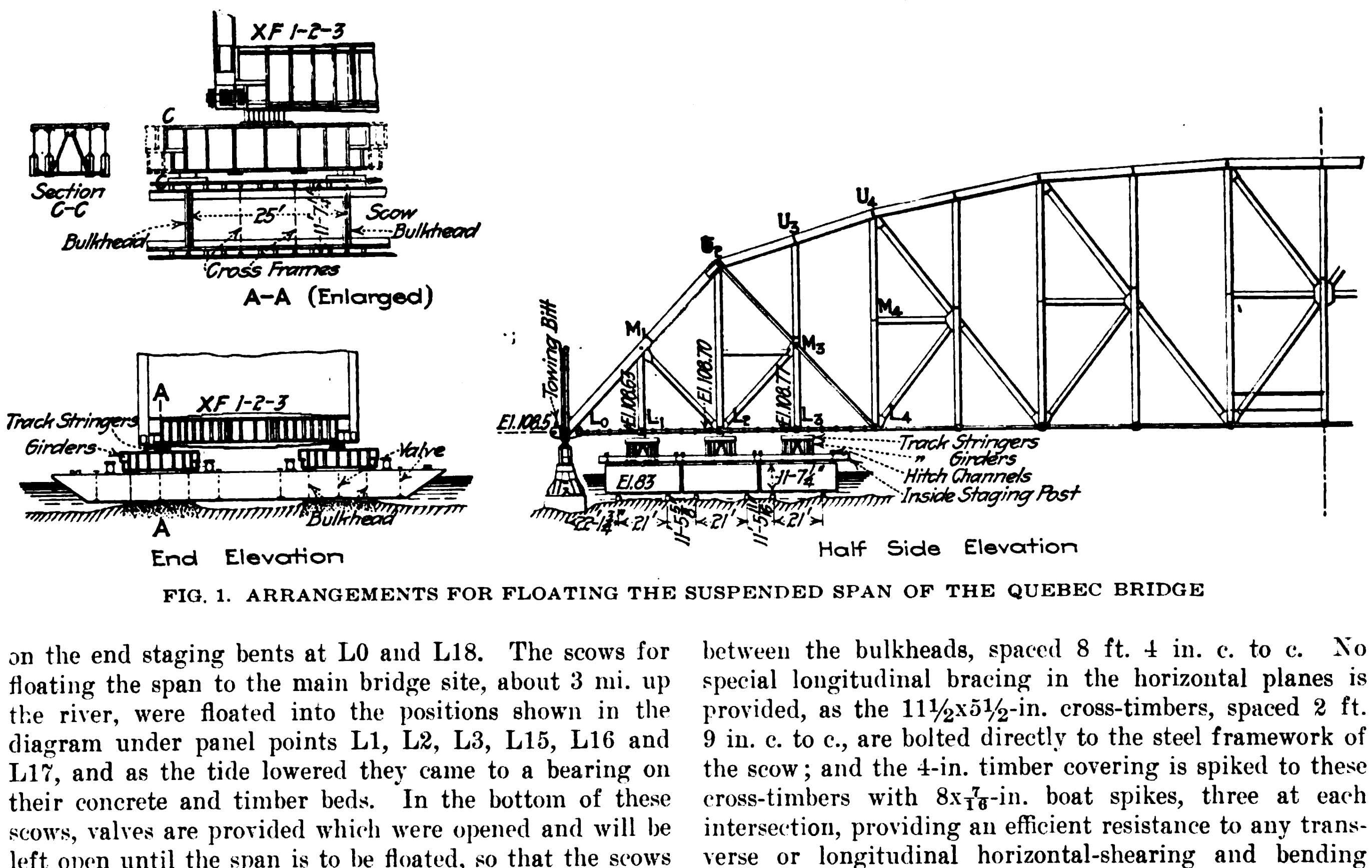


FIG. 1. ARRANGEMENTS FOR FLOATING THE SUSPENDED SPAN OF THE QUEBEC BRIDGE

Scows for Floating-In

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 after their work of floating-in the suspended span was completed. The average length from crest to crest of wave at the bridge site is about 40 ft.; the wave height is 4 ft. This unevenness of the surface of the river produces unequal upward pressures at the four corners of the span and consequent stresses in the sway and lateral bracing. The unequality of pressure is 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 would preferably be used. With the design of scow adopted the oscillation of the span from wave action produces only stresses in the sway and lateral bracing, which these systems are well able to resist.

The scows as built are 32 ft. 5½ in. wide, 164 ft. 6 in. long and 11 ft. 7½ in. draft over bilge timbers. Each has a steel frame made up of three longitudinal trusses, spaced 10 ft. 6 in. c. to c. and braced transversely by four water-tight steel bulkheads with intermediate cross-frames between the bulkheads, spaced 8 ft. 4 in. c. to c. No special longitudinal bracing in the horizontal planes is provided, as the 11½x5½-in. cross-timbers, spaced 2 ft. 9 in. c. to c., are bolted directly to the steel framework of the scow; and the 4-in. timber covering is spiked to these cross-timbers with 8x$\frac{7}{16}$-in. boat spikes, three at each intersection, providing an efficient resistance to any transverse or longitudinal horizontal-shearing and bending forces that may arise.

