Volume XXXII Part I

Transactions

of

The Engineering Institute of Canada

(Incorporated 1887
as
The Canadian Society of Civil Engineers)

being a
Series of Papers Presented to the Institute
in 1917, 1918 and 1919
descriptive of

THE QUEBEC BRIDGE

Montreal
1919

PREFACE

THIS volume of the Transactions of The Engineering Institute of Canada consists of a series of papers descriptive of the Quebec Bridge, presented during the years 1917, 1918 and 1919. These papers are being published together, as it is felt by the Council that it is advantageous to have the full description of this important structure contained within one volume.

The Quebec Bridge has been fully described in illustrated addresses and in papers before Headquarters in Montreal as well as before the great majority of the Branches.

On November 22nd, 1917, Mr. C. N. Monsarrat, M.E.I.C., Past Vice-President, gave an illustrated address at Headquarters, dealing generally with the whole structure and particularly with the details of the substructure. The text of this address is incorporated in the present volume on pages 1 to 16 inclusive.

On December 6th, 1917, Mr. George F. Porter, M.E.I.C., delivered a comprehensive, illustrated address at Headquarters, on the superstructure of the Bridge and its erection.

On December 20th, 1917, Mr. Phelps Johnson, M.E.I.C., Past President, presented before Headquarters a series of views descriptive of the shopwork of the superstructure.

On January 10th, 1918, Mr. G. H. Duggan, M.E.I.C., Past President, read an exhaustive discussion on the design of the Bridge before Headquarters, which is to be found on pages 17 to 62 of this volume.

On April 17th and April 24th, 1919, there was presented before the Montreal Branch of the Institute an exhaustive paper by Mr. Johnson, Mr. Duggan and Mr. Porter, describing the design, the manufacture and the erection of the superstructure, to be found on pages 63 to 162 of the present volume.

In addition to the above presentations, in 1917 Mr. Monsarrat delivered his address before the Ottawa Branch and the Toronto Branch, while during the Spring of 1918 Mr. Porter delivered his address before all the Branches from Winnipeg West, as well as at the Professional Meeting in Halifax.

Montreal, May 1919. The Editor

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The Engineering Institute of Canada

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The Canadian Society of Civil Engineers

THE SUBSTRUCTURE
of
THE QUEBEC BRIDGE

by

C. N. MONSARRAT, M.E.I.C.

IN August 1908, a Board of Engineers was appointed by the Dominion Government to prepare a new design, specifications, and working drawings, and to supervise the rebuilding of the Quebec Bridge across the St. Lawrence, following the failure of the partly erected superstructure on August 29th, 1907 This Board was composed of a Canadian, H. E. Vautelet, M.E.I.C., former Asst. Chief Engineer, C. P. Ry., Montreal, as Chairman and Chief Engineer; an engineer from Great Britain, Maurice Fitzmaurice, C.M.G., M.E.I.C., Chief Engineer of the London County Council, who had been on the staff of the Forth Bridge; and an American, Ralph Modjeski, M.E.I.C., Consulting Engineer of Chicago.

One of the first steps to be taken in connection with this work was the removal of the debris of the fallen structure which filled the space between the anchor and main piers on the south side of the river and extended into deep water north of the main pier. Some 9,000 tons of this material had to be cut up and removed, the work extending over a period of two years. The oxy-acetylene torch and dynamite played an equal part in the cutting of this material, and no great difficulty was experienced, except just south of the main pier, where some of the larger members were driven some 10 to 15 feet into the bed of the river.

The possibility of utilizing the old masonry for the new bridge was a matter that required considerable study and investigation. As the Board's studies of the superstructure developed, it was found that it would be desirable to widen the bridge from 67 to 88 ft. centre to centre of trusses. In order to accommodate the necessary increase in live load and also the additional weight of steel required for the new design of this structure, it was found that the reactions on the piers would be nearly 100% greater than those of the original structure. As a consequence, the old piers as they existed, would not be large enough to accommodate the new bridge.

It was considered impracticable to enlarge or reinforce the north main pier owing to the fact that it was comparatively shallow and a reinforced pier would not lend itself to a proper distribution of the loads coming upon it.

The south main pier, being considerably deeper, offered a better opportunity for enlargement. A scheme of reconstruction was worked out by which the old pier was to be taken down to the top of the original caisson and two new caissons 30 ft. wide were to be sunk, one along the south side and the other across the West end, and a new pier built distributed over the old and two new caissons. In order to properly distribute the reactions from the superstructure it would be necessary to move the centre of the pier 15 ft. to the west and south.

On the north shore it would be necessary to build an entirely new pier which was to be placed 57 ft. south of the existing pier, or towards the river. The difference between the final location of these two piers would result in reducing the length of the main span from 1800 ft. to 1758 ft., centre to centre of piers.

Before entering into a contract for the construction of the substructure, a series of borings were made at and about the location of the main and anchor piers, which were driven to bed rock in each case and extending 15 ft. into solid rock in order to make sure that it was bed rock rather than a boulder that had been struck. These borings showed that bed rock would be encountered at approximately the same elevation on both sides of the river, at a depth of about 101 ft. below extreme high water, or about 85 ft. below the bed of the river at the location of the main piers. The formation of the bed of the river on the two sides, however, was entirely different. On the north shore heavy boulder formation was encountered for the entire depth, while on the south shore sand was encountered with only a sprinkling of boulders at various depths.

The contract for the substructure was awarded to Messrs. M. P. & J. T. Davis, of Quebec, in February, 1910, and work was started immediately.

The caisson for the north main pier was built at Sillery, about three miles below the bridge site, and was 180 ft. long and 55 ft. wide. It was constructed of 12″ x 12″ Southern Pine with a cutting edge of the same material 30″ square. The caisson had a working, or air, chamber under the roof 8 ft. high in the clear and divided by longitudinal and transverse bulkheads into eighteen compartments. It was built on launchways laid with a 10% grade leading out into deep water, and when the walls of the caisson had been built up about 40 ft. it was lowered into its inclined position by means of jacks. When ready for launching an impetus was given by means of horizontal jacks at the rear as well as vertical jacks to reduce the friction on the launchways, the whole operation working smoothly and without mishap.

The caisson was towed to the bridge site August 1, 1910, and, without difficulty, was placed in its proper position. It was held in place against the heavy current by steel cables attached to "dead men" about 500 ft. up and down the river. These wire ropes were supported by floats about 100 ft. apart, and were provided with a tackle to take up the slack and keep the caisson in its correct position at all stages of the tide.

When concreting started the caisson was drawing about 19 ft. of water, and weighed about 2,000 tons. The caisson first began to touch bottom at low tide after some 2,000 cubic yards of concrete had been deposited. It had been leaking to a certain extent but was easily kept dry by means of two steam pumps installed on the top of the caisson. At this time, however, an accident happened to the boiler equipment, and before it could be repaired, the caisson had filled with water, causing it to ground on the uneven bottom. As a result, the caisson was seriously strained, the seams opening up to such an extent that it was impossible to keep air in the working chamber. It was, therefore, decided that it would be necessary to remove the concrete from the caisson and tow it to the dry-dock at Levis and attempt to repair it during the coming winter.

This accident emphasized the difficulties of landing such a large caisson and there was an entire reconsideration of the design of the caissons and masonry, resulting in the abandonment of the scheme of enlarging the foundations of the old south pier. Having in mind the difficulties already encountered on the north shore, it was considered advisable to use the repaired caisson on the south shore where sinking operations would be much easier owing to the sandy formation of the river bottom. This entailed the sinking of an entirely new pier south of the old location. On the north shore it was considered advisable, in view of the experience gained, to sink two smaller caissons with a 10 ft. space between the final location of both caissons being fixed 65 ft. south of the existing masonry, thus restoring the length of the main span to 1800 ft., the same as that of the original bridge, and keeping the centre line coincident with that of the old structure.

An examination of the large caisson, or caisson No. 1, showed that the longitudinal joints had opened up near the centre from 1 to 2 inches, and that the total length of the caisson had increased about 5½ inches. During the fall of 1910 and spring of 1911, this caisson was thoroughly overhauled and repaired, the seams being caulked and the outside of the caisson and roof of the working chamber re-sheathed. On May 28th, 1911, it was floated out and towed up the river a distance of about nine miles to the site on the south side, which, being exposed at low water, had been carefully prepared and levelled off. Before leaving dry-dock the caisson had been provided with a false bottom which reduced the draught from 19 to 11 feet. As there was 15 ft. of water over the site of the pier at extreme high water, the caisson was floated in and placed in exact position for sinking without serious difficulty. A portion of the false bottom was then removed, and the various shafts were left unobstructed in order that the rise and fall of the tide would not lift it from its bed. No further work was done on this caisson during the season of 1911, the contractor directing all his efforts towards the sinking of the caissons on the north side of the river.

Caissons No. 2 and No. 3, for the north main pier, were constructed at Sillery, at the same location as caisson No. 1, the same details of construction being followed throughout. Each of these caissons was 85 ft. long and 60 ft. wide. Caisson No. 2 was towed to the bridge site on June 10, 1911, and caisson No. 3 on July 11 of the same year. In the sinking of these two caissons the contractor did not meet the same difficulty as experienced with caisson No. 1. The walls and roof of the working chamber were tight, preventing the leakage of water and escape of air. Some difficulty was, however, encountered in sinking owing to large boulders fouling the cutting edge, which resulted in this edge being forced inward from 6″ to 10″ in both caissons. In order to prevent further difficulties of this kind, which might seriously handicap sinking operations, the method of sinking was changed.

Timber blocking was placed beneath the bulkheads, after which a trench was excavated all around and below the cutting edge, allowing the full weight of the caisson to be carried on this blocking. This trench was excavated to a depth of about 2 ft., after which it was filled with blue clay in bags. When all was ready the blocking was underscoured with water jets and the caisson lowered on a cushion of clay, which acted as a lubricant and also prevented considerable air leakage. This method was found to work very advantageously and further damage to the cutting edge was prevented.

It was the original intention to sink both these caissons to rock, but, as the work progressed, the sinking became more difficult owing to the heavy boulder formation, and finally, when the caisson had reached elevation 20.0, the Board decided that the foundations at this point should be quite satisfactory for the loads which they would be called upon to support. Bearing tests were made and it was found that a load of 59 tons per sq. ft. showed a settlement of 1/8 in., practically no settlement being noted at 20 tons per sq. ft. As the average working load at the foot of this pier was only eight tons, the Board considered that there would be no justification in carrying the foundations to a lower level. Caisson No. 2 was sunk a total distance of 41.6 ft. at an average rate of 4.9 in. per day. Caisson No. 3 was sunk 38.3 ft. with an average rate of 5-3/8 in. per day.

After the caissons had reached their final location the working chambers were filled with concrete composed of one part of cement, two and a half parts of sand and four parts of small crushed stone. This concrete was made much drier than the concrete used in the main caisson, it being found that concrete deposited under compressed air gave better results when very dry than in a more or less liquid state.

Concrete was deposited in terraces, the men working towards the centre from the sides and ends. Great care was taken to ram the concrete thoroughly round the roof timbers so that a bearing would be assured under the roof of the working chamber. After the working chamber was filled as carefully as possible by hand the shafts were filled with concrete. As a still further precaution, a rich grout was forced in through the 4 in. blow pipes by compressed air under a pressure of 100 lbs. per sq. inch. After caissons No. 2 and No. 3 had reached their final depth, the material in the 10 ft. space between them was excavated to a depth of 38 ft. below high water this space being then enclosed by means of timbers connected to the outside walls of both caissons. The area between the caissons was then filled with concrete up to a point 7 ft. below low water. The water was then pumped out and six steel girders 6 ft. deep were placed through the ends of the caissons and into cavities left in the adjacent concrete. The entire space was then filled with concrete in which the steel girders were embedded, thus forming a monolith upon which the masonry shaft of the pier could be built.

The sinking of the large caisson for the south main pier was started July 28th, 1912, and was completed October 24th, 1912, or at the average rate of 0.75 feet per day during the entire period. The material encountered at this point was, as indicated by the borings, chiefly sand, and the caisson was carried down to rock, which was reached at 101 ft. below high water or 86 ft. below the bed of the river.

The difficulty experienced on the north side in keeping the cutting edge intact and also the fact that the caisson had previously been overstrained, led the contractors to take unusual precautions to prevent the possibility of any accident happening to the caisson during the sinking operations. For this reason, special appliances were devised for relieving the cutting edge from carrying all the load, and by the use of sand jacks the total weight of the caisson was distributed over the entire bottom area. The manner of using these sand-jacks was one of the most interesting features connected with the sinking of this caisson, and possibly merits special description.

The jacks themselves were of very simple construction. The cylinders had an internal diameter of 31 inches, and were 36 inches long, constructed of ¼ inch steel plate with 4-inch lap joint; two angles 1½″ x 1½″ x ¼″ reinforced the cylinder at top and bottom. The piston was a block of yellow pine 2′ 6″ square and 5 ft. long. Four feet at one end was round with a diameter of 29 in., thereby allowing 1 inch play in the cylinder. The lower end of the piston was reinforced by a 2½″ x ⅜″ welded iron band. During operation the piston was attached rigidly to the roof of the working chamber by long screw bolts, and remained there permanently during the entire period of sinking.

In preparing for a drop, the first step was to excavate a hole under the piston. The cylinder was filled about two-thirds full of sand and placed in position under the piston and blocked up hard against it by means of timbers. While this was being done the caisson was supported on timber blocking under the bulkheads and other points. At the bottom of the sand jack was a 2″ iron pipe extending entirely across the cylinder, the side of which was split and opened up to allow the sand to escape. This type had no bottom to the cylinder, the timbers acting as a support for the sand. Another type used had a steel bottom and two 3″ holes with sliding cover at each side at the foot of the cylinder. The operation in both cases was the same.

When everything was ready for a drop, the timber blocking supporting the caisson was undermined by a water jet and the full load taken by the sand jacks. A man was stationed at every jack, and at a given signal, afforded by the flashing of the electric lights, each man turned a hydraulic jet with 60 lbs. pressure into the hole at the bottom of the cylinder, thus washing the sand out. The sand was caught in canvas bags of uniform size. When the canvas bag was full the lights flashed again and the water jet was turned off. Another bag was then obtained, and at the signal the jet was again turned on and the bags filled. Each cylinder contained in the neighborhood of 16 bags of sand, and this operation was continued until the required settlement was obtained. By adopting the signal system and emptying the sand into bags, it was possible to ensure that the whole caisson was being sunk at a uniform rate, and that there was no reasonable possibility of any part of the caisson being strained by being sunk more rapidly than another portion. As a rule a drop of from 18 in. to 2 feet could be effected at each operation, the recurrence of the operations depending entirely on the nature of the material to be removed. When the drop had been finished the blocking was again placed under the bulkheads to take the load of the caisson, and the holes under the sand jacks deepened in order that the operation might be repeated. The greater part of the material excavated in this caisson, being sand, was forced out through blow pipes.

Practically no problems were encountered in the construction of the north and south anchor piers and the north intermediate pier. Both anchor piers were constructed south of the existing anchor piers. For the north anchor pier a coffer dam had to be constructed around the foundations since the foot of the pier was below high water mark. The south anchor pier was well above high water mark so that all excavation was in the dry.

The anchorage girders were embedded in concrete in the base of these piers and the first length of anchorage eyebars connected thereto. Above the top of these eyebars a shaft was left at each end of the anchor pier to allow the remaining anchorage eyebars to be connected at the proper time. When the full dead load stresses were applied to these anchorage bars, the shafts were filled with concrete to the top of the first length of eyebars.

For the main piers entirely new stone was used, but for the anchor and intermediate piers the specification allowed the use of the stone from the old masonry. The greater portion of the old stone was consequently used in the construction of these piers. The abutments were not radically changed, it being only necessary to raise the ballast walls and make minor alterations to suit the new design.

The masonry in the pier shaft consisted of grey granite rock faced ashlar obtained from quarries situated about 60 miles north of the bridge site. This facing was laid with alternate headers and stretchers and backed with concrete in which were embedded displacer stones having an approximate volume of about one cubic yard. Headers were required to have a length of at least 2½ times their breadth with a minimum length of 7 ft. Bed joints were ½ inch throughout and vertical joints ⅜ in. for a distance of 12 in. back from the face and not exceeding 4 in. at any point. All face stones of the top course were doweled to the second course with two 1¼ in. dia. steel dowels to each stone and extending 6 in. into each course. All face stones of the second and third courses were clamped together horizontally with steel clamps 16″ in length x 1″, with a 3 in. vertical projection at each end, and were also doweled to the course below, as described above. All face stones of the rounded end of the piers below the third course were both clamped and doweled. The upper 18 ft. of the main piers were built with cut granite backing and about 40% of these backing stones were made to project up into the course above in order to give a good horizontal bond.

The concrete used in the caisson and the backing of the piers was 1 of cement, 2½ sand and 5 parts broken stone by volume, except the concrete in the working chamber, which was 1 of cement, 2½ sand, and 4 parts broken stone. The cement was required to pass a tensile test for neat cement of 450 and 540 lbs. for 7 and 28 days respectively, and for 1 part of cement and 3 parts of sand, 140 and 220 lbs., respectively.

The power plant, dining-room for "sand-hogs" and a two-storey bunk house were located at the water's edge, at the foot of the cliff, as were also the mechanical plants which furnished the power for the various operations. All supplies and materials were delivered at the top of the cliff, some 160 ft. above high water level, and thence by gravity through chutes or elevator to the concrete plant and service tracks below.

The air compressors discharged into a 12-inch main from which 7-inch branches with flexible rubber connections led into the two caissons, each branch being fitted with a gate valve so that the air could be cut out of either caisson at will. The main 12-inch pipe was about 500 ft. long and was laid in a sluice of running water in order to keep the temperature of the air down to about 75 degrees Fah. As a consequence the temperature in the working chamber rarely exceeded 90 degrees Fah., although in the service shaft and man-locks the temperature exceeded 100 degrees Fah.

The concrete mixing plant was placed just at the foot of the cliff. Half-way up the slope was the rock crushing plant. The rock used for the concrete was obtained from an adjoining cut and was brought to the brow of the hill in cars which dumped into a chute leading to the crusher plant. The stone was fed into two gyratory crushers which were capable of dealing with about 500 cubic yards in twelve hours. After passing through the crushers the stone was led over an inclined screen of 2-in. mesh, and thence into a storage hopper bin of about 200 yards capacity. Three chutes led from this to the concrete mixing platform below, the mouth of each chute being directly over a mixer. From this platform the sand, stone, cement and water, were fed in the proper proportions to the mixers underneath the platform, which in turn dumped into self-discharging buckets on trucks, which were hauled to the caissons by horses. Three mixers were used on the work, two having a capacity of 2/3 cubic yards and the other 1-1/3 cubic yards. Owing to the conditions under which the work was carried on the mixers never had a chance to work to their full capacity; their best day's work being 450 cubic yards for the 24 hours.

The sand and coal were delivered from the upper level to the lower through chutes. On top of the coal chute was a double line of rails with balanced trucks which conveyed the cement from cars at the upper level to the storage shed at the level of the concrete mixing platform.

As the "sand-hogs" employed on the excavation in the working chamber of the caisson were compelled to work in shifts during the whole 24 hours, sleeping and dining accommodation was provided on the lower level capable of accommodating about 100 men. Similar accommodation was provided on the upper level for about half this number.

On the dock the contractor erected a number of buildings, which included an office and bath room for the accommodation of the inspectors and a hospital with a doctor in continual attendance where first aid might be administered in case of serious accident, or regular treatment in case of minor troubles. There was also provided a coffee-house, kept at a high temperature, where the "sand-hogs" could change their clothes and receive hot coffee at the end of their shift in the working chamber. In addition to the above there were the usual stores, offices, etc., for the contractor's own use. In connection with the hospital arrangements there was also provided a steel hospital lock connected with the compressed air system, to which men suffering from the "bends" could be immediately transferred and treated.

For serving each caisson, four 30-inch shafts for material and two 30-inch ladder shafts were employed. For ejecting the sand and smaller stones, four 4-inch blow pipes were used. The larger boulders were broken up and hoisted through the material shaft in buckets having a capacity of 2/3 cubic yards. Four 7-in. compressed air pipes supplied air to the working chamber and served the blow pipes. Two 6-inch pipes supplied the water for "washing" the sand. One 2-inch pipe supplied high-pressure air for drilling, etc., and a second 2-inch pipe carried the wires for the electric lighting of the working chamber and ladder shaft.

As soon as the sinking was completed on the north shore as much of the plant as could be spared was moved to the south side. The men's dressing rooms and sleeping quarters were placed on skids, launched into the river, floated across, and placed in position on the other side. The layout for the mixing plant, sand chute, coal chute, etc., was practically the same as on the north side of the river, all the materials being led to the lower level by gravity. The stone for the crushers was quarried directly from the top of the cliff so that one derrick could pick up the stone in the quarry and deposit it in the hopper leading to the crushing plant half-way down the cliff.

The mechanical plants used on this side of the river were much the same as on the north side, although increased about 50%. Water at 100 lbs. pressure was led into the working chamber and distributed around all four sides in a horizontal main from 4 in. to 6 in. in diameter. Each of the 18 compartments was provided with a valved outlet and jet pipe with 1 in. nozzle which was used to loosen the sand and gravel and facilitate excavation. Each chamber was also provided with a 6-inch vertical blow-out pipe and lighted throughout with electricity.

The caisson was fitted with six 3-ft. material shafts, each having a Moran air lock, and with four 3-ft. ladder shafts having simple air locks composed of short upper sections with top and bottom diaphragms. In order to enable the maximum number of men to leave the caisson at one time, a 6-ft. horizontal steel man-lock, about 30 ft. long, was located on the deck of the caisson and built permanently into the solid concrete of the pier. This man-lock was reached from above by means of a 4-ft. vertical stair shaft. This lock was large enough to accommodate 30 or 40 men at once, thus greatly expediting the entrance and exit of each successive shift and effecting an economy of air consumption. When the men wished to leave the working chamber they assembled in this air lock and the air pressure was very gradually reduced until normal pressure was reached. When nearing the final stage the average time that elapsed from the moment of entering the man-lock until the pressure was sufficiently reduced to enable the men to pass into the outer air was about 35 minutes. The same process was repeated on entering the working chamber, although this could be done much more rapidly, the total length of time required being only about five minutes.

A hospital lock was also established on the shore. If a man should pass from the working chamber to the outer air too quickly he was affected by what is known as the "bends." When a man was so affected, he was admitted to the steel hospital lock and pressure turned on until the pain ceased, after which the pressure was very gradually reduced until normal pressure was reached. Under moderate pressures 100 men were employed in eight-hour shifts. As the caisson went down and the pressures increased, the lengths of the shifts were diminished gradually until the minimum of two 1-hour shifts in the 24 was reached. As the length of the shifts decreased the rate of pay of the men increased.

The total amount of masonry included in this contract is as follows:—

North Abutment (Alterations)	404.5	cubic	yards.
North Intermediate Pier	1665.6	"	"
North Anchor Pier	17736.	"	"
North Main Pier	31870.4	"	"
South Main Pier	38279.4	"	"
South Anchor Pier	16073.	"	"
South Abutment (Alterations)	61.1	"	"
Total	106,090.0	"	"

After the piers had been constructed and pointed they were all thoroughly cleaned by sand blast. The work of dressing the bridge seats, especially on the two main piers, was very important as it was necessary to ensure an absolutely level bearing for the large steel shoes carrying the structure. In order to provide datum points for the guidance of the stone cutters in dressing these bridge seats, grooves were first cut along the edges of each dress stone and points established at each intersection which were absolutely true in elevation. The stone cutters then cut away the stone between these points. When they got close to the finishing cut the surface was gauged by means of long steel straight edges manufactured from small eyebeams. The bottom surface of this straight edge was coated with paint and swept over the surface. The high spots on the masonry, therefore was indicated by the paint scraped off the straight edge. These were further dressed and the process continued until the whole surface was absolutely level. As far as can be judged this bed was finished with a variation of not more than 1/100 of an inch at any point. About six weeks steady work with stone cutting machines and masons was required on each of the four bridge seats of the main piers.

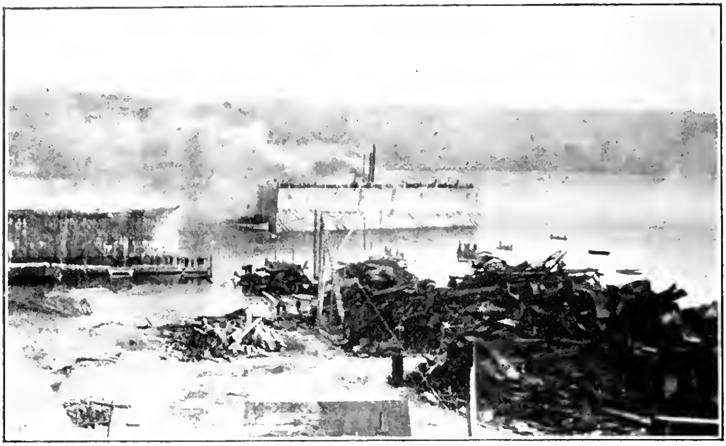
Owing to the great length of span it was impossible to make actual measurements across the river for the location of the piers. Base lines over 1300 ft. long were established on each side of the river, and by an elaborate system of triangulation the piers were located. After the main piers had been constructed a further series of triangulations was made, and it was found that the centre to centre distance was 1800 ft. 2-7/16 inches)—about 2-7/16 inches more than originally planned. This extra distance was provided for in the design of the steel work. Actual measurements of the length centre to centre of main piers, made after the bridge was erected, checked the calculated lengths very closely.

S. H. Woodard, M.E.I.C., was engineer for the contractors and was responsible for the design of the caissons and for the many unusual features employed in sinking.

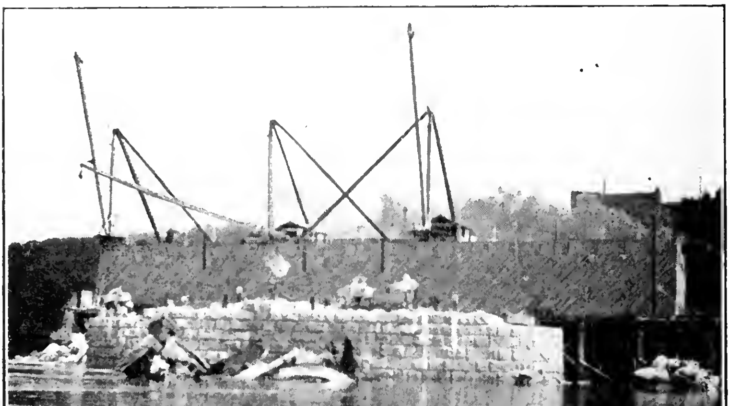
During the progress of the work, several changes occurred in the personnel of the Board, Mr. Fitzmaurice resigned in June, 1910, and was replaced by Charles MacDonald of Gananoque, Past President, Am. Soc. C.E. In February, 1911, Mr. Vautelet resigned. On April 4th, 1911, when the contract for the superstructure was signed, Mr. MacDonald, acting Chairman and Chief Engineer, retired, leaving Mr. Modjeski the sole remaining member of the Board as originally appointed. On May 1st, 1911, C. N. Monsarrat, M.E.I.C., Engineer of Bridges, Canadian Pacific Railway, was appointed to succeed Mr. Vautelet as Chairman and Chief Engineer, and on May 17th, 1911, C. C. Schneider, past President, Am. Soc. C.E., was appointed as third member, replacing Mr. MacDonald. On the death of Mr. Schneider in January, 1916, H. P. Borden, M.E.I.C., was appointed to succeed him.

The final report of the Board, in which a more detailed description of the substructure will appear, is in course of preparation and will be published at an early date.

C. N. Monsarrat, M.E.I.C.

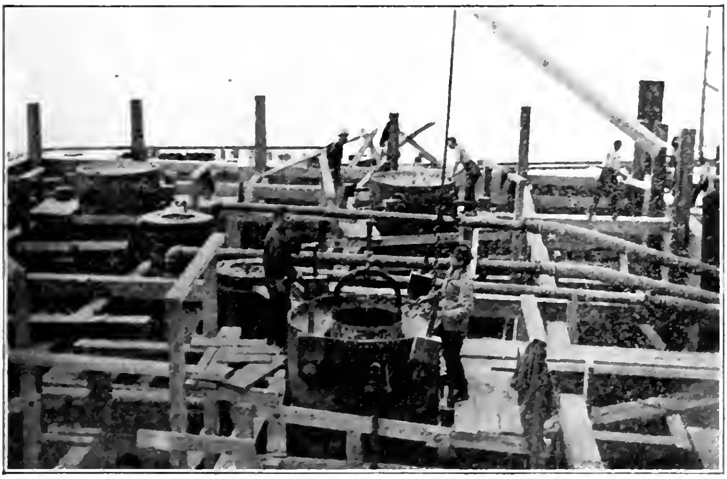


Floating Caisson No. 1 into position on South Shore after repairs had been made. Debris of old Structure on foreground.

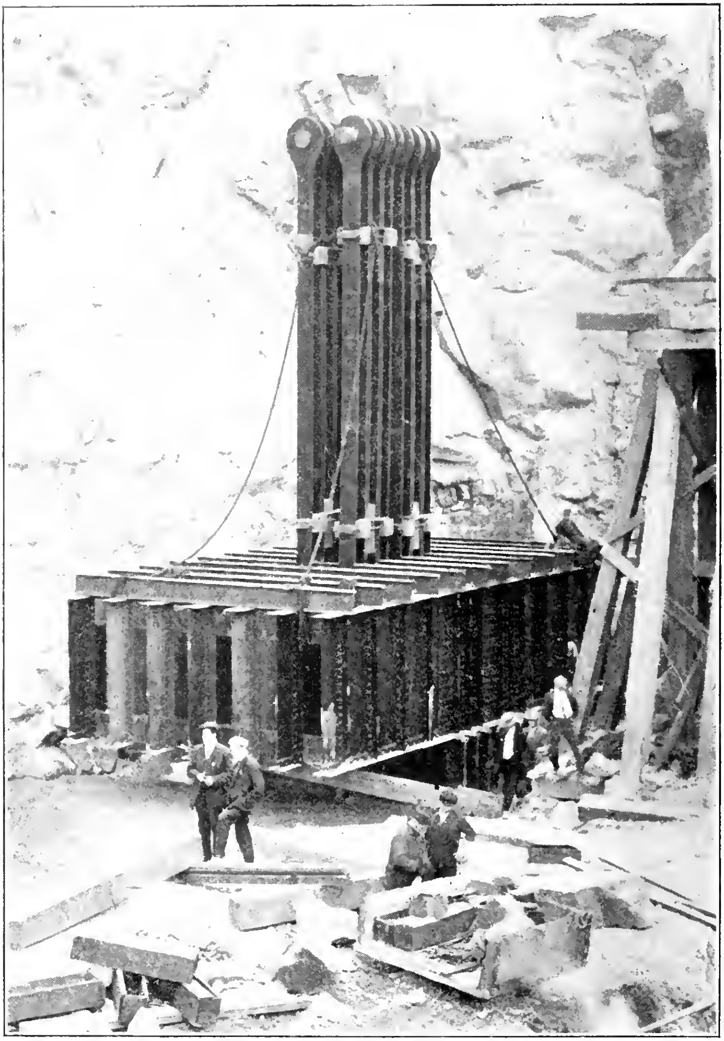


Caisson No. 1 in position behind old Main South Pier which is in course of demolition.

Smaller Caissons Nos. 2 and 3 ready for launching at Sillery.

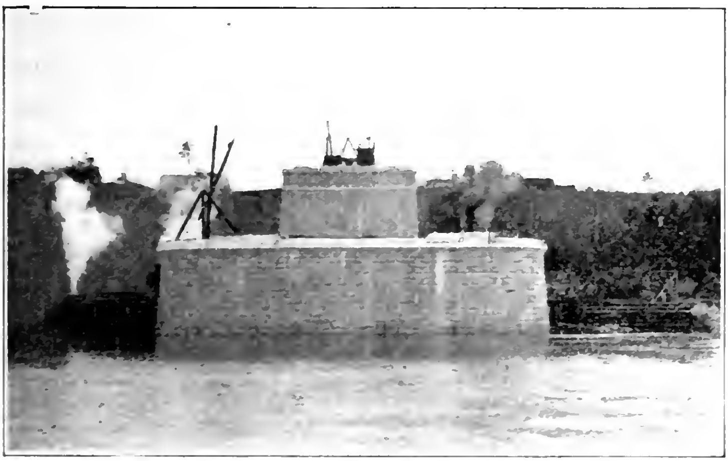


View showing Sinking of Caisson No. 2 and Operation of Material Locks.