The load of the suspended span is transferred to the bulkheads by means of the cross-girders and I-beams shown in Fig. 1. The bulkheads transfer this load to the longitudinal trusses, which distribute it over the length of the scows.

In addition to the scows being designed to carry the load of the suspended span, in order that they may be used for freight-carrying purposes after their work of floating-in the span is completed, they are built to carry a load of 1,400 tons uniformly distributed over a length of 123 ft. and symmetrically placed about the transverse and

*Chief Draftsman, Board of Engineers, Quebec Bridge, ntreal, P. Q.

longitudinal center lines of the scows, or a load of 1,075 tons uniformly distributed over a length of 50 ft. at each end of the scow and symmetrically placed about its center lines. These two conditions of loading give the largest stresses in the web members of the three longitudinal trusses. The largest stresses in the chords of these trusses are caused by a loading of 280 tons uniformly distributed over a length of 50 ft. on either side of, but immediately adjacent to, the transverse center line of the scow, or a load of 180 tons uniformly distributed over a length of 50 ft. at the center of the scow and symmetrically placed about its center lines.

It was assumed that the weight of the scow itself produced no stresses in the longitudinal scow trusses and that the total superimposed load was carried equally by these three trusses. In calculating the maximum bending stresses in the chords of these trusses a coefficient of 0.75 was used to allow for continuity over the panel points. The allowable unit stresses provided for a safe unit stress in these trusses with the suspended span carried on four instead of six scows. The scows are placed under the suspended span and shimmed against the bottom flanges of the floor-beams. To make sure of favorable weather conditions while the span is floating, it should be lifted from its supports at L0 and L18 only immediately before its journey to the bridge site. In order to prevent the lifting of the span by the scows when the weather conditions are not favorable, there are a number of 8-in. disk bottom valves in each scow, and these are left open. The elevations of the beds of the scows were so chosen that the scows will be emptied through these bottom valves during the last low tide before beginning the journey to the site of the main span. The valves have a total area of one five-thousandth the clear area of the scows. All interior areas of the scows are given unobstructed access to some one of the valves, and it is estimated that the water will drain out practically as fast as the tide falls.

Scow Loading

As illustrated in Fig. 1, the load of the suspended span is transferred from the floor-beams at each of the panel points L1, L2, L3, L15, L16 and L17 to the bulkheads of the scows by means of eight 24-in. 80-lb. I-beam track stringers, with their end-connection angles interlocked at the ends, and four track girders braced together by sway-bracing frames and top laterals. These I-beam stringers and track girders are part of the permanent floor material of the span—all the floor steel and the railway track floor, except the main floor-beams, being left off the span during the operation of floating-in and hoisting. (This floor material will be placed afterward by means of derrick cars.) The total load carried by one scow under these conditions is 970 tons, distributed over four bulkheads. The draft of the unloaded scow is 1 ft. 6 in., and when carrying the load of 970 tons the draft is 8 ft. 2 in.

The stresses in the truss members of the span while it is being supported entirely by the scows are such that a tension connection had to be provided at the joint U2; and the bottom-chord eye-bars between the panel points L0-L4 and L14-L18 had to be stiffened temporarily, as indicated in Fig. 1, with longitudinal timbers and transverse blocking and bolts. The subtension verticals and the subcompression diagonals directly over the scows had also to be specially designed and stiffened to take reversal of stress while floating the span.

The three scows at each end of the span are braced and connected together, as shown in Fig. 1, by using the inside staging posts from the falsework of the anchor arm as continuous connecting girders. Four of these posts are used for each set of three scows and are spaced 42 ft. c. to c. These posts were connected to the scows by means of a pair of cross-channels, pin-connected to vertical angles which were in turn bolted to the transverse bracing frames of the scows. Wedges were driven between the posts and the scow decks. These connections and the connecting girders were calculated to resist the bending and shearing stresses arising from the action of a 4-ft. wave.

Just before the span is lifted off the supports at L0 and L18 the load is nearly all taken by the scows, and the span could be easily displaced from its position by the current and wind, unless it is anchored against their combined effect. It is desirable to prevent this shifting off before the actual moment of starting arrives, inasmuch as it may happen that after deciding to raise the span preparatory to moving out, a change in the weather conditions may make it desirable not to proceed on the journey and the span would have to be returned again to its bearings on the staging bents, to await the next favorable opportunity for making a trial.