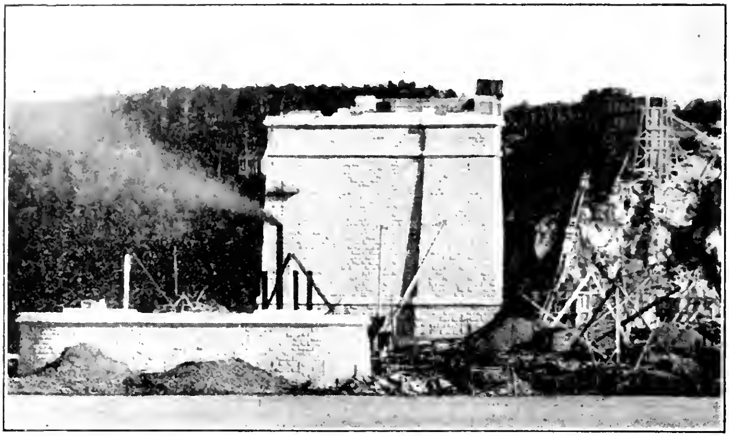


Steel Grillage and first length of Anchor Bars, South Anchor Pier.

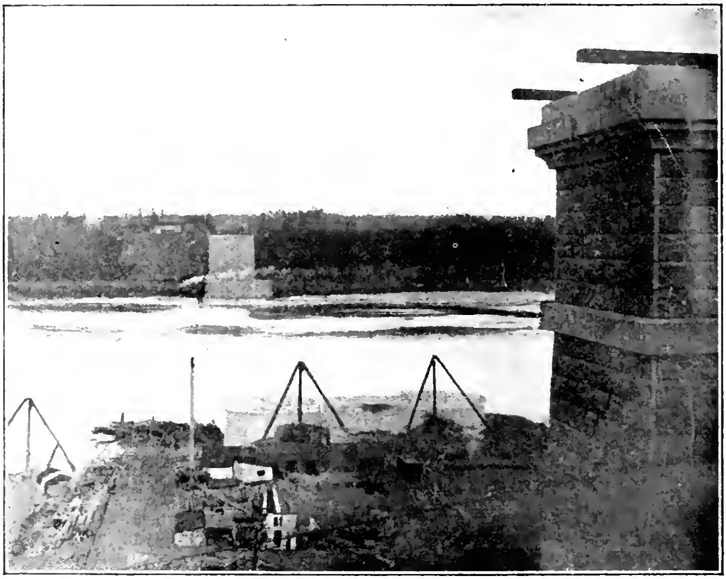
South Main Pier in Course of Construction. Note the Alternate Headers and Stretchers in Granite Face and Vertical Bond Stones projecting through the Courses.



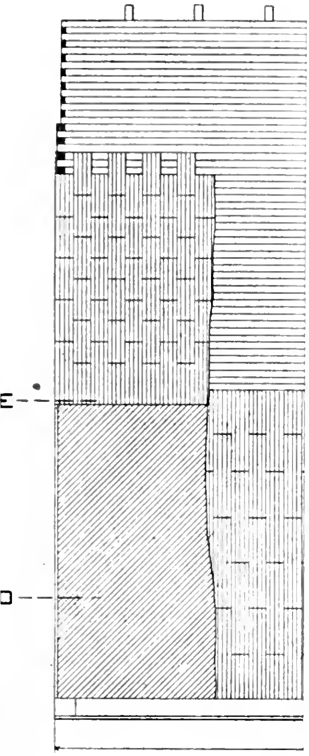
Completed Main and Anchor Piers in North Shore.



Complete Main and Anchor Piers on South Shore.



General View of Crossing Showing Completed Masonry.



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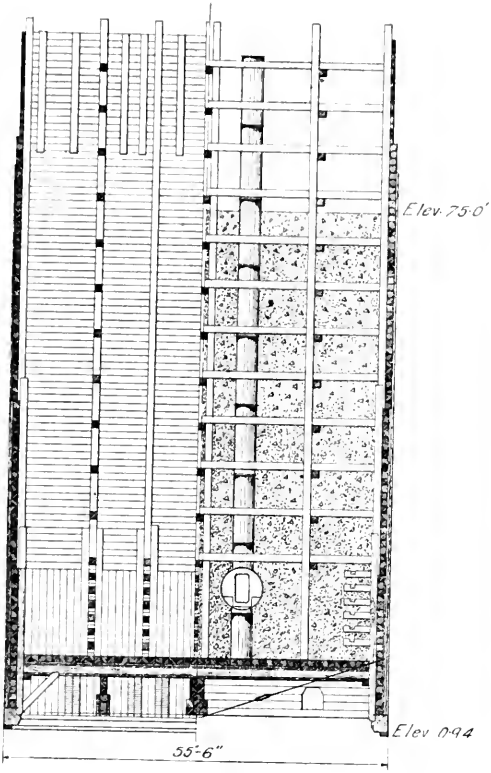

SECTION B–B

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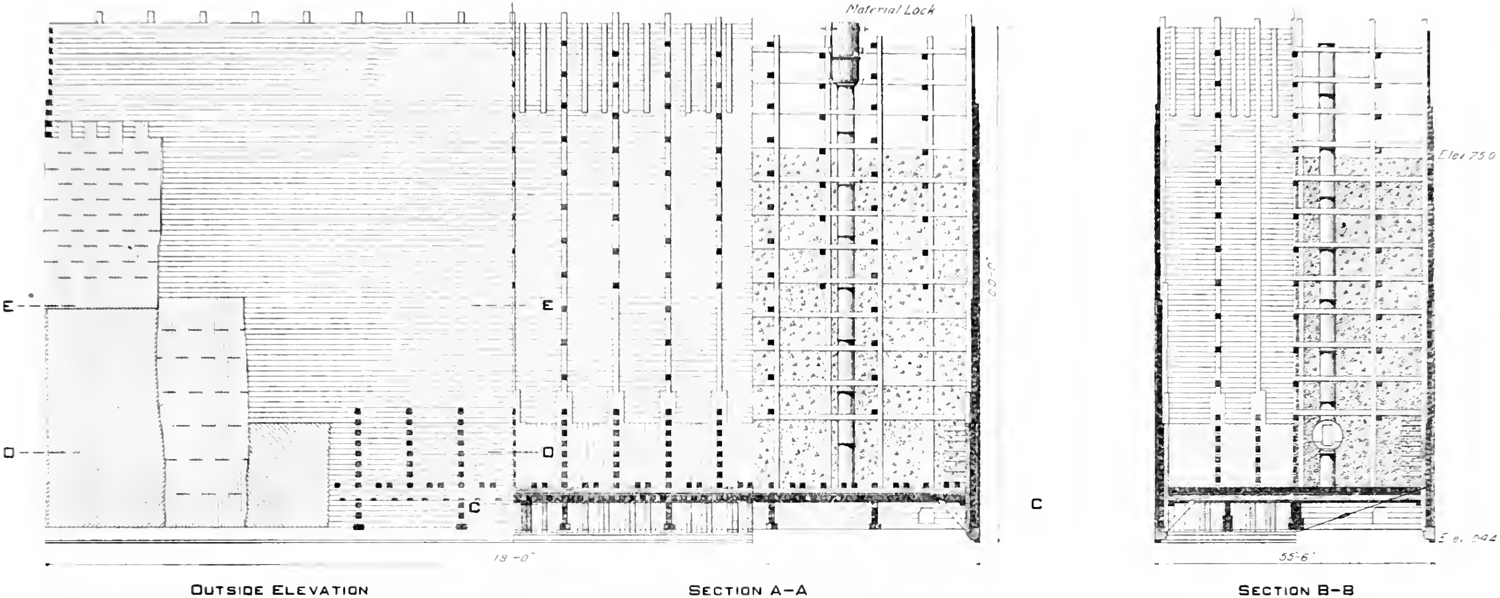
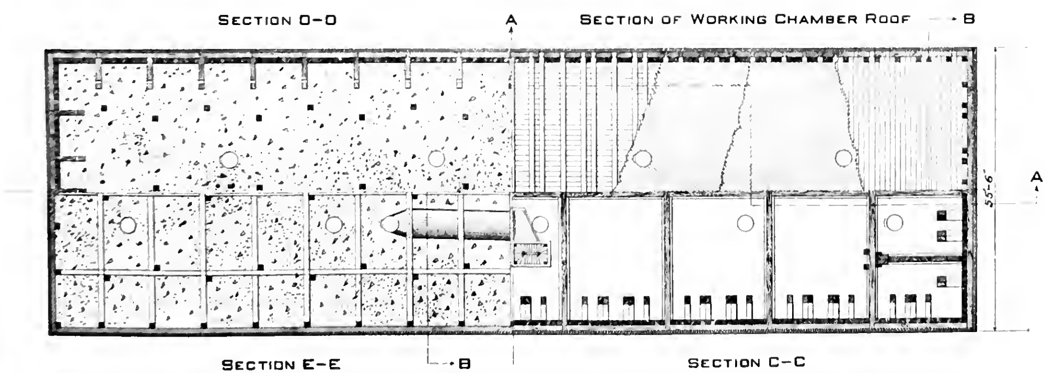
DETAILS OF CAISSON

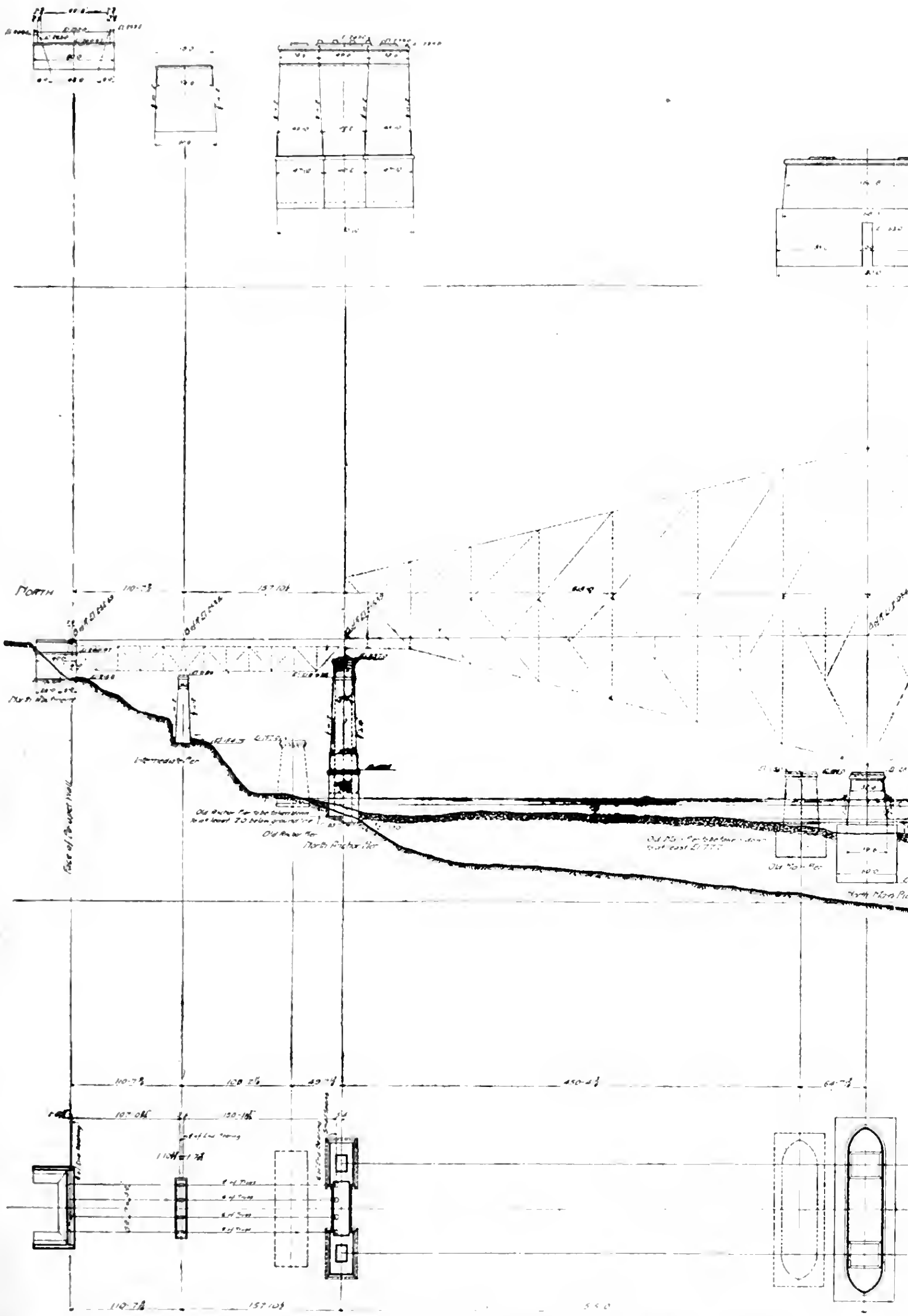
No. 1.—Originally Constructed for the North Main Pier but ultimately used on the South Shore.





DETAILS OF CAISSON

No. 1.—Originally Constructed for the North Main Pier but ultimately used on the South Shore.



General Diagram of Crossing showing location of Old and New Masonry

NOTES ON THE WORK OF THE ST. LAWRENCE BRIDGE COMPANY

in preparing

THE ACCEPTED DESIGN OF THE SUPERSTRUCTURE

of

THE QUEBEC BRIDGE

by

G. H. DUGGAN, M.E.I.C.

A PAPER describing the superstructure of the Quebec Bridge is in course of preparation by the Engineers of the St. Lawrence Bridge Company. It will however be some time before the paper is presented as many calculations must be abbreviated, and the working drawings, and many of the drawings for the construction equipment, as well as those issued for instruction to the shop and the erection force, must be condensed into forms that will come within the compass of an ordinary paper.

The paper is being written in order that the members of the Society may have for reference a technical description of the structure as built, and of the special plant and methods employed in its manufacture and erection, but it will not come within the scope of the paper to discuss the designs prepared for tender or the considerations that led to the adoption of the final design.

There are however some engineering considerations in connection with the preliminary designs and trial work leading up to the final design which, while beyond the scope of the paper, may be interesting to the members of the Society. Moreover, I feel that the Society should have some record of those who actively contributed to the success of the undertaking because so many engineers have been employed upon the work, and it has been so unusual, and of such long duration, that a mere statement of those in charge of various Departments at the finish of the work would omit the names and responsibilities of some who should be recorded in the history of the undertaking.

As it is probable that I have the most intimate knowledge of all phases of the work from the time our designs for tenders were started until its completion, the duty of giving this information and record seems to devolve upon me, and no better way of presenting it occurs to me than to tell the story of our work, with a discussion of our designs up to the point where the description will be taken up by the forth-coming paper.

A brief history of the preliminaries, and the award of the contract to the St. Lawrence Bridge Company, will assist in an understanding of the competition and the designs.

The project of bridging the St. Lawrence at Quebec is an old one, but its early history has no bearing on the present Quebec Bridge, and may be neglected. Tenders were called by the Quebec Bridge & Railway Company in 1899, which later resulted in the award of the contract to the Phoenix Bridge Company for the bridge which failed on August 29th, 1907.

Immediately after the accident the Dominion Government appointed a Royal Commission to report upon it, and after receiving the report, appointed a Board of Engineers to prepare a new design for the bridge. I was told by the then Minister of Railways and Canals that in appointing this Board it had been the desire of the Government to select Engineers best qualified by their experience in long span bridges to deal with this unusual problem, and to appoint one engineer from Canada, one from Great Britain, and one from the United States. After advisement he had appointed Mr. H. E. Vautelet, M. Can. Soc. C. E., for a long time Bridge Engineer of the Canadian Pacific Railway, as Chairman and Chief Engineer; Mr.(now Sir Maurice) FitzMaurice, C.M.G.,M.I.C.E., of London, England, who had been one of the engineers on the staff of the Forth Bridge; and from the United States, Mr. Ralph Modjeski, M. Am. Soc. C.E., who had been connected with many long span bridges.

The report of the Royal Commission appointed to investigate the failure of the Phoenix Bridge in 1907 is very comprehensive, and goes beyond the mere taking of evidence and the investigation of the faults of the bridge, as the Commission assembled most of the available data on other long span bridges, illustrated their important features, recorded the tests on large size compression members that had any bearing upon the work, and made a number of tests to supply some lacking experimental data of the behavior of large compression members under stress.

The Board of Engineers continued the investigations of the Royal Commission and made a number of trial designs. Mr. Vautelet selected one of these trial designs and brought it to the condition of a working design about the end of 1909. It was his intention to make this the Official Design on which the bridge was to be built, and to call tenders for the construction on it only. The other members of the Board did not consider the design to be in all respects a desirable one and consented to tenders being called upon it only on the condition that contractors would be allowed to submit tenders on their own designs if they so desired and, probably due to these disagreements, tenders were not actually advertised until the 17th June 1910. Before tenders were called and during the discussion on the design, Mr. FitzMaurice resigned from the Board. Mr. Chas. MacDonald, M. Can. Soc. C.E., a noted bridge builder, a Canadian by birth and permanent residence, but then retired from active practice and spending much of his time in the United States, consented to become a member of the Board until the Contract could be awarded.

After tenders were received, Mr. Vautelet still strongly contended for his design while his colleagues, Mr. Modjeski and Mr. MacDonald, favored the design of the St. Lawrence Bridge Company. To settle this dispute the Minister, as provided for in the original Order-in-Council, called in, with Mr. Vautelet's consent, two additional Engineers, Messrs. M. J. Butler, C.M.G., Past President, Can.Soc.C.E. and Henry Hodge, Am.Soc.C.E. of New York, to assist the Board in coming to a decision. Four engineers of the Advisory Board recommended the design of the St. Lawrence Bridge Company, Mr. Vautelet only dissenting. Mr. Vautelet resigned when his colleagues' recommendation was accepted about the end of February 1911.

Messrs. Butler and Hodge were appointed to the Board only to assist in deciding upon a design and specifications and their duties ceased with the signing of the Contract. Mr. MacDonald had also made it a condition that he should be relieved when the design was arranged and the contract awarded, and after the Contract was signed, on April 4th, 1911, Mr. Modjeski was the only member left of the Board. About a month later, Mr. C. N. Monsarrat, M.Can.Soc.C.E., was appointed to the position made vacant by Mr. Vautelet's resignation, and shortly after Mr. C. C. Schneider, Past President of the Am.Soc.C.E., joined the Board. Mr. Schneider was regarded as the Dean of Bridge Engineers in America and was a valued member of the Board until his death in 1916. When Mr. Schneider died, Mr. H. P. Borden, M.Can.Soc.C.E., who had been Secretary of the Board, was appointed to fill the vacancy.

Sometime before this Mr. Phelps Johnson, President of the Dominion Bridge Company, Past President of the Society, had arranged with Mr. F. C. McMath, M.Can.Soc.C.E., President of The Canadian Bridge Company, that in view of the magnitude of the work, and the interest of Canadians in making the work a Canadian enterprise, the Dominion and Canadian Bridge Companies should combine their forces in the organization of a special Company to tender on the bridge, and, if successful, to carry out its construction, each Company taking a half interest in the venture and contributing such of its staff as might be necessary to make an organization for the new Company. This Company was later incorporated as the St. Lawrence Bridge Company. Prior to this Mr. G. F. Porter, M. Can. Soc. C.E., Chief Draftsman of the Canadian Bridge Company, and Mr. P. L. Pratley, M. Can. Soc. C.E., of the Engineering Staff of the Dominion Bridge Company, had been released to the Board of Engineers to assist in the preparation of the official design.

The magnitude of the disaster to the bridge being erected by the Phoenix Bridge Company, with its lamentable loss of life and serious financial loss, coupled with the fact that the bridge was larger than anything that had heretofore been attempted and the probable very heavy cost of constructing the bridge in a proper manner, had caused serious misgivings in the minds of the public and the Government as to the practicability of the undertaking, and from the outset the Government had safeguarded itself in every possible way.

A prominent clause of the contract read as follows:

"The Contractor must satisfy himself as to the sufficiency and "suitability of the design, plans and specifications upon which the bridge "is to be built, as the Contractor will be required to guarantee the satisfactory "erection and completion of the bridge, and it is to be expressly understood "that he undertakes the entire responsibility not only for the materials "and construction of the bridge, but also for the design, calculations, "plans and specifications, and for the sufficiency of the bridge for the "loads therein specified. And the enforcement of any part, or all parts, "of the specifications shall not in any way relieve the Contractor from "such responsibility".

To implement the above guarantee the St. Lawrence Bridge Company was obliged to make a cash deposit of $1,297,500., and in addition both the Canadian and Dominion Bridge Companies signed guarantees for the completion of the bridge putting their whole assets at stake.

The discussion of the designs will be addressed to those who have not made special study of long span bridges, and it is hoped those who are conversant with the subject will forgive if taken over familiar ground, or if the subject seems to be treated in too elementary a manner.

Plate I. — Shows in outline the elevation of—

(1) The design of the Phoenix Bridge Company which failed in 1907;

(2) The Official Design when tenders were advertised, but afterwards known as Design 1, and supplemented by Designs up to V before tenders closed.

Plate II. — Shows in outline the elevation of the great cantilever bridges that had been built up to that time.

It will be seen that all existing bridges, except the Forth, were of so much less span than the Quebec Bridge that the chief guides or precedents for the problems in hand were the Forth Bridge, and the Phoenix Bridge which had failed. Many good features of the Forth Bridge were not applicable for reasons that will be given later.

The report of the Royal Commission which investigated the failure of the Phoenix Bridge, brought out clearly many faults of that design that could easily be corrected in a new design, some of these being the small width centre to centre of truss, the very high unit stresses, the curved compression chords, the inadequate lacing of compression members and the poor splice connections of the members.

The report also disclosed the use of open joints during erection. In our view the failure of the Phoenix Bridge may be ascribed chiefly to these open joints, and as open joints are difficult to avoid in designs requiring sub-divided main panels, it may be well to discuss this subject briefly.

Plate III. — Taken from the report of the Royal Commission, shows the deformation diagram of the anchor arm of the Phoenix truss, and the open joints in erection.

In all framed structures the lengths of the members as manufactured or before entering into the structure, differ from the lengths they will have in the completed structure, due to the elongation or compression, as the case may be, induced by the stresses to which they are subjected. It is customary to make provision for this in the "framed lengths" of the members, by calculating the amount of the extension or compression, and diminishing or increasing the length of the member so that it may have its goemetrical length in the structure after the stresses are imposed. Thus, if the members were assembled on their sides on a flat surface without any stresses in them, the whole truss would be considerably distorted from the form it would have when erected, as shown by the heavy lines of the drawing.

In cantilever erection the members are necessarily placed without load upon them and until the work has proceeded a considerable distance the truss will approximate to the form which it would adopt if laid down flat. As the load comes on, the truss is gradually brought toward the form it will finally have, when the weight of the centre span is attached to the end of the cantilever.

In certain forms of trusses with sub-divided panels, the "framed lengths" which will bring the truss to its proper form when under full load, cause some of the members to take a considerable bend during erection although they will eventually be straight when under load. As erection requirements cause these members to be put up in sections and then spliced at the joints, they cannot be assembled easily without allowing all the bend to come where they are spliced, thus causing a wedge shaped opening at the joint with the sections of the member only bearing at one edge. As the load comes on and the structure deflects the members straighten and the joints gradually close, but before the joint comes to a true bearing over its whole surface, the intensity of the pressure on one edge is very great, and may cause failure, as in the case of the Phœnix Bridge.

Diag. 2 of Plate I shows the original design prepared by Mr. Vautelet.

Plate IV. — Copied from the Official Drawing, shows the anchor arm to a larger scale.

Plate V. — Illustrates the cantilever arm and suspended span to the same scale.

The truss had a main web system of the single warren or triangular type, each truss panel being sub-divided to give a point of support for an intermediate floor beam, and thus make the floor system panels of moderate length. This sub-division was accomplished by means of a triangular frame suspended from the mid-heights of the main diagonals. This system of bracing overcame in large measure the local deformations and secondary stresses of the ordinary sub-divided panels, but it required an adjustment in the top chord of the triangular frame, as the framed length of this chord during erection would be different from its final length when the whole truss was completed and had taken its normal deflection.

Figures 1-1a-1b-1c show in outline the progressive erection of the cantilever arm of the Official Design.

Mr. Vautelet had worked out two schemes of erection for this bridge, the two schemes differing only in the method of erecting the anchor arm.

In the first it was intended to erect the anchor arm complete on false-work; while in the second it was intended to place a temporary supporting pier at the first main panel point shorewards from the main pier, erect this panel on false-work and then erect both the River and anchor arms as cantilevers—each arm to be carried out at such a rate that they would always balance over the base formed by the main and temporary piers.

In either case, the scheme of cantilever erection was to use a top chord traveller starting at the main pier and working out panel by panel.

The general form of the truss prevented any panel from being self-supporting until a main panel point had been reached, and it was intended to overcome this, by making use of the compression chord of the triangular sub-truss frame as the upper tension member of a cantilever that would hold up the lower half of the second main panel until the long tension diagonal could be connected, this small cantilever being anchored by means of tie-bars to the corresponding point of a completed panel in the anchor arm. Similarly, when the third panel was being erected, temporary tie-bars were to be placed between the chords of the first and second sub-trusses; in the first instance to hold the long compression diagonal during erection, and afterwards to make use of the sub-truss as a cantilever as in the first panel. The principle was to be continued throughout the cantilever arm.

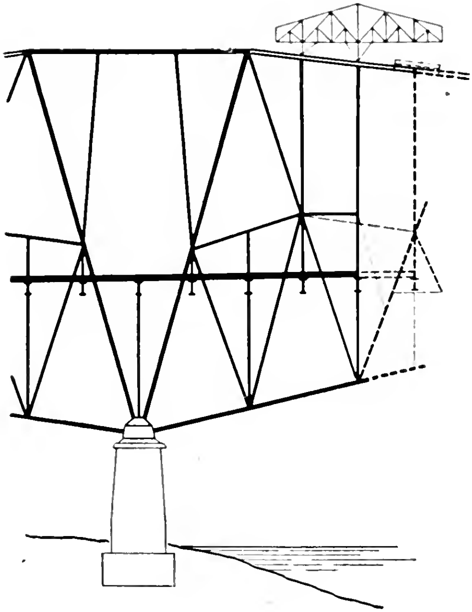


Figure 1

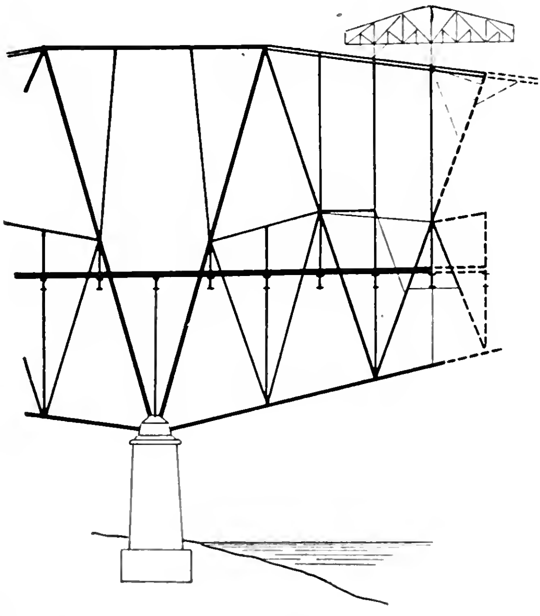


Figure 1a

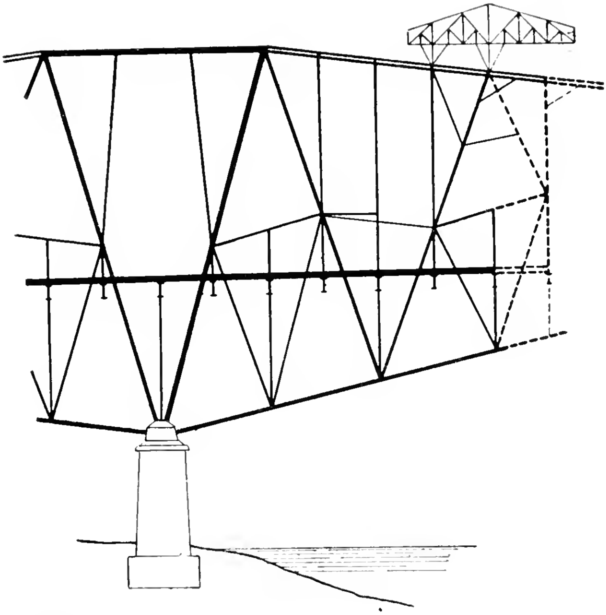


Figure 1b

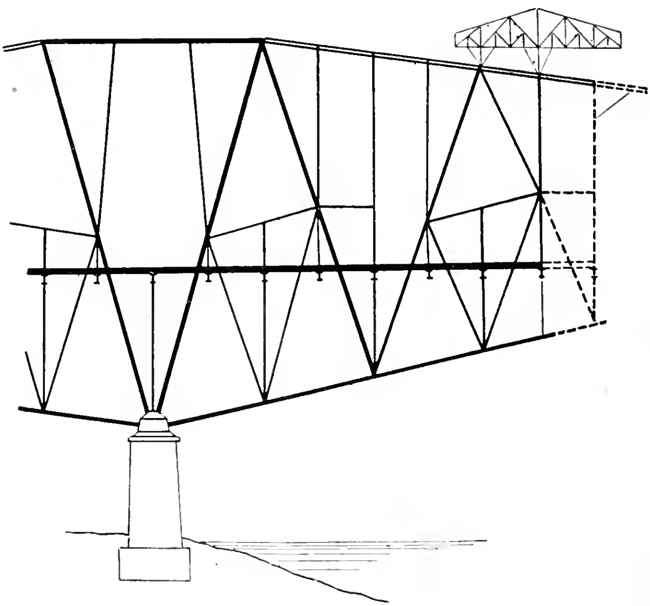


Figure 1c

This method of erection required that the upper chords of the subframes should be designed to carry their compression stresses when the bridge was completed, and should in addition be capable of carrying the heavy tension stresses imposed during erection. Moreover, and herein lay a considerable difficulty, they should be capable of being adjusted with nicety to varying lengths while under heavy stresses.

A consideration of the Official Design in the light of the investigations and the data assembled by the Royal Commission and by the Board of Engineers assured us that Mr. Vautelet had put on paper a bridge in which, if built, every confidence could be placed that it would perform the work for which it was designed. It was, however, manifest that many of the members would be much too large to manufacture with any existing equipment, and that the manufacture must be carried out with a degree of accuracy hitherto unattained to assure that the parts would go together in the structure and perform their intended functions properly. This requirement could, however, be fulfilled as it only needed proper equipment and exceptionally good workmanship, the serious difficulty being the erection or the assembling of the members to construct the Bridge.

When I returned to my old position as Chief Engineer of the Dominion Bridge Company in January 1910, all the essential details of the official design had been worked up, inquiries were being made as to the largest sizes of material obtainable and possible methods of erection were being considered. Mr. Johnson had followed the work of the Board, and when it was decided that the Dominion and Canadian Bridge Companies would tender jointly for the construction of the Bridge, he had given much thought to the problems of its construction. He had early foreseen such great difficulties in manufacture and erection that he considered desirable some radical departures from Mr. Vautelet's design or, indeed, from any design for a long span bridge that had heretofore been illustrated. Mr. Johnson's studies led him to an appreciation of the possibilities of what has since come to be known as the "K" form of bracing, never before used for an important structure and as he and his staff were at the time fully occupied with other important work, he asked me to take up the development of this design and generally supervise the preparation of tenders.

Mr. F. P. Shearwood, M. Can. Soc. C.E., Assistant Chief Engineer of the Dominion Bridge Company, had been collaborating with Mr. Johnson in considering the erection of the Official Design and the possibilities of the alternative "K" design. He had already made a number of preliminary sketches and outline strain sheets, and it was unfortunate that Mr. Shearwood's regular duties prevented him from following up this work. He was, however, available for consultation and was of much assistance throughout the development of the "K" design.

The Designing staff was built up as rapidly as possible, Messrs. Harkness, Wilson and Linton of the Dominion Bridge Company Staff early being transferred to it. As the work developed the staff was added to until when Mr. Porter and Mr. Pratley returned from the Board of Engineers, about May 1910, the staff had outgrown the available accommodation and a new office was built in which the preparation of our designs could be carried out. The Staff consisted of Messrs. G. F. Porter, M. Can. Soc. C.E.; P. L. Pratley, M. Can. Soc. C.E., A. L. Harkness, A.M. Can. Soc. C.E., L. R. Wilson, A. M. Can. Soc. C.E. (now Major); E. C. Kerrigan, A.M., Can. Soc. C.E., H.M. Lamb, A.M. Can. Soc. C.E. (now Professor): A. P. Linton, A.M. Can. Soc. C.E., (now Major): Jas. McNiven, A.M. Can. Soc. C.E. (also overseas) and F. C. MacDonald, with frequent assistance of others from the Drawing Office in the making of tracings and the computation of weights and ordinary stresses.

The work was so unusual, with so little precedent to guide, that it required a large staff of engineers to carry it on.

In preparing competing designs for ordinary bridges there is so much data at hand as to the cost of shop manufacture, erection costs, and the weight of details, that it is generally unnecessary to do more than prepare strain sheets with a few details from which weights can be very closely estimated; but, the magnitude of this bridge required, in addition to calculations for dead and live load stresses, extended calculations of wind, temperature and traction stresses and also of the bending stresses due to the weight of the unsupported length of the members, and to the elastic deformation of the structure or what are usually termed secondary stresses. The make-up of the members, as well as the details for connecting the several members together, also necessarily differed much from previous practice with smaller members.