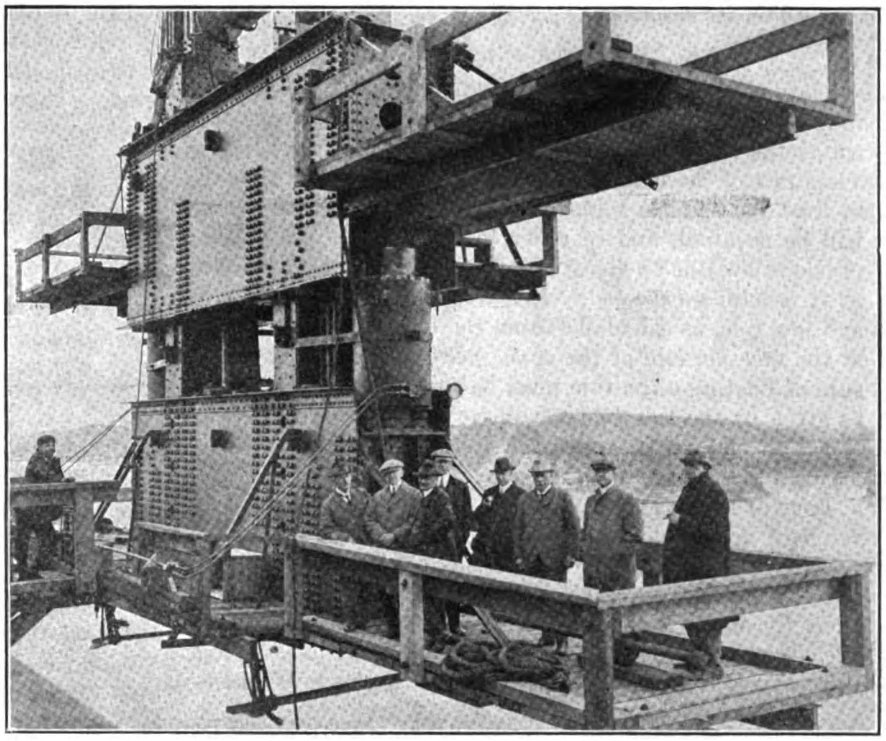


FIG. 2. UPPER AND LOWER JACKING GIRDERS FOR HOISTING QUEBEC SUSPENDED SPAN

The two 1,000-ton hydraulic jacks and operating platforms in place. On the platform—Messrs. Duggan, Mitchell, Porter, Johnson and Fortune, of the St. Lawrence Bridge Co.

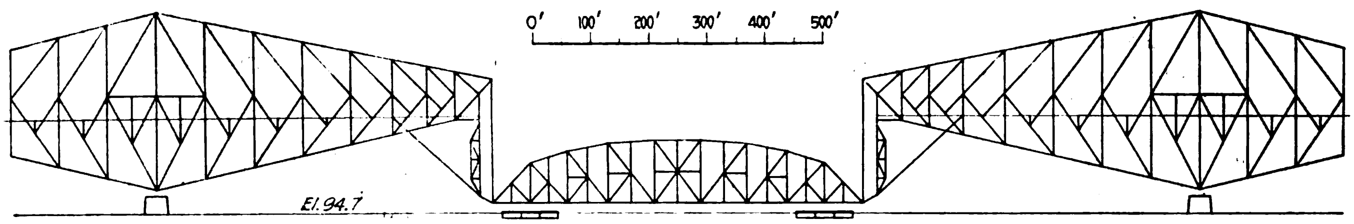


FIG. 3. GENERAL SCHEME FOR HOISTING THE QUEBEC BRIDGE SUSPENDED SPAN

To keep the span in its position until the final decision to float away has been made, timber bents will be placed between the points L0 and L18 and the adjacent scows and also bents on the shore side of the span, against which the scows will guide themselves as the span is raised or lowered on its supports.

Weather and Tide

The bottoms of the scows are placed at El. 83, where they rest on bearing timbers. The bed of the river over which the scows must pass will be cleared off to El. 82. The draft of the loaded scows will be 8 ft. 2 in. In order to float the span, a high-tide elevation of at least 92 ft. will be required, and in order to drain the scows at the previous low tide an elevation of low tide of not more than 82 ft. would be expected.[1] Inasmuch as elevations of high and low tide, as calculated from the tide tables, may vary at the erection site of the span ± 2½ ft., in order to be sure of floating off, a tide must be chosen whose elevation, as given by the tide tables, will correspond to a high-tide elevation of 94.5 ft. and a low-tide elevation of 79.5 ft., giving a range of tide of 15 ft.

Four or five days in succession, when the elevations and range of tide would be suitable for draining the scows and floating the span, occur at intervals of about two weeks' time. The first favorable tide period, after the preparations for floating the span are complete, is about Sept. 12. If the weather conditions are not favorable during this period, it will be necessary to await the next favorable height and range of tide, and so on until suitable tide and weather conditions coexist.

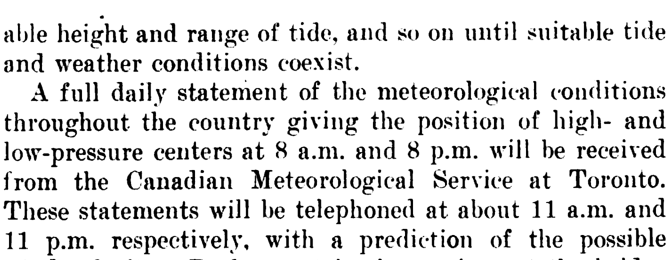


FIG. 4. LINES FROM CENTER SPAN TO MOORING TRUSSES

A full daily statement of the meteorological conditions throughout the country giving the position of high- and low-pressure centers at 8 a.m. and 8 p.m. will be received from the Canadian Meteorological Service at Toronto. These statements will be telephoned at about 11 a.m. and 11 p.m. respectively, with a prediction of the possible wind velocity. By barometric observations at the bridge site it can be estimated whether any threatening centers of low pressure, indicating strong winds, at a great distance on the previous day, have moved more quickly or slowly than was expected. The appearance of the sky, the velocity and direction of the wind just before starting and the indications on an electric storm detector will also be well considered before deciding whether or not to start. It is estimated that any winds which will exert a greater pressure than 2 lb. per sq.ft. can be foreseen, and in that event no start will be made. The current velocity at the bridge site is a maximum, one hour before high tide, and is flowing westward at a rate of 6.3 to 7.3 mi. per hr. in a direction which will carry the span toward the main bridge site. At high tide the current velocity is less by about 1 mi. per hr. The change of current from a westward to an eastward direction, when the velocity is zero, occurs about 1 hr. after the time of high tide.

Towing the Span

The span on its journey to the bridge site will be towed and controlled by tugs, assisted by the westward current and influenced by the coexisting wind of unknown direction, but exerting a force of not more than 2 lb. per sq.ft. With tugs having a pulling capacity of 100,000 lb. in a 4-mi. current, a velocity of the span of 4 mi. per hr. can be produced relative to the water, and at the same time overcome the effect of a 2-lb. wind on the span.

About 50 min. before high tide the span will be floated away from its erection site, with a westward current having a velocity of about 6 mi. per hr. At first the tugs will be used mainly for guiding the span. While on its journey to the bridge site the rate of progress will be observed by means of a series of ranges placed 0.2 mi. apart within 1 mi. of the bridge and ½ mi. apart from 1 to 3 mi. distant from the bridge. The span should arrive at the bridge with a velocity of current of about 4 mi. per hr. With such a current and a wind velocity of not more than 2 mi. per hr. the tugs will have no difficulty in stopping the span before coupling up to the mooring trusses.

The time of arrival will be controlled by the tugs so that the span will be in position about half an hour after high tide, when, for a period of 1 hr., the current does not exceed 3 mi. per hr. and during which it changes direction. The tugs will hold the span against the wind and current while the 1¼-in. steel mooring lines are being connected, as shown in Figs. 4 and 5. The span will then be pulled directly under its final position in the bridge by means of these 1¼-in. mooring ropes, eight in number, two connecting at each corner of the span. At the end of each rope is a loop which, as soon as the span has come within reach and the speed is controlled, will be thrown over a double-headed cast-steel snubbing block or towing bitt, bolted to a seat provided at the joint XL0 of the suspended span. Each 1¼-in. rope is calculated to take a pull of 75,000 lb. The ropes pass through sheaves at

[1]In order to obtain the relation between the Quebec elevations of tide, as given in the Dominion Naval Service tables, to the elevations used at the bridge site, it is necessary to add to the high-tide elevations 80 ft., and to the low-tide elevations 79.25 ft. Observations, extending over a considerable time, have shown that the deviation from the elevation for the bridge site of high tide will amount to a maximum of ±2 ft. for high tide and for low tide ±2.05 ft. At the erection site of the suspended span the deviation from the elevation as calculated from the tide tables is ±2½ ft. for high and low tides.

the lower corners of the mooring trusses and from there run vertically to the trusses, where they connect to a nine-part 3/4-in. wire-rope tackle, which leads back to the drums of the derrick hoists, situated on the floor at the ends of each cantilever arm.

With one line out on each end, the load for a 7-mi. current and 1-lb. wind (77,500) would be carried. With two lines cut (second position, Fig. 4) the upstream pull of 145,000 lb. would provide for a 7-mi. current and a 6-lb. wind (145,000 lb.). With all four out at each end, a 220,000-lb. force could be exerted to overcome a 7-mi. current plus an 11½-mi. wind (219,000 lb.). In final hoisting position the upstream force becomes 121,000 lb., which is good for a 7-mi. current and a 4-lb. wind. By transferring line 1 to bitt 2 in the third position a 7-mi. chords of the cantilever arms, by means of a nine-part 7/8-in. wire-rope tackle leading from the lower corners of the trusses to the connection to the floor between panel points CF5 and CF6 of the cantilever arms and from there to the main hoists, situated at the floor level of the cantilever arms and on the center line of the bridge. These trusses and their connections throughout were designed to take a transverse pull from the suspended span of 300,000 lb.

Work of Hoisting

As soon as the span is pulled into position, before being lifted from the scows, the hanger chains will be swung down and connected through the slotted holes at the lower end to the pins at the top of the short hanger link, shown