The above considerations made it impracticable to follow the ordinary course of balancing the merits of trial designs by comparing outline strain sheets, and for every design which was considered worth a real trial it was necessary, after fixing the length of the suspended span and deciding how it might be erected, to work back from the ends of the cantilever arms to the piers, designing and detailing each member with sufficient accuracy to obtain a very close approximation of the final weight of the member in the structure, and particularly to consider how the larger members could be manufactured and erected in the bridge.

Concurrently with the design of the bridge itself it was therefore necessary to keep in view the design and cost of the plant and equipment for manufacturing the work, the transportation of the members to the site and the design and cost of the erection plant necessary for the design in hand. Included in the latter are the special erection travellers, steel erection staging, pontoons and storage yards with heavy cranes, in addition to the ordinary erection equipment required for a large bridge.

Our preparations are evidence that we were most anxious to obtain the contract, but we were equally anxious, if successful in our tender, that the Bridge should have a pleasing appearance, that none of the errors of the Phoenix design should be repeated, that the make-up of the members, the details and the connections should be of the most efficient character and, indeed, that we might be confident the bridge would be in all respects a credit to us and to Canadian bridge building.

A superficial examination of the Official Design had revealed that it was the result of much careful work, and that it had many excellent features. It was therefore felt that before making, or concurrently with the preparation of, our alternative designs, a careful detailed study of Mr. Vautelet's design would be of much assistance in developing our own work. The critical examination confirmed the opinion that in many respects, little or no improvement could be made. The principal features that were adopted, to be incorporated in the alternative designs were:

(a) The general form of the compression members with abutting joints fully spliced before being subjected to stress;

(b) The compression chords without bends in their length but increasing in depth towards the piers to keep a proper ratio of thickness of material as the stresses increased.

The curved bottom chords of the Phoenix design while perhaps tending to economy of material, presented several very objectionable features, the principal being that the horizontal wind forces, and these are very considerable, cause heavy vertical components at the joints, reversing in direction as the chord is in tension or compression. These do not exist with the chords lying in one plane from end to end. It is most difficult if not impracticable to fully or efficiently splice chords deflected at panel points.

(c) All of the main compression members in the Official Design were made up of four webs, shop connected in pairs, so that each truss was virtually two trusses placed close together and connected by tie-plates and lattice bars in the field. This assured a better distribution of stress throughout the members of the truss, permitted the heavy members to be shipped in practical lengths and greatly facilitated the erection.

(d) The construction of the shoe and transfer of the load from the steel work to the masonry. The total vertical load was estimated at 30,500 tons; the transverse wind load at 790 tons, and the longitudinal wind load at 3,730 tons; a horizontal compression load of 15,200 tons. The loads required a bearing area on the masonry of 700 square feet, and there were many difficulties in the way of evenly distributing such large forces over so great an area while at the same time providing for the transverse forces. The pins were so placed that each loaded the area of the bed-plate tributary to it and the design amply provided for all the forces as well as the necessary stability.

(c) The sleeves on the pins to reduce the friction and thus provide for the necessary deflection during erection were also good.

The displacement diagrams were good as well as the secondary stresses resulting from deformation.

The objectionable features from the theoretical view point and from that of construction will be referred to in comparing the Official Design with the present Bridge, where it will be shown that we were able to depart from the make up of some members, and from the details and the connections between web members and chords in a manner much to our advantage in shop and erection, and we think also to the betterment of the final structure.

Plate VI. — Gives a comparison of the larger chord sections of the present Quebec, Forth and other great cantilever bridges. The section of the Quebec chords is much larger than that of any of the other bridges, having 1940 square inches against the next largest of 853 square inches, and the section of the Official Design was still larger, having 2,038 square inches. The Hell Gate Arch at New York, since constructed, has a large section, 1,392 inches, but that is of different type of construction and is not really comparable for our purpose. Similarly, the circular section of the Forth Bridge is not comparable for reasons to be given.

Plate VII. — Shows the cross sections of all the important members in the cantilever arm of the bridge as constructed. The bottom chords between panel points were about 86 feet long, and the heaviest section of bottom chord weighed, with its details, about 380 tons. It would be impracticable to place so large a member with its center of gravity some 45 feet from its connection to the work already built. By splitting it down the longitudinal medial line and splicing it about the middle of its length, each panel of lower chord could be manufactured, shipped and erected in four pieces, none exceeding 93 tons in weight and of which the centre of gravity was only some 25 feet away from the point of its connection or where the erection traveller could stand. The large compression diagonals, while not so great in sectional area, were considerably longer and were really more difficult to handle. The load of these heavy members and the reach necessary to place them, thus became the measure of capacity of the erection traveller.

Plate VIII. — Shows the general character of the erection operations of the Official Design, the temporary members for holding up the permanent members of the bridge already placed but not finally connected, and the members that the traveller will place when in the position shown. Although the amount of permanent bridge to be supported by temporary members would necessarily vary for different positions of the traveller, the principle remains the same throughout the whole erection of the cantilever arm.

Plate IX. — Shows the top chord traveller designed for the above method of erection.

Travellers having a reach of one panel, one and one-half panels, and two panels, were each designed and compared, the one illustrated with a two panel reach being adopted as having the minimum of objectionable features.

Work of this magnitude cannot be handled without risk in spite of every precaution, and much consideration was given to adopting methods and plant that promised a maximum of safety. In a general way the short reach traveller is much lighter and the traveller itself is thus safer to handle, but it requires more temporary material for holding up portions of the structure that cannot be permanently connected up, with the traveller in the position for placing these members. It was therefore considered that while the long traveller was more difficult to handle, its use involved less risk than having so much of the permanent structure hanging on temporary adjustable members as would be required with the short traveller. The long traveller also permitted much more rapid work.

It was intended to carry the traveller upon temporary stringers set by the traveller itself on the top of the vertical posts as these were erected, those over which the traveller had passed being picked up and shifted to the forward position as required.

The traveller alone was estimated to weigh about 1,000,000 pounds, with a moving load on its front wheels of nearly 900,000 pounds, and when lifting its maximum loads the reaction of the front supports would be over 1,500,000 lbs. These heavy loads necessitated heavy stringers to carry them, and very heavy stools to rest on the top of the posts on which the traveller could be blocked when at rest and working. They also required considerable increase in the sections of the vertical posts of the truss and in the sway bracing connecting these posts, the normal functions of these posts in the completed structure being merely to carry the weight of the top chord.

It was difficult to plan how the stringers already passed and left to the rear could be picked up, transferred to the front end of the traveller, and set in the position required to supply a track on which the traveller could proceed. There was a decided risk in passing this very heavy traveller down the sloping chord and stopping it at exactly the right point, as any positive stops used could only be of a temporary nature, it being necessary to remove them to allow the traveller to pass on after performing its work at the point in question. It was difficult to give lateral stability to the stringers carrying the traveller with its great reach and large wind surface.

In addition to the above, the operation of the traveller when erecting the members of the bridge presented many serious problems. It has been stated that for the sake of reducing the weight of the members to pieces that could be handled, every main member of each truss was field connected down a medial plane. The general width of the important compression members was about 10 feet out to out, and the width centre to centre about 5′6″. The heavy members to be erected could only be brought out on the floor of the bridge as far as the forward point of support of the traveller. They must then be picked up by the tackles, eased forward and transversely before being hoisted into their exact position. To do this conveniently the tackles should be hung over the centre line of the piece being hoisted, but, if over the centre line of the outside half, the tackles would be 5′6″ too far out from the inside half. Generally this placing is performed by a guy tackle, but with such heavy weights and short drift it would add appreciably to the stresses, and consequent necessary capacity of the hoisting equipment.

The above difficulties were not due to the form of truss or to the general design of the structure beyond its magnitude, the problems arising from the large dimensions of the members both in length and width and the very heavy weights to be handled at a long distance from the point of support. There were, however, other difficulties due to the form of truss; the principal being the large quantity of temporary adjustable members and their connection to the permanent members, together with the exact adjustment under very heavy stress necessary in order to make connections of main panel points. Some of these members would have a stress of 3,300,000 lbs., a heavy force under which to make delicate adjustments.

Other difficulties presented themselves in carrying out the erection step by step and, while none of them were insurmountable, the sum was so great that it seemed worth making every effort to avoid them. I think it was the knowledge that these problems would arise long before the erection was worked out in detail that led Mr. Johnson to the "K" form of bracing.

The Official Design I. was estimated to weigh 72,700 tons, of which 52,000 tons was nickel steel. In very long span bridges a limiting length exists where the structure can only carry its own weight, and is unable to carry any superimposed load. The length naturally depends on the relation existing between the strength of the material under stress and its weight. For ordinary carbon steel construction and the loading specified, the length of the Quebec Bridge span begins to approach this limit. It was therefore of great importance that there should be no unnecessary dead-weight, particularly in the suspended span and at the ends of the cantilevers, where every pound of load, either dead or live, requires several pounds of material in the cantilevers to carry its effect to the supporting piers.

Nickel steel, which was known to be about 40% stronger in tension and was expected to be 40% stronger in compression, although not proved by experiment at that time, was hence a necessity in portions of the structure. It cost, however, at the time the estimates were being made, nearly two and one-half times as much as carbon steel, and it was apparent that if carbon steel were used where it would not affect the sections of the members in the long channel span,—namely: in those members supported directly by the piers and throughout the anchor arm,—while it would considerably increase the total weight of the structure, much economy in cost would result. It was also apparent that there was much unnecessary weight in the long vertical redundant members inserted only to support the weight of the top chord and the erection traveller, but which could not carry any live load nor serve any other useful purpose in the completed bridge. There was promise of economy and improvement in appearance in reducing the depth at the ends of the cantilevers of both the river and anchor arms, and in reducing the lengths of the anchor arms.

Plate X.—Taken from Mr. Modjeski's paper on long span bridges, shows a graphic comparison of the weights of the Quebec and Forth Bridges. While this does not enter into the present discussion it shows the very great difference in weight per foot of the centre portions of the bridge, and the portions near the main supporting piers and serves to illustrate the importance of reducing the weight towards the centre of the channel span.

Tenders were advertised on the 17th June 1910, to be submitted on the 1st of September, this date later being extended to the 1st of October 1910. The time allowed would have been altogether too short to prepare alternative designs, but the Board had issued a preliminary specification at the end of 1909 and the designs and information of the Board were available to contractors who expected to tender, so that in reality there was about eight months in which to prepare tenders. As already stated, the Board had evolved satisfactory forms of compression members and in many instances details that could be used and the Official Design supplied data from which close approximations of weight could be made for alternative designs—indeed the work of the Board was so thorough that practically all existing knowledge on the subject was reviewed and put into workable form.

While Mr. Vautelet and I did not agree on the design of the Bridge, I wish to acknowledge the large contribution made by him to the knowledge of the subject and his readiness to explain his designs and the results of his studies. A history of the undertaking would be incomplete if Mr. Vautelet's part therein were not included.

We are also assisted by the return of the members of our staff, Messrs. Porter and Pratley. Mr. Porter had been in charge of the design of the details for the Board of Engineers and, while the make-up of the members and the details finally used differed considerably from those of the official designs, the familiarity with these large and unusual details acquired by Mr. Porter during his service with the Board, no doubt enabled him to lay out this work much more rapidly than if it had been new to him. Mr. Pratley too, had much experience in the intricate calculations of the secondary stresses for the Board's designs, and was thus enabled to encompass a very large amount of work in calculating our designs, and because of the experience of these two men with the Board of Engineers our work was very much lightened.

The specifications issued, fixed the distance centre to centre of piers, 1758 feet; a clear head room of 150 feet for a distance of at least 600 feet in the centre of the span; the elevation of grade; the maximum height of steel work over the pier, 290 feet; the cross section of the roadway and the width, centre to centre of trusses, 88 feet; the loading and the unit stresses.

The requirements for material, testing and workmanship called for the very highest standard.

It was required that tenders on Contractors' designs be accompanied by complete stress sheets, deformation diagrams, details and make-up of all members, splices, lacing, and all information necessary to judge the adequacy and agreement with the specification of the proposed design.

The loading and unit stresses were briefly as follows:—

LIVE LOADS.

Trusses: Railway Load —on one or two tracks, 5000 lbs. per lin. ft. with 2-Cooper's E-50 Engines placed to give maximum stress.

Highway Load — 920 lbs. per lin. ft. of each roadway for trusses.

Floorbeams & Stringers and Members receiving Maximum Stress for a length of moving load covering 2 panels or less.	100 lbs. per sq. ft. or 4,600 per lin. ft. of bridge. 2-53 Ton cars each 60 ft. long and 12 ft. wide on each track. A concentrated load of 24,000 lbs. on 2 axles 10 feet apart.

DEAD LOAD.

Railway: Floor 670 lbs. per lin. ft. of each track.

Highway: Floor above stringers 2300 lbs. per lin. ft. each roadway.
Snow load of 1500 lbs. per lin. ft. of bridge.

WIND LOAD. 30 lbs. per sq. ft. on surface of two trusses and train 14 feet high.

TRACTION. 750 lbs. per lin. ft. of one track.

UNIT STRESSES (CARBON STEEL)

	Live Load	Dead Load & Snow	All Coexisting stresses except secondary strains	All Coexisting stresses including secondary strains
	lbs. per sq. in.	lbs. per sq. in.	lbs. per sq. in.	lbs. per sq. in.
Tension Main Trusses........	10,000	20,000	20,000	22,000
Suspenders and Members liable tc sudden loading............	7,000	14,000	14,000	15,400
Railway Stringers...........	8,000	16,000	16,000	17,600
Floor beams & Highway stringers...........	9,000	18,000	18,000	19,800
Compression Members in Main Trusses..	$10{,}000-40\frac{l}{r}$	$20{,}000-80\frac{l}{r}$	$20{,}000-80\frac{l}{r}$	$22{,}000-88\frac{l}{r}$

NICKEL STEEL increase Units given for Carbon Steel as follows:

Tension.................................. 40%
Compression & Pins........................ 25%

Plate XI.—Shows in outline all the designs submitted with the tenders. Official No. 1 was the only design exhibited at the time tenders were called, but it was later supplemented by Official No. V and tenders were asked on various modifications of these two designs.

The study of the Official Design and its erection had shown that in these great bridges the dominating factor of the design must be the practicability and safety of its erection, and this was emphasized in the development of our alternative designs. Economy of material is also an important factor because, in addition to its first cost, the effect of unnecessary weight may be cumulative, considerably increasing the sections of the heaviest members which are the measure of the shop equipment and of the erection appliances.

It was known that long panels with diagonals approximating an inclination of 45 degrees to the horizontal would give the most economical form of Web System, but a diagonal inclined at 45 degrees would require that the deep panels near the pier should be about 200 feet long. It was seen at once that these long panels could not be constructed by the erection methods we had decided to adopt, and that some compromise was necessary either by shortening the panels or adopting a system of sub-trussing.

A consideration of the Forth Bridge construction will perhaps assist in the further discussion of the other designs.

A length of the Forth Bridge equal to the projected Bridge at Quebec weighed about 35,000 tons—all carbon steel. The Official Design was estimated to weigh 72,700 tons, 52,000 tons of which was nickel steel. The Forth Bridge had successfully performed its duties for some 28 years and we were naturally not unmindful of this precedent—indeed, serious consideration was given to its design, and to the possibilities of adopting all or some of the special features giving it its remarkable economy in weight.

Referring to Drawings Nos. 2 and 7 it will be seen that the Forth Bridge differs in several particulars from the design of any of the other great bridges, and from any of the designs prepared for the Quebec Bridge.

The compression members are all circular in section and of large diameter. The circular section gives the largest radius of gyration for the metal employed, but this in itself was not an important factor as nearly every member in the bridge required sufficient area to give an effective radius with any of the ordinary forms of cross section. It results, however, in economy in the details because latticebars, tie-plates and other material which does not carry direct stress but is merely introduced to stiffen the rectangular member may be almost entirely omitted.

The trusses of all the bridges illustrated, including those of the designs submitted with tenders, lie in vertical parallel planes, while the trusses of the Forth Bridge are battered from a width of 120 feet at the shoes to 33 feet at the top.

There is important economy in this construction. The floor beams may be made only long enough to accommodate the traffic; the weight of the top lateral bracing and sway bracing is very much reduced and the section of the compression chords is also reduced. The total stress in the first panel of the bottom chord of the Quebec Bridge is about 15,000 tons, requiring a section of 1628 square inches; of this stress 4,000 tons is due to wind, requiring a section of 450 sq. inches, and had it been practicable to increase the width at the shoes to 120 feet, as in the Forth Bridge, the wind stress would have been reduced by about one-quarter and the section of the chord by 112 sq. inches.