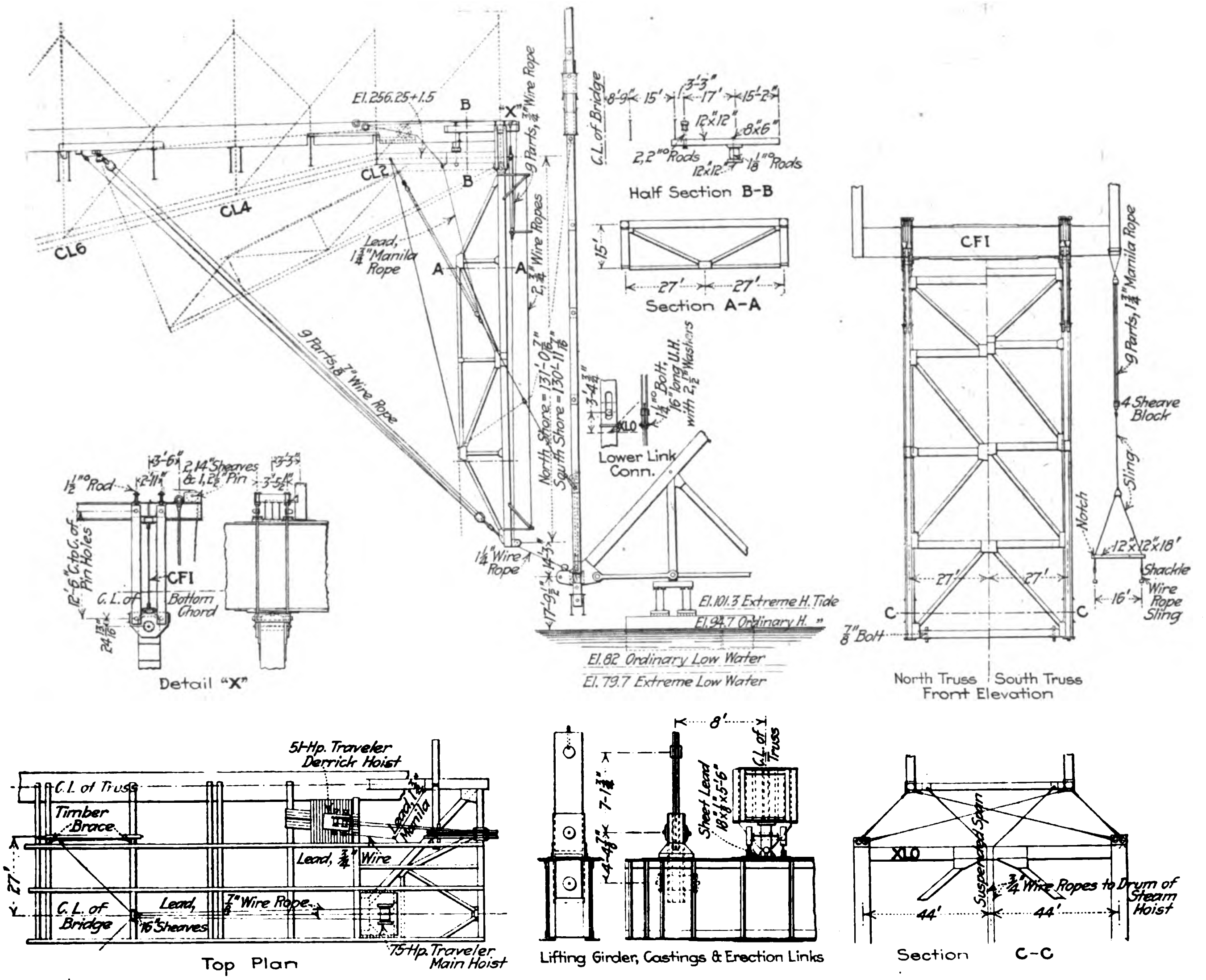


FIG. 5. DETAIL OF ARRANGEMENTS FOR MOORING AND HOISTING THE SUSPENDED SPAN OF THE QUEBEC BRIDGE

current and a 17-lb. wind (293,000 lb.) would be overcome; by transferring line 4 to bitt 2 in the final position a 7-mi. current and a 9-lb. wind (186,000 lb.) could be provided for.

The mooring frames, as shown in Figs. 3 and 5, are made of two steel trusses with bracing and are suspended from the cantilever-arm floor-beams at panel point CF1. They are hung at the upper ends so that they can be swung back, in order not to obstruct the channel unnecessarily, practically up against the plane of the bottom in Fig. 6, connecting to the supporting girders under the joint XLO.

These hanger chains at each corner of the span are made up of four strings of slabs to each chain. Each slab is built up of two 30x1⅛-in. carbon-steel plates. The allowable working erection unit stress through the pin holes was 20,000 lb. per sq.in., which included the stress from 20% of the lifted load as impact. No reinforcing pin plates were used around the pin holes, and special tests made showed that this apparently high working stress through the pin holes was perfectly safe for this type of connection. The slabs were manufactured and shipped in lengths of about 30 ft. c. to c. of end pin holes. They were controlled after being suspended from the jacking girders by means of a two-part tackle connecting to the cantilever-arm trusses at panel point CL2.

The hanger chains connect at the lower end to supporting girders, shown in Fig. 6. These supporting girders are 6 ft. 11½ in. back to back of angles and 25 ft. long. They are built up of two plate girders, connected together by bearing, stiffening and pin connection diaphragms and also by cover-plates. The load of the suspended span is transmitted to the girders by means of a cast-steel rocker joint, designed to allow turning about the transverse and longitudinal axes of the bearing.

The upper supporting girders at the CUO joint of the cantilever arm are designed in a similar manner to the lower supporting girders, the rocker bearing for the girders and the pin connection for the vertical hangers allowing turning about both the transverse and longitudinal axes of the supporting girders. With bearings of this design the suspended span may move in any direction under the influence of whatever external forces from wind or current may act on it during the hoisting of the span.