The drawings do not show it, but if a horizontal section were taken at the floor level it would be seen that this section of all other bridges is rectangular, while in the Forth Bridge, the cantilevers taper towards each other, until at their ends they are only 32 feet wide. By making the suspended span only wide enough to accommodate the traffic and for its own lateral stability considered as a simple span, much economy is realized due to the shortening of the lateral bracing and floor beams. As already pointed out, any saving in the weight of this portion of the bridge is multiplied several times in the weight of the cantilevers.

The diagonals have an effective inclination throughout the trusses.

Members of such large dimensions as those used at the Forth could not be fabricated in shops at a distance, transported to the site and erected, and in the case of the Forth Bridge the manufacturing plant was actually built at the site of the Bridge.

Moreover, many of the details for connecting the circular compression web members were of necessity so complicated that it would have been next to impossible to manufacture them, except by the method of laying out each piece as the work was built up. The shapes and plates entering into the large members of the Forth Bridge were therefore marked off, bent, fitted and drilled or punched at the site, and the work was erected plate by plate and section by section—the whole system of laying off, fitting up, and riveting, being closely analagous to steel shipbuilding.

This method of erection not calling for the handling of heavy pieces permitted each large member to be projected out plate by plate, carrying itself as a cantilever, until it reached a point of intersection with some other member, where they would give mutual support, or where in the case of a chord, a tie or strut could support the member from the intersection of the diagonals.

To carry out the work in the case of the Forth Bridge a large force of men was required, the number being at times over four thousand. The work proceeded continuously throughout the year for over four years. There were plenty of men trained in steel shipbuilding on the Clyde, and climatic conditions permitted the work to be carried on at all seasons of the year.

A sufficient number of skilled hand mechanics was not available in Canada, it is doubtful if they could have been assembled even for continuous work, and when it is realized that erection work at Quebec could only be carried on for some six months in the year, it will be seen that it was quite impracticable to adopt this type of construction.

We therefore decided early that the only practical method of erecting the bridge in a reasonable time would be so to design it that the field force would be a minimum. This necessitated manufacturing the members into as large pieces as could be transported and erected, and in order that these pieces might be manufactured economically it was necessary to install special plant capable of handling unusually large members, and requiring special machinery and mechanical appliances.

The use of this machinery also reduced the manufacturing force to the smallest possible limit. Our shop forces at no time exceeded five hundred men, and our field force working at the same time on the cantilever arms on both sides of the River, and on the erection of the suspended span at Sillery Cove, did not exceed, exclusive of field painters, 500 men, or about 1,000 all told for shop and erection. The average field force when only one cantilever was being erected, or before the erection of the suspended span, was very much less.

Although it was thus found impracticable to adopt circular compression sections or the field construction of the Forth Bridge, consideration was given to adapting the method of construction we proposed to battered trusses and to vertical trusses tapering in plan.

It will be realized from a consideration of the erection traveller and erection methods discussed, that members of the dimensions actually handled, or of any practicable size, could not be erected in an inclined plane without a very large amount of temporary supporting material, and that the field connections of inclined members would be most difficult.

With regard to the plan of vertical trussses with cantilever arms tapering in a horizontal plane, this also had ultimately to be abandoned on account of the difficulties of erection. A top chord traveller would have been impossible, due to the constant change of gauge of the tracks on which it would run and a through traveller could not be made wide enough for stability. Indeed, the only way of erecting either plan, seemed to be by means of an outside traveller supported on temporary cantilever beams such as was used on the Phoenix Bridge. A few preliminary sketches and estimates quickly demonstrated that the cost of erection and temporary material would far out-weigh the saving in weight to be gained by the narrower centre span. Even had this not been the case, the bent detail for connecting the members where the planes of the tapered trusses intersected, offered many difficulties, and it is doubtful if it could have been made with sufficient accuracy to insure the calculated distribution of stress throughout the intersecting members.

In a cantilever bridge of large dimensions the suspended span is the first element to be considered, as it may be designed and estimated without regard to the rest of the structure except as to its width, while the weight of the suspended span and the method of its erection must be considered in designing the trusses of the cantilever and anchor arms.

There was sufficient data on long simple spans to permit an economical arrangement of panels, outline of trusses and form of bracing to be readily chosen, and little trial designing was necessary here. It was desirable, however, to determine the method of placing the suspended span in position before fixing upon its length, its depth at the ends of the trusses or the outline of the trusses. If it were determined to erect it on falsework and float it into position, the depth at the ends would be kept low enough to give only sufficient head room for efficient portal bracing, so that the form of the chords might approximate to a parabola which was known by experience to give much the most economical outline for very long simple spans.

If it were to be erected by cantilevering out from each side and joining in the centre, the end depth would of necessity be increased to something approximating the depth at the centre, otherwise it would be difficult to provide material for the heavy moment caused by the weight of the cantilevered portion, together with the weight of the erection travellers and material at the ends of the half span cantilevers, when the final connection was being made.

Even with the greatest practicable depth at the ends of the trusses, the cantilevering method of erecting the centre span called for a very considerable increase of material in the cantilevers and the span to provide for the erection stresses, and this in itself seemed a sufficient reason for early determining to float the centre span into position, thus permitting the most economical outline to be adopted.

There were, however, other considerations in addition to that of economy leading to this conclusion. When a suspended span is cantilevered out from each side, in order that the connections at the centre which convert it to an ordinary span may be made, the two halves must meet exactly in the centre and the upper and lower chords must be in alignment both vertically and horizontally. The length of the steel work varies with the temperature and from change of load, and the horizontal alignment may also vary from changes in temperature and wind loads. Adjustments must therefore be provided where the span connects to the cantilever in order that these conditions may be met. Satisfactory appliances for these adjustments have been devised, and successfully used on all the great cantilever bridges heretofore built, the centre spans of these having been erected by cantilevering, but in some instances the final connection at the centre of the span was only made after great trouble and with some risk. The element of danger has lain in the difficulty of dealing sufficiently rapidly with the existing heavy forces, when making the delicate adjustments necessary to follow the changes in length and alignment that take place through the variations of temperature. Furthermore, when the centre connection is made, if the chord connections to the cantilever arm are not immediately released a change of temperature may cause the centre span to so connect up the cantilevers as to complete an arch from pier to pier, or at the other extreme to develop an undue amount of tension in the top chords. Tentative designs for the adjustment mechanism required for the very heavy stresses found here, showed that it would be cumbersome and costly.

Having decided to float the centre span into position, its length became a question of balancing the increased difficulty of floating and placing a very long span against the saving in weight that might be effected by approaching the theoretical length for economy.

Theoretically, the least weight of material in the finished structure would result from making the suspended span about 1100 feet long, but this is a quite impracticable length for the span itself, and 668 feet was the longest simple span hitherto built. The Official Design divided the total span into three equal portions, making the cantilever arms and the suspended span each 586 feet. This arrangement was adopted as it fairly balanced the above considerations, and it facilitated the use of the official data in the new designs.

The types of trusses from which to choose may be enumerated as follows:

(1) The Official Design, Figures 1 and 2, Plate XI a Warren truss with each main panel divided in two and floor stringers of moderate length, forcing an uneconomical inclination to the diagonals.

(2) "M" Design, Figure 5; a Warren truss with each main panel divided in four, permitting the most economical inclination of the web members to be chosen while retaining short stringer lengths.

(3) A Warren truss having the diagonals at a favorable inclination without redundant members to support the top chord, each long panel being sub-divided in the middle to give a stringer length about twice that adopted for any of the other systems. See Figure 6.

(4) Double intersection trusses with and without sub-divisions.

(5) The "K" form of bracing.

The Phoenix Bridge with all diagonals in tension and sub-divided to give two stringer panels in each main panel, gained a favorable inclination of the diagonals and an economical form through the curvature of the bottom chord, but this form of truss was naturally not considered.

It was at once apparent that type No. 3 would give the least weight in the finished structure and have a pleasing appearance, but unless it were fabricated and erected in the same manner as the Forth Bridge, there seemed so many obstacles in the way of erecting it with safety and at reasonable expense, that merely outline sketches were prepared and serious consideration was not given to this type.

Some preliminary designs were made for double intersection trusses, but although the stresses and sections of the web members are only half those of the single intersection and the web members thus lighter and easier to handle, there seemed no way of getting favorable inclination of the web members without encountering similar erection difficulties to those met with in the single intersection design. A tentative design had been made by the Board on this system, but Mr. Vautelet had not found advantages in it to compensate for the stresses being statically indeterminate, and he was strongly opposed to its use.

Mr. Johnson and I early pinned our faith to the "K" form of bracing, but some of our associates wished to compare the merits of a bridge with a more conventional and more economical arrangement of bracing by preparing a complete design. Mr. Emil Larsson, Assistant Chief Engineer of the American Bridge Company, was asked to make this design, which is illustrated on Plate XI as "M-N".

Anticipating the more detailed discussion of the "M" design which will follow, it was at once realized that while it would make a very economical arrangement of the members actually carrying stresses there would be objections to its employment, in that the sub-truss system would probably introduce heavy secondary stresses from local panel loads; the top chord must be supported from the panel points as in the Official Design by long vertical members not useful for carrying live loads. During erection, very heavy temporary members would be necessary to hold up the inclined struts and the bottom chords until the main panel points were reached.

Plate XII. — Is a general elevation of the "K" truss design submitted with the tenders.

The "K" form of bracing is quite different from the trusses of any of the other great cantilever bridges or from any form of trussing that had been previously illustrated in British or American practice.

It may be likened to a double intersection Warren truss with every half panel reversed, and a vertical member inserted, thus retaining the advantages of the double intersection truss in halving the shears between two members in each panel. It is statically determinate as to stresses, the shear being positively divided at each panel point, whereas in the double intersection truss, however great the care in calculating the stresses, one system or the other may accumulate more than its share of stress due to errors in manufacture and erection. The top chord length of the double intersection truss is halved without redundant members. It will be demonstrated later that no temporary work is required in erection and that it is much the safest and easiest form of trussing to erect.

Most of the difficulties anticipated in manufacturing and erecting the Official Design had their principal source in the steep and uneconomical inclination of the web members, the large section of these members, and the necessity of either having the erection traveller extend out over two panels or having a very large quantity of temporary holding-up material, expensive to supply and very difficult to place and adjust with the required nicety. With the "K" form of truss the main panel could be halved for the same angle of web bracing; or, conversely, for the full panel the angle of the bracing would be nearly twice as favorable. The early sketches were for the full panel of 84 feet, and for several lengths down to panels of 65 feet long, all to be erected by a top chord traveller.

The "K" truss design lent itself to the use of a top chord traveller, the normal stresses from live and dead load in the vertical posts requiring sufficient section to carry the erection stresses imposed by the traveller, and the cost of the erection equipment for this arrangemeut would have been comparatively small. The long panels, however, required such heavy floor beams that there were difficulties in the way of manufacturing, transporting and erecting them. The stringers were also heavy, and the general arrangement did not realize the economy expected. Moreover, the risk of using a top chord traveller with two panels reach was not in any way abated.

Deformation diagrams were made for the sub-panel now used. It was found that this form of sub-trussing did not introduce objectionable secondary stresses, and did not interfere with fully splicing the joints of the compression members as the erection proceeded. Altogether this arrangement had everything to recommend it, and it was adopted.

The trial of the sub-panel was initiated in the endeavour to use a through traveller which would avoid the risks attendant on the use of the top chord traveller. The through traveller is necessarily much heavier than the top traveller, and one to reach 84 feet or even 65 feet, was found quite impracticable. There was no doubt, however, that a traveller could easily be designed of moderate weight that would place members of the size to be handled at a distance of about 42 feet from its forward point of support. The early designs for this traveller were made with four swinging booms each capable of lifting the heaviest member to be handled and swung by guy-tackles from a horizontal frame-work attached to the top of the traveller. As the erection was worked out in detail, it was found that the use of the booms would not be entirely satisfactory, and we were led to adopt the travelling cranes, notwithstanding that this arrangement added much to the cost of the equipment.

Touching on the outline of the whole structure,—the length and outline of the centre span had early been determined. The maximum height of the cantilever arms over the main piers was fixed by the specification. A lower height was undesirable, because even if it resulted in economy of total material, it would require heavier chord members near the piers and these large members, as already pointed out, fixed the size and capacity of the erection traveller and lifting devices as well as the cranes and many shop tools.

Economy demanded that the end height of the cantilevers should be as low as practicable provided the end post from which the centre span is suspended should have an economical inclination approaching 45 degrees. The outline of the bottom chord of the bridge was also fixed by the specified height of the central portion to give clearance for navigation, and the requirement of straight chords between this central portion and the main shoes.

For the sake of appearance, repetition of shop work and other reasons, it was desirable that the anchor and cantilever arms should have the same panel lengths and form of trussing, but it was found economical to shorten the anchor arms by one full panel. There would probably have been economy of material in a still shorter anchor arm, but it would have increased the already heavy load on the main piers, 502 feet was the shortest anchor arm of the Official Designs, and a shorter design was not tried.

Plates XIII and XIV. — Show the general arrangement of the traveller used for erecting the Quebec Bridge. It was equipped with two travelling cranes, each having two trolleys of 60 tons capacity, that could be run out far enough to lift the outside members of the trusses, and auxiliary hoists at the ends of each crane of 7 tons capacity. The four derrick booms each had a capacity of 15 tons. The total weight of the traveller and rigging was about 940 tons, and with both cranes run to the rear position for moving, the load was almost equally distributed on the front and rear trucks. When lifting the maximum loads the reaction at the front points of support was about 1300 tons.

The top of the steel work of the traveller was about 210 feet above the rail on which it ran. The crane runways were about 140 feet and permitted a crane to work 49 feet in advance of the point of support.

The above short description of the traveller is given here to convey an idea of its size, and the importance of this portion of the field equipment. It will be fully described in the forthcoming paper.

After planning our erection for the "K" truss we found that this form of traveller could be advantageously applied to Official Design V and we estimated on using it for erecting that design.

Figure 2. — Shows the method proposed for using this traveller in erecting Official Design V. It will be seen that as the traveller is moved out from floor beam to floor beam, it is necessary to support it, and a portion of the bridge, by means of temporary ties until it has reached a position where it stands on the fourth floor beam away from the shoe where the unit of the main truss system is completed, and the same condition obtains for the succeeding four floor beams. It will also be noted that until its rear portion has moved past the top of the inclined strut, the permanent sway bracing cannot be placed between the compression struts, and a large portion of the bridge is without transverse bracing above the floor system.

To understand this we shall have to refer to the system of wind bracing adopted. It has been usual to place a system of lateral bracing in the planes of both the top and bottom chords to provide for wind loads and vibrations, and to keep the compression members in line. The stresses, or part of them, that go through the lateral bracing in the plane of the top chord are then considered as being carried to the piers by a system of sway bracing between the main posts.

This arrangement results in some ambiguity of stress, and the Board properly decided that the more correct and economical method would be by means of sway bracing between the compression members, to transfer all horizontal loads to the bottom chord, and thus carry all the horizontal loads through the lateral bracing of the bottom chord to the piers where they must eventually be resisted.

This system of top lateral and sway bracing, excellent in itself, was not well adapted to a through traveller as in several positions of the traveller the sway bracing would have to be omitted from between the inclined struts until the traveller had moved out clear of them and there would thus be two panels of truss without permanent lateral bracing.

Figures 3 and 4. — Show the traveller when erecting the "K" truss. It will be noted that in all positions of the traveller, each floor beam when placed is carried by the permanent construction of the bridge and no temporary work is required except for holding up the bottom chord, and that, only because it is convenient to place the lower members first. If the diagonal members were placed first they would interfore with the tackle necessary for placing the lower chord.

Another advantage in the erection of the "K" truss is that the lateral and sway bracing can be placed between the vertical posts as soon as the traveller has moved past one of these posts and, moreover, can be erected by the traveller itself, so that the structure as it is erected is braced and securely held against all lateral forces by the permanent construction.

Figure 5. — Is a deformation diagram for both anchor and cantilever arms showing the framed form of the trusses. It is shown here to illustrate the absence of sharp bends and the generally uniform deformation resulting from the "K" form of bracing, an advantage that had not altogether been anticipated when the designs were started. It was early demonstrated, however, as it was necessary to make deformation diagrams for each stage of the erection, to determine whether there would be bends or undue distortions during erection. As a further illustration of the uniform deformation of the "K" truss.—

Figures 6, 7 and 8 — Show the anchor arms of the Phoenix truss, the Official Design and the "K" truss. The distortions of the Phœnix truss are very marked, particularly those of the bottom chord. The lower chord of the Official Design has a uniform deflection, the long diagonal members are kept straight through the adjustment of the upper chord of the sub-truss before referred to, but there are marked kinks in the upper chord where the long vertical redundant members push it out of line.