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

As shown in Fig. 6, the jacking girders are located at the same elevation as the floor of the cantilever arm. They are hung from the upper supporting girders by stiff hangers that are pin-connected at the upper and lower ends. At the lower ends these stiff hangers are attached to guides built of plates and angles that pass through the upper jacking girders and are riveted into the lower jacking girders. The position of the lower girders is therefore fixed, and their distance from the panel point CUO does not change during the jacking operation. The upper girders are the movable girders, and they slide up and down the guides as the 1,000-ton jacks are operated. These jacks are placed between the upper and lower jacking girders, two at each corner of the span, and do 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 are provided with rocker seats at their upper and lower bearings. They are located at the extreme ends of the jacking girders, where they bear

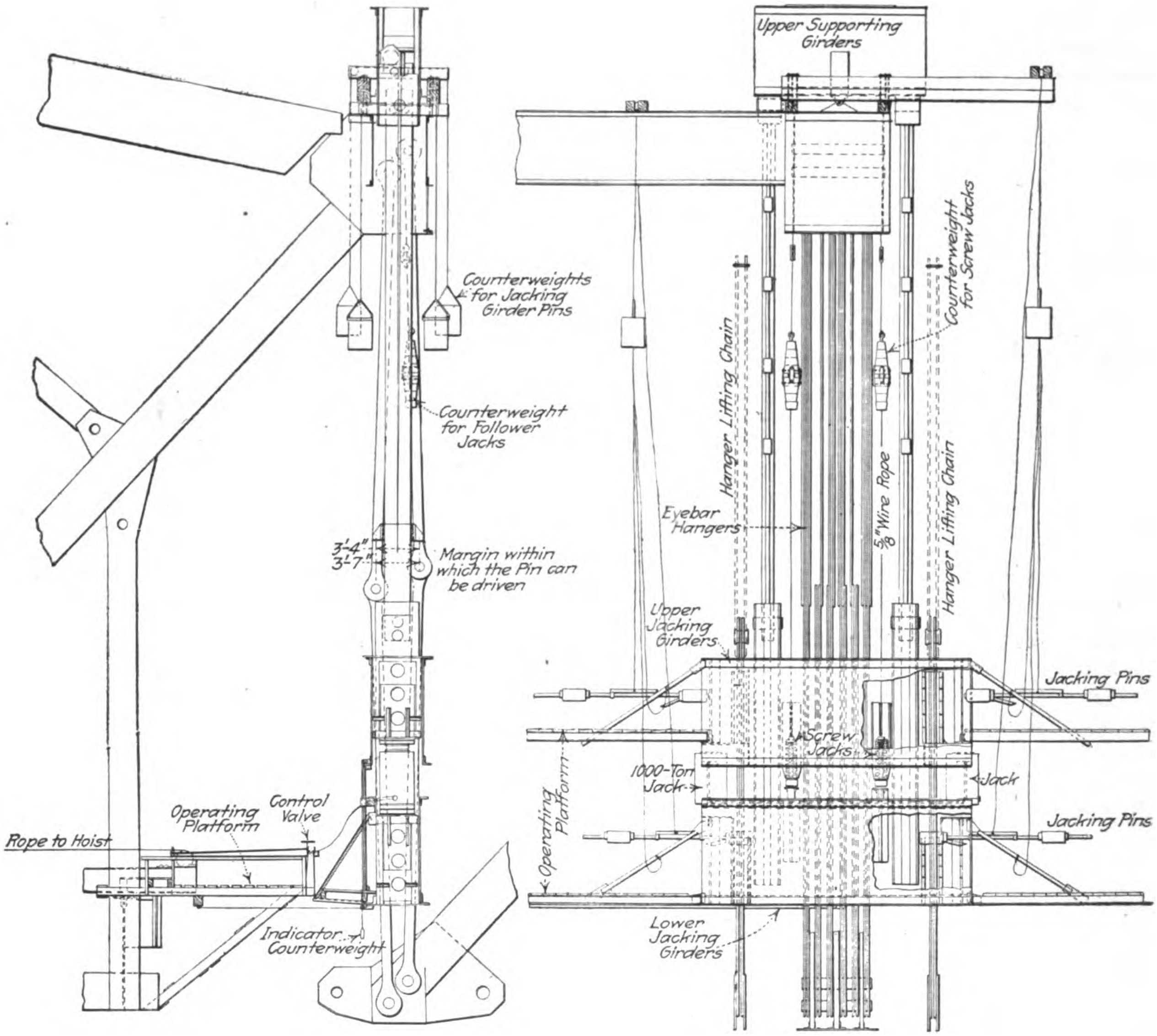


FIG. 6. JACKING EQUIPMENT FOR HOISTING THE QUEBEC SUSPENDED SPAN

against transverse diaphragms riveted into the jacking girders.

In addition to the hydraulic jacks, following-up screw jacks are provided as a safety device in case anything should go wrong with the pumping system for the hydraulic jacks or the jacks themselves, so that they should fail to maintain the pressure of about 4,500 lb. per sq.in. necessary to hold the weight of the suspended span while being lifted. These screw jacks also react against cross-girder diaphragms in the jacking girders. The screw itself is counterweighted so that practically all the friction due to its own weight is eliminated, and the operator of the screw jacks will be able to turn the screw without difficulty and follow the operations of the hydraulic jacks with equal speed and very little exertion.

The hanger lifting chains are guided between cross pin-bearing diaphragms riveted into the jacking girders. These chains are bored every 6 ft. c. to c. to receive a 12-in. pin while the cross-diaphragms have holes for the same diameter of pin bored at 2-ft. centers. The clearance provided in the pin holes of the hanger chains is ½ in. transversely and ⅞ in. longitudinally, and in the pin holes of the cross-diaphragms 1 in. transversely and 1¾ in. longitudinally. This clearance is considered ample to allow the pins to be driven, no matter what position the span may take while hoisting, due to the action of current and wind. Having the pin holes in the cross-diaphragms at 2-ft. centers enabled the pin holes in the hangers to be bored at 6-ft. centers and at the same time accommodated the 2-ft. stroke of the jacks.

Each operation of the jacks will lift the span 2 ft. During the lifting or upward stroke, the 12-in. pins engage the hanger chains through the diaphragms in the upper jacking girders. At the finish of the stroke the pins are entered in the diaphragms of the lower jacking girders to engage the hanger chains. The upper pins are then removed, the jacks and upper girders lowered, the upper pins again entered, the lower pins removed and the jacks again operated. As each 30-ft. length of hanger chain passes up through the upper jacking girders it is disconnected and removed. The jacking pins are counterweighted and balanced to enable them to be handled with facility by the men on the operating platform.

The jacks are supplied with water under pressure of about 4,500 lb. per sq.in. by a pair of direct-acting double-plunger pumps, operated by compressed air and located on the center line of the bridge floor at the ends of the cantilever arm. By means of a pair of control valves installed in front of the pumps the supply of water sent to each corner of the span can be regulated, and in this manner with the aid of a simple counterweighted-line indicator in front of the valve operator, which will show any difference in level between the lifting girders on each side of the bridge, the two corners of each end of the span can be kept at the same elevation. Another set of valves with a similar indicator is placed on the operating platform in front of each set of jacking girders to control the water-supply to each separate jack, so that the ends of the jacking girders, during jacking operations, can be kept level. The feed-pipe line is connected to the jacks by means of two ⅜-in. copper pipes which are sufficiently flexible to allow for any swaying motion of the span while being hoisted.

Tests of the complete water-feed pipe lines and hydraulic jacks were made under a pressure of 6,000 lb. per sq.in. The jacking girders were tested with a load on the jacks equal to 5,000 lb. per sq.in. pressure. The working load on these jacks would be about 4,500 lb. per sq.in. Each individual scow was tested for leakage after the scows were in place, by closing the bottom valves of one scow at a time and allowing it to react against the weight of the span as the tide rose.

The vertical distance through which the span will be hoisted depends upon the varying elevation of the water level, but will be approximately 145 ft. Each operation of the jacks hoists the span 2 ft., and a cycle will take about 15 min. to complete. Altogether there will be approximately 73 separate lifting operations, and the time consumed from the moment of coupling up to the hanger lifting chains to the moment of driving the last pins connecting the two portions of the permanent eye-bar suspenders will be approximately 20 hr., provided no unforeseen delays occur.

The work is being carried out under the supervision of the Board of Engineers, Quebec Bridge, composed of C. N. Monsarrat (Chairman and Chief Engineer), Ralph Modjeski and H. P. Borden. The St. Lawrence Bridge Co. is the contractor for the superstructure, of which company Phelps Johnson is President; G. H. Duggan, Chief Engineer; George F. Porter, Engineer of Construction; S. P. Mitchell, Consulting Engineer of Erection; and W. B. Fortune, Erection Superintendent.

Enlarging the Engineering Societies' Building

The August *Proceedings* of the American Society of Civil Engineers contains the final agreement between the governing board of that society and the United Engineering Society, under which the former becomes one of the founder societies and acquires the ownership of a one-fourth interest in the Engineering Societies' Building at 29 West 39th St., New York City, with the ground on which it stands.

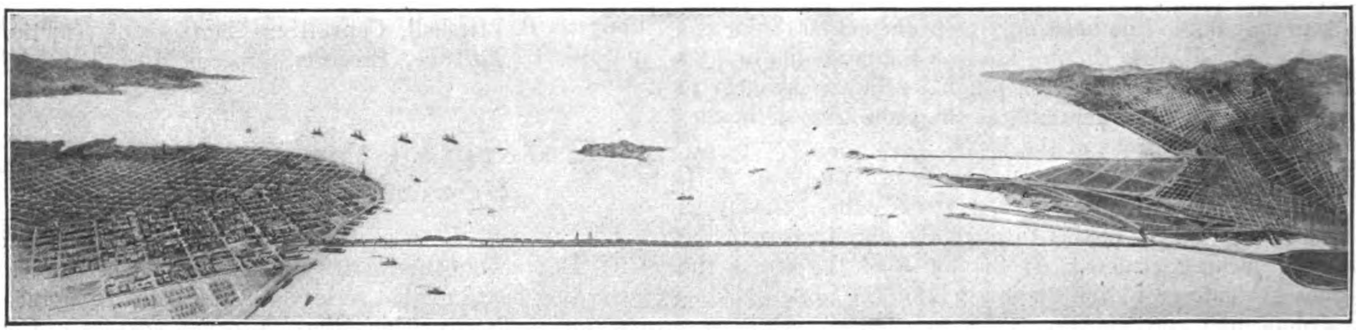
The building is to be enlarged by the addition of three more stories, making a total height of 15 stories. The building committee in charge of this work is made up of one member from each of the constituent societies, Messrs. H. H. Barnes, Jr., H. G. Stott, Charles Warren Hunt and Charles F. Rand. The committee has awarded the contract for the preliminary structural work, and the contractor began work on Aug. 1.