Figure 9. — Shows a comparison of the bottom chord deflections under dead plus live load for the three designs.

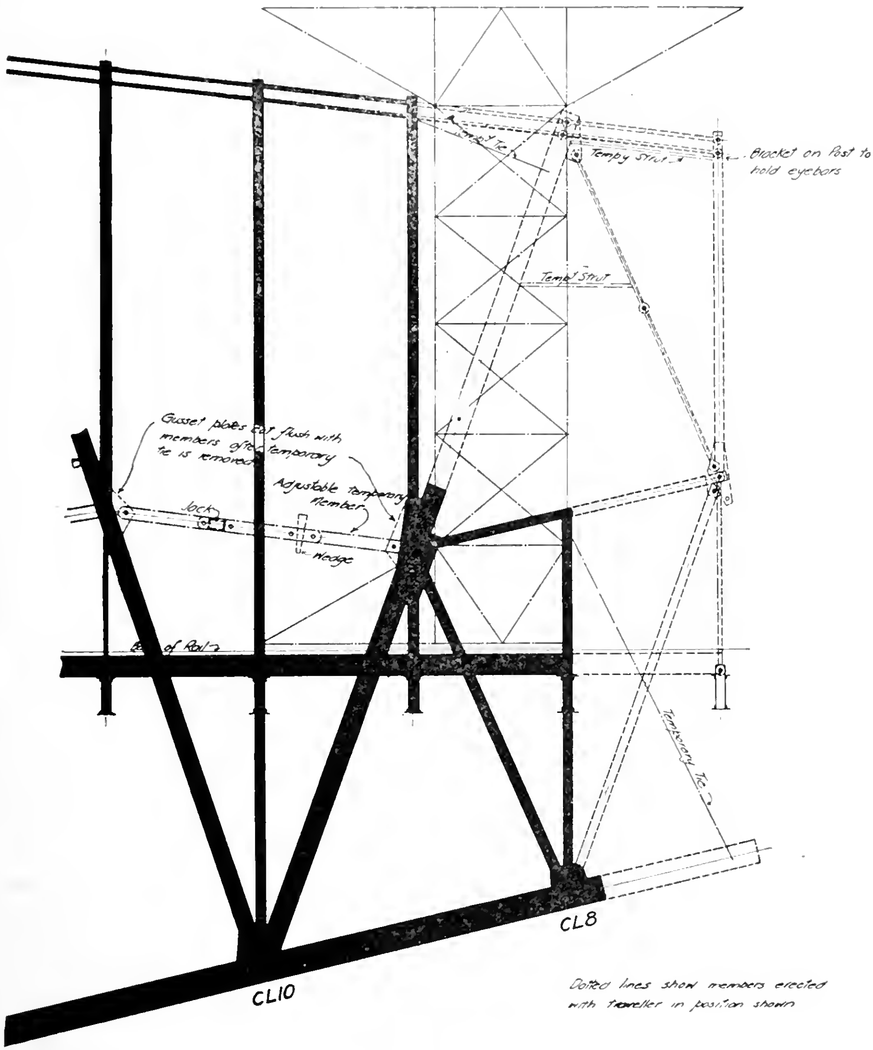

Tempy Tie
Tempy Strut
Bracket on Post to hold eyebars
Temp'y Strut
Gusset plates cut flush with members after temporary tie is removed
Adjustable temporary member
Jack
Wedge
Base of Rail
Temporary Tie
CL8
CL10
Dotted lines show members erected with traveller in position shown

Figure 2

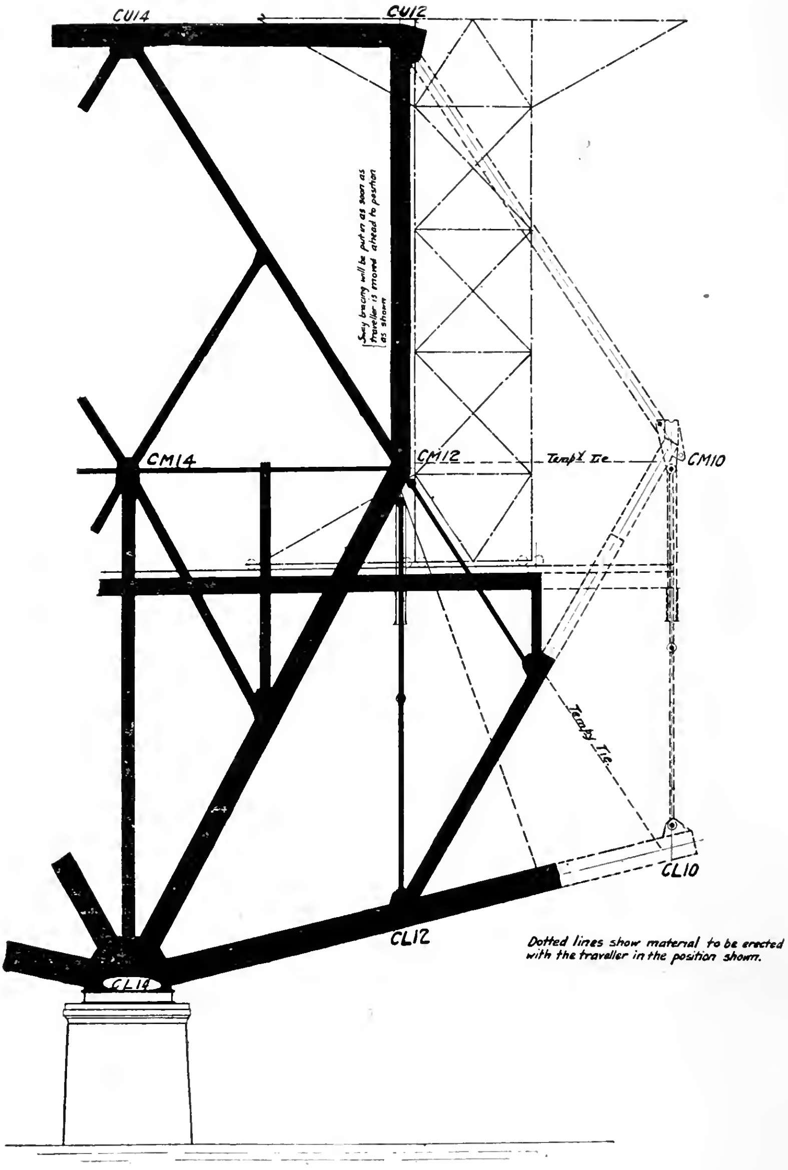

CU14
CU12
Sway bracing will be put in as soon as traveller is moved ahead to position as shown
CM14
CM12
Temp'y Tie
CM10
Temp'y Tie
CL10
CL12
CL14
Dotted lines show material to be erected with the traveller in the position shown.

Figure 3

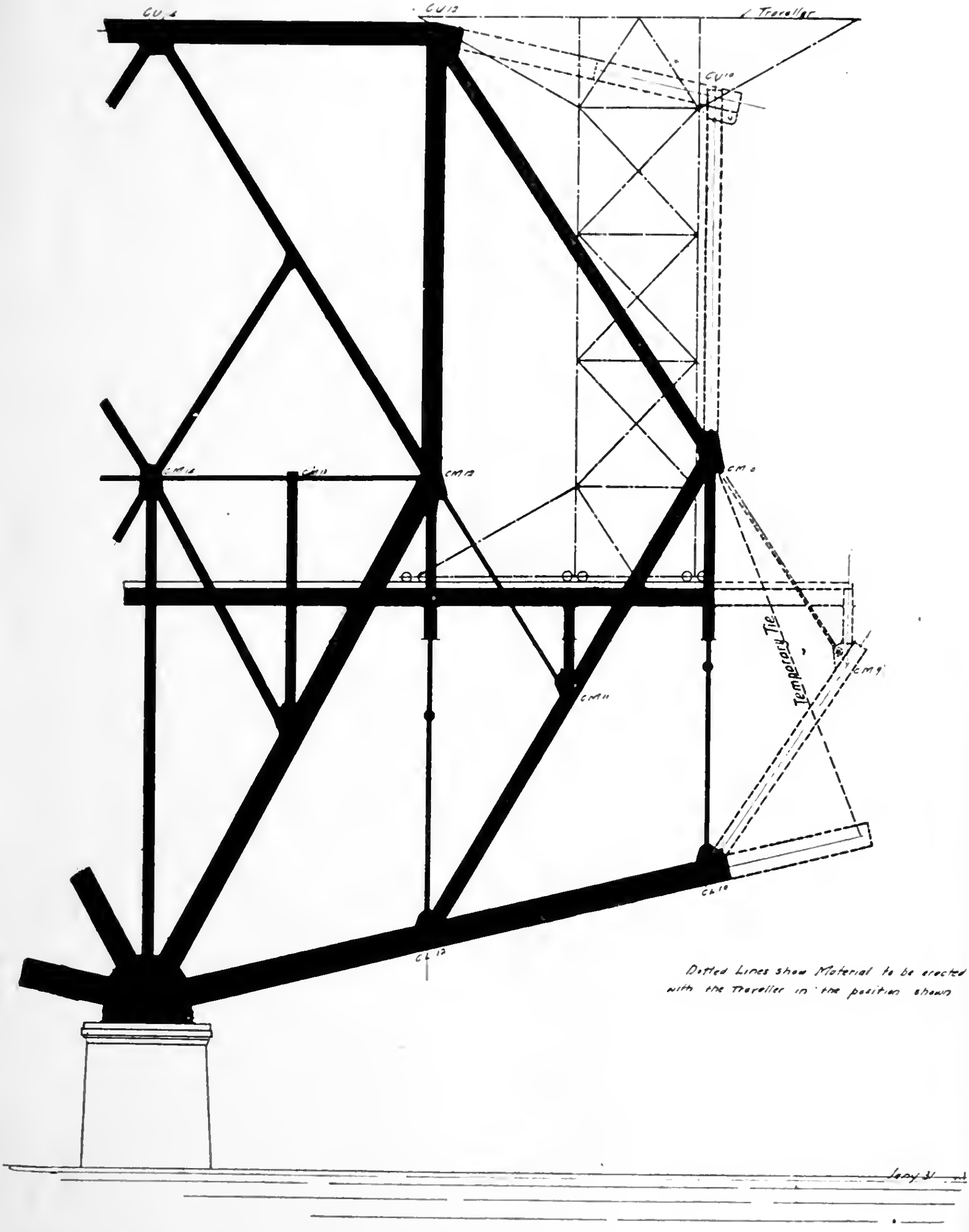

Temporary Tie
Traveller
Dotted Lines show Material to be erected with the Traveller in the position shown

Figure 4

DISPLACEMENT OF PANEL POINTS
Co-ordinates given in inches

		ANCHOR ARM																						
		TOP CHORD								MIDDLE POINTS								BOTTOM CHORD							
		14	12	10	8	6	4	2	0	14	12	10	8	6	4	2	0	14	12	10	8	6	4	2	0
D1	HORIZONTAL	-.256	-.220	-.185	-.155	-.126	-.105	-.089	-.081	-.119	-.115	-.128	-.120	.112	.105	-.090	—	0	+.022	.045	.165	-.080	.089	.093	-.073
	VERTICAL	+.074	-.049	-.098	-.133	-.145	-.134	-.125	.070	+.013	-.053	-.103	.137	-.151	-.146	-.175	—		+.039	-.096	-.130	-.143	.38	-.14	-.073
D2	HORIZONTAL	-.204	-.175	-.146	-.118	-.099	-.084	.069	.062	-.108	-.093	-.097	-.093	-.086	.081	-.075			+.016	-.034	+.049	+.062	+.069	-.073	
	VERTICAL	+.017	-.043	-.077	-.104	-.116	-.110	.099	.055	+.038	-.043	-.081	-.110	.114	-.116	-.099			-.037	.075	.30	-.112	.128	-.091	
D3	HORIZONTAL	-.460	.395	-.331	-.273	-.225	-.189	-.158	-.143	-.227	-.208	-.225	-.212	-.198	-.186	-.171			+.038	-.074	-.114	-.142	.158	-.166	
	VERTICAL	+.041	-.099	-.175	.237	-.261	-.249	-.220	-.125	+.051	-.096	-.184	-.247	.269	-.262	-.274			-.086	.171	-.230	.255	-.246	-.235	
D4	HORIZONTAL	+.196	+.167	+.140	+.114	+.095	+.081	+.066	+.058	+.106	+.087	+.090	+.084	+.055	+.077	+.072			+.015	+.032	+.047	+.058	+.066	+.037	
	VERTICAL	-.016	+.038	+.074	+.100	+.111	+.106	+.295	+.258	-.038	+.041	+.077	+.105	+.113	+.111	+.095			+.035	+.072	+.098	+.177	+.128	+.087	
D5	HORIZONTAL	-.264	-.228	-.191	-.159	-.130	-.108	-.092	-.084	.121	-.119	-.132	-.123	-.116	-.109	-.099			-.023	-.047	-.067	-.087	-.092	-.096	
	VERTICAL	+.025	-.051	-.101	-.137	-.150	.143	-.129	-.072	+.013	-.055	.107	.142	-.156	-.131	-.129			-.051	-.099	-.132	-.148	-.142	-.18	

		CANTILEVER ARM																									
		TOP CHORD									MIDDLE POINTS									BOTTOM CHORD								
		16	14	12	10	8	6	4	2	0	16	14	12	10	8	6	4	2	0	16	14	12	10	8	6	4	2	0
D1	HORIZONTAL	-.256	-.256	-.254	-.251	.246	-.243	.235	.229	-.215	-.119	-.128	-.143	.148	.154	-.160	-.164	-.167	—		-.07	-.031	-.043	-.062	-.077	-.093	1.36	
	VERTICAL	-.024	-.105	+.217	-.334	+.462	+.580	-.688	+.778	+.915	-.013	+.097	+.210	+.337	+.464	+.573	-.685	+.777			+.16	+.277	[illegible]	+.430	+.588	+.694	+.781	
D2	HORIZONTAL	-.204	.208	.210	-.213	.215	-.215	-.212	-.210	-.195	.09	-.124	.15	-.120	-.124	.130	-.135	.140			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-.357	-.071	
	VERTICAL	+.036	+.070	+.143	[illegible]	-.331	+.458	-.531	-.639	+.805	[illegible]	-.067	[illegible]	-.225	-.327	+.437	-.515	[illegible]			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	+.508	[illegible]	
D3	HORIZONTAL	-.460	-.464	.464	.464	.46	-.455	-.447	-.438	.410	.228	.288	-.256	[illegible]	[illegible]	-.290	-.294	-.307			-.035	[illegible]	[illegible]	[illegible]	.23	-.157	[illegible]	
	VERTICAL	[illegible]	+.175	+.363	+.564	[illegible]	[illegible]	[illegible]	+1.417	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-.781	[illegible]	+1.222	[illegible]			+.32	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
D4	HORIZONTAL	[illegible]	+.199	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]			[illegible]	[illegible]	[illegible]	-.039	[illegible]	[illegible]	[illegible]	
	VERTICAL	[illegible]	-.267	[illegible]	[illegible]	-.319	-.471	-.521	[illegible]	[illegible]	[illegible]	.264	.34	[illegible]	-.314	[illegible]	-.516	[illegible]			.069	[illegible]	[illegible]	-.324	.425	-.575	[illegible]	
D5	HORIZONTAL	-.264	-.265	.264	[illegible]	-.256	-.253	-.244	-.238	[illegible]	.23	.69	.48	-.155	.159	[illegible]	[illegible]	-.172			[illegible]	-.03	[illegible]	[illegible]	-.079	-.095	[illegible]	
	VERTICAL	[illegible]	[illegible]	+.273	+.344	[illegible]	+.597	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	+.337	[illegible]	+.589	[illegible]	[illegible]			[illegible]	[illegible]	[illegible]	+.483	[illegible]	[illegible]	[illegible]	

D1 = Displacement from Geometric with bridge fully erected and without Live Load see S211 and S311.

D2 = Displacement due to load of 2,564,600 lbs at CU0 obtained as follows

Finished weight of Suspended Span =	3,108,000	see S102
plus Material not erected at end of Cantilever Arm =	168,600	
	3,276,600	
minus Weight of Lifting Apparatus (including Mooring Truss) =	712,000	
	2,564,600	

D3 = D1 + D2 = Displacement from Geometric with Cantilever Arm completed and Lifting Apparatus in place.