The American Society of Civil Engineers is to pay the cost of adding these three additional stories, provided the cost is not in excess of $250,000, in which case the balance will be paid by the United Engineering Society. The library now owned by the American Society of Civil Engineers is to be added to the Engineering Societies' library, the whole to be conducted as one public library under the management of the United Engineering Society. The representatives of the American Society of Civil Engineers in the United Engineering Society are Clemens Herschel, Charles Warren Hunt and J. V. Davies. Suitable changes have been made in the constitution and bylaws of the United Engineering Society so that the four societies stand on an equal footing in it, exactly as would have been the case had the American Society of Civil Engineers joined the other three societies in the building enterprise twelve years ago.

Planning for a Bridge Across San Francisco Bay

All traffic, both freight and passenger, entering San Francisco from the east has to be transferred across San Francisco Bay, a body of water nearly five miles in width. The cities of Oakland, Alameda, and Berkeley, situated on the east side of the bay, contain the residences of many thousands who work in San Francisco. The steady growth of traffic across the bay has caused the agitation of plans for constructing bridges or tunnels to give direct passage across the bay. On account of possible interference with navigation, the construction of such a bridge requires the approval of the War Department.

A public hearing was held at San Francisco Aug. 15 and 16 and in Oakland on Aug. 17 by a Board of Engineers made up of Col. Thomas H. Rees of San Francisco, Col. Charles L. Porter of Portland, and Major George B. Pillsbury of Los Angeles. Three plans for bridging the bay were presented. The first was offered by a firm known as the Associated Bridge Engineers, composed of Wilbur J. Watson & Co. of Cleveland, Wm. Russell Davis of Albany, and Harlan D. Miller of Albany, N. Y., and Oakland, Calif. A photograph of the design submitted is reproduced herewith. These engineers propose to cross the bay at a point farther south than the bridge sites hitherto considered. Practically all the piers on the San Francisco water front used by ocean shipping are located north of the line of the bridge so that the interference with navigation would be a minimum. The bridge is therefore placed at a comparatively low level, the vertical clearance above mean high water being 20 to 90 ft. Near the San Francisco shore there are two 575 ft. spans with a vertical clearance of 115 ft. To admit the passage of occasional vessels which might require even a greater height, a drawspan is located about one mile from the San Francisco shore having a length of 250 ft. This could be of either the bascule or the direct-lift type, and in the latter case a clear height of 200 ft. would be afforded. A similar provision would be made for the benefit of navigation where the bridge crosses the Oakland channel. The span here would have a vertical clearance of at least 60 ft. above high water when the bridge was closed and at least 200 ft. with the bridge open.



"ASSOCIATED BRIDGE ENGINEERS'" BRIDGE ACROSS SAN FRANCISCO BAY

The total length of the structure would be approximately 30,200 ft., of which 17,400 ft. would be of steel spans. There would be sixty steel spans 250 ft. long and twenty-three 100 to 200 ft. long. As designed, the bridge would have a double deck, the upper one carrying two roadways for slow moving vehicles and a double-track roadway for high speed automobiles. The lower deck would carry two railway tracks for overland trains and two railroad tracks for interurban trains.

On the eastern side of the bay the water is very shallow for a long distance out from the shore. The present terminals on the eastern side of the bay for the ferries running to San Francisco are located between one and two miles out from the mainland, being carried partly by a fill and party by pile trestles across the mud flats and shallow water. The Associated Bridge Engineers base their estimates for the bridge on pier foundations made by driving piles, cutting them off below the clay line so that they would be in no danger from the teredo, sinking interlocking cellular caissons of reinforced concrete upon the cluster of piles and filling the chambers of the caissons with concrete deposited through a tremie. Where the spans are placed high above the water this construction would be varied by constructing steel towers on piers carried 10 ft. or more above high water.

The estimated cost of construction of the bridge is $18,700,000, and the estimated cost of land damages, interest during construction, engineering and incidentals is $5,200,000 making a total of $23,900,000.

The plan for financing the Associated Bridge Engineers' project is to organize a bridge assessment district, made up of San Francisco and the cities on the western side of the bay which would assume responsibility for the cost of construction. It is estimated that by charging a toll of 2½c. per passenger and suitable rates for trains, automobiles, etc., the bridge would earn an annual revenue sufficient to meet operating expenses, maintenance, interest and sinking fund charges.

Plans for a long span cantilever bridge were presented by Charles E. Fowler of Seattle. The site on which he proposed a bridge is from Telegraph Hill in San Francisco to Goat Island, and thence to Oakland. He proposed a clear height of 150 ft. and steel spans of a total length of 2,950 ft., with one cantilever span 800 ft. long. The estimate of cost was $75,000,000. Mr. Fowler proposed that a corporation be organized to build the bridge and operate trains over it in connection with the operation of the street railways on the east side of the bay. Plans for a suspension bridge with 2,000 ft. span, and a height of 180 ft., were presented by Alan C. Rush. The estimate of cost was $32,000,000.

On the following day plans for tunnels under the bay were presented by Taggart Aston, Jerome Newman, John G. Little, and Carl F. Reuter. A serious objection to the tunnel plan is that a tunnel would not be satisfactory for automobile traffic across the bay, which is very heavy, constantly increasing, and which will stand a high rate of toll and therefore be very profitable for a bridge.