D4 = Displacement due to Lifting Weight of Suspended Span = 2,460,000 lbs

D5 = D3 - D4 = Displacement from Geometric when lifting Suspended Span.

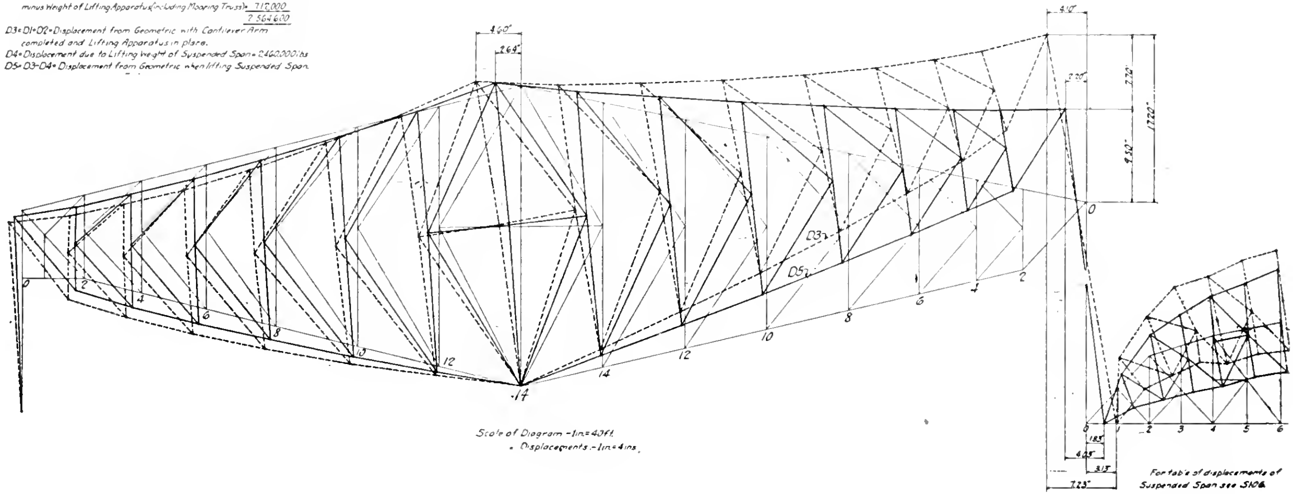


Figure 5

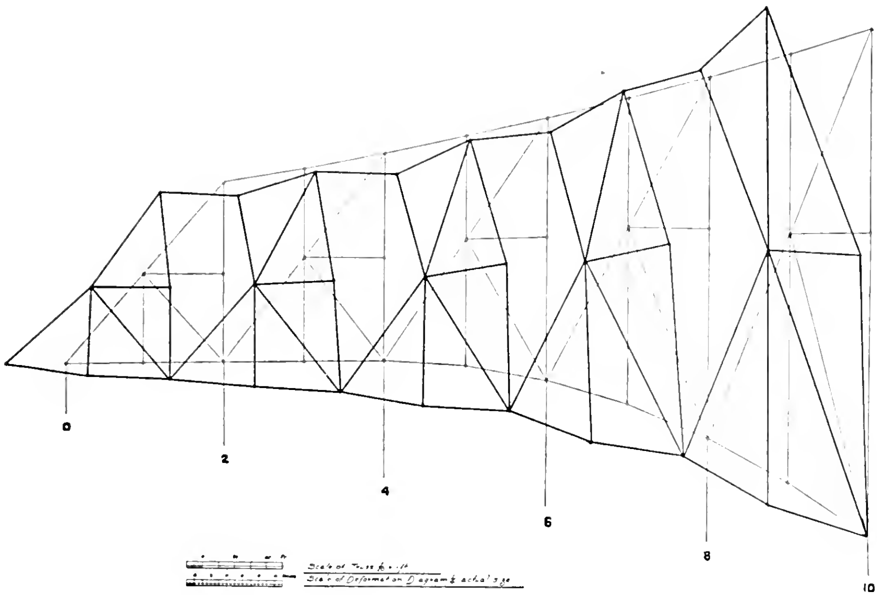


Figure 6

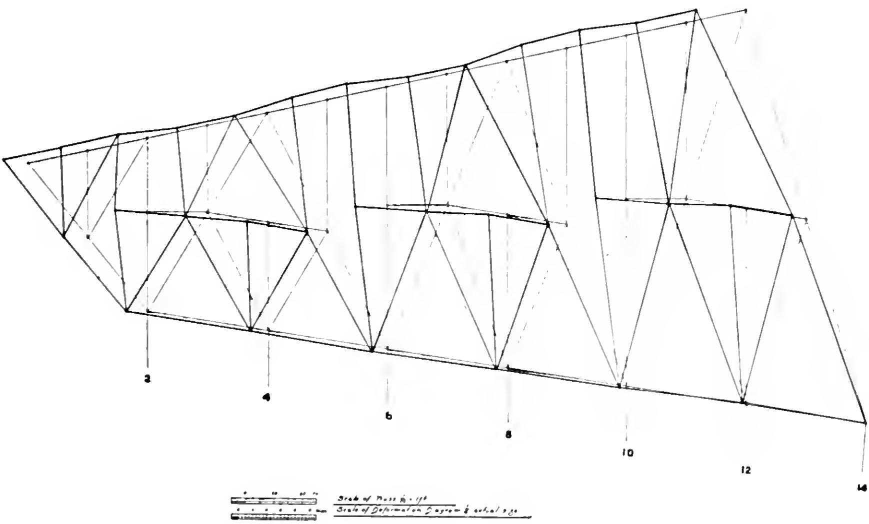


Figure 7

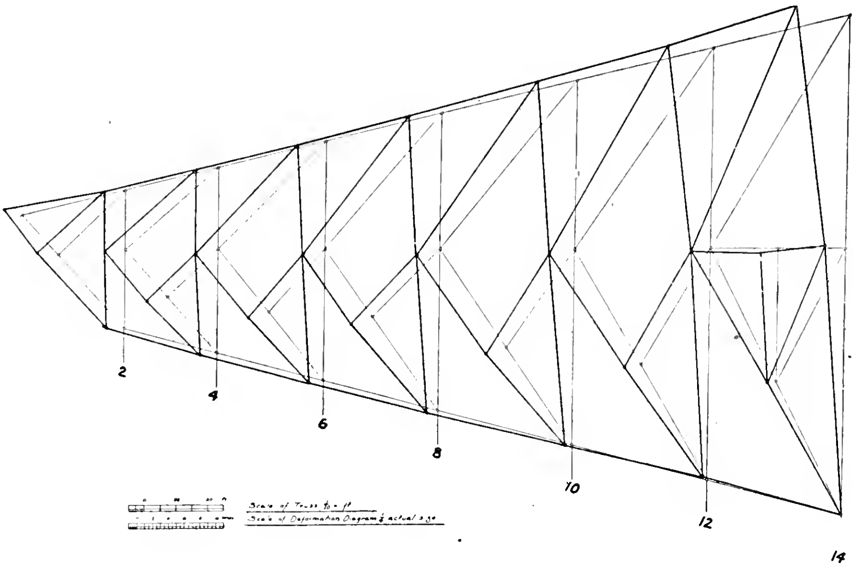


Figure 8

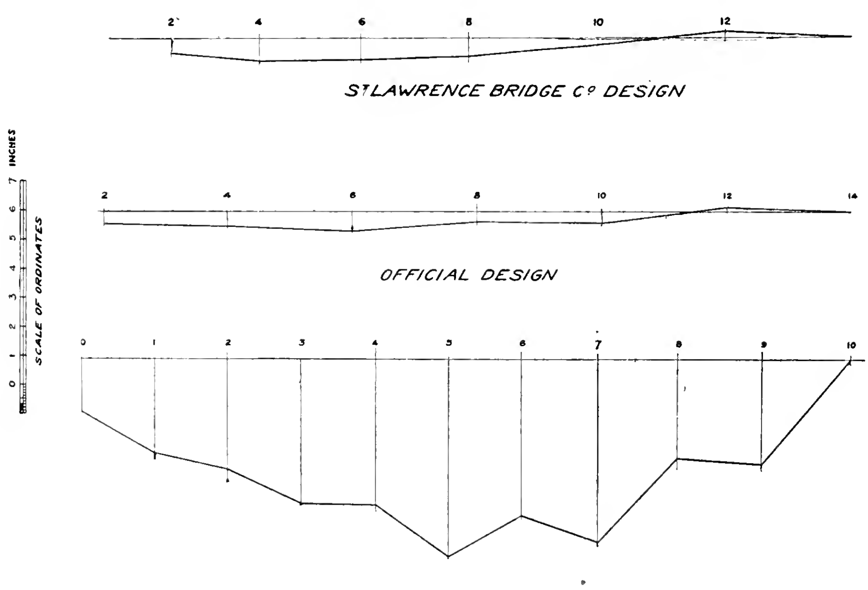


Figure 9

Figure 10. — Shows the connections of the web members of the bottom chord at the first main panel point of the Official Design, and the connections for a corresponding panel point of the "K" design. The Diagram also shows sections of the chords and compression diagonals of the Official Design and the "K" design.

There are many objectionable features in the details of the connections of the Official Design. The main compression members must be forked for a long distance in order to connect on pins in the center of the chord. The clearances of the long forked ends were necessarily small and called for great care in manufacture to permit the field connections to be made.

The pins on which these members abut carry heavy moments, and are very large. The plates to which the tension diagonals connect are very deep, and the pin holes a long distance from the bottom of the chord.

The pins in the centre of the chord prevented the use of centre diaphragms, and to provide the necessary lateral stability for each pair of webs they were connected by cover plates on the bottom flange with a row of lattice bars at the centre of the section and another row on the top flange. The section of the cover plate on the bottom was compensated for on top by narrow plates riveted to the flange of each web girder. While the area of the metal was thus properly distributed about the centre line, the section was unsymmetrical and required for lateral stability an unduly large proportion of lattice bars not carrying stress.

In the "K" design the stresses in the web members are practically halved, as there are two diagonals in each panel to carry the vertical shear. Owing to this and the more favorable angle of inclination of the diagonals, it was possible to make all connections on pins outside the chords, and to keep the webs of the compression diagonals in the same plane as the connection plates, thus relieving the pins of all moments and making it necessary to provide pins only of sufficient size to carry the direct bearing.

There were no pin holes in the centre line of the chord, and centre diaphragms were carried through between the outside webs, making an exactly symmetrical section that tested well, was easy to manufacture and handle, and considerably more economical in the weight of details than the Official Design.

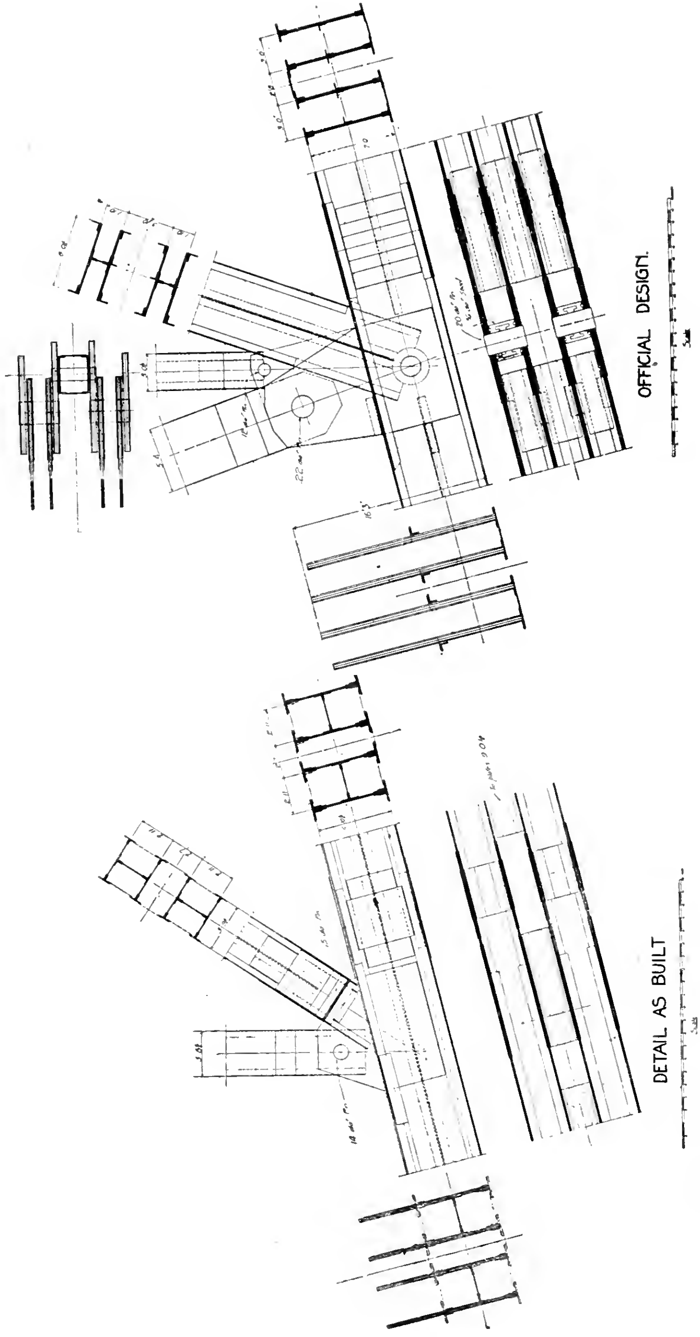


Figure 10

There was another advantage in the chords of the "K" truss not apparent from the sketches—namely, in the more uniform increment of load received at each panel point from the cantilever end to the shoe, permitting a more gradual increase of section, and for this reason better splices and better details throughout. This will be better understood when it is considered that in the Official Design there is no change in chord stress, due to vertical loads, from the shoe to the first main panel point, approximately 170 feet from the shoe : and, similarly, there is no change in the chord stress from this panel to the next main panel point, 170 feet further on. In the "K" design the main panel points occur at half the distance of those in the Official Design, and the increments of stress are correspondingly less and more often applied, so that the increase in the chord section at any panel point of the "K" design is only about half the increase at the main panel points of the Official Design.

Reference has been made to the large number of vertical members in the Official Design, and in our "M" design, introduced simply for carrying the weight of the top chord. It will be seen that in the "K" truss every member of the truss carries its proportion of live as well as dead load and there are no redundant members. As shown on the deflection diagrams of the Official and Phoenix designs, the redundant members cause considerable distortions of the frame, and the omission of these members which have no part in carrying the live load of the structure results in important economy.

The question of appearance must be largely one of individual preference, and it is perhaps difficult to compare designs shown only on a flat elevation. The redundant members in the Official Design and the varying inclination of the chords of the sub-trusses and stiffening struts, seemed to us to detract from its appearance. We think the symmetry and evident purpose of every member in the "K" design gives it a certain appearance of fitness and dignity.

When referring to the webs of the compression diagonals being practically in the same plane as the webs of the chords, attention was not called to the fact that this gave extra spacing between the outside webs, permitting each web to have double angles, thus making it symmetrical and of stronger section than the channel form of web, and moreover, giving a much larger and sufficient radius of gyration for the columns as they were shipped from the shop and erected, making it unnecessary to place much dependence on the lacing and tie-plates field riveted between these members after their erection.

Another important consideration from the purchasers view point is that, with the construction adopted, access for inspection and painting can be obtained to all parts of the interior of the chords and large compression members; whereas, with the long forked ends proposed for the Official Design there were of necessity many narrow spaces that could not be reached after the Bridge was erected.

The other comparisons have principally to do with the cost of material and operations in detail and cannot easily be set forth. We, however, estimated there were many economies in the "K" design.

Plate XV. — Shows a general elevation of the "M" design. It will be seen that the design is pleasing in appearance; the detailed estimates of weight were, as expected, considerably lower than for any of the other designs, being only about 85% of the next lightest, and it may be asked why we did not make a stronger effort to obtain the contract on this design.

Plate XVI. — Is a portion of the "M" design to a larger scale, showing the general character of the details and connections proposed.

Figure 11. — Shows the detail of the first main panel point in the bottom chord compared with the corresponding panel point of the "K" design.

In order to carry out the design logically and in such a manner that it could be erected safely, it was necessary to introduce pins at all panel points in the compression members and the use of pins introduced many objectionable features which Mr. Vautelet and the Board of Engineers had been at great pains to avoid in the Official Design, views in which we entirely concurred. When the design was started it was not known how these details would work out—indeed, as already pointed out, the details of any new design for a bridge of this magnitude could only be worked up by carrying the design through by the tedious method of working from the centre span and the cantilever ends back to the piers. Having carried the design to this stage of completion, we thought it worth while putting in a tender as a matter of record, although we did not in any way urge its claims.

It has been shown in discussing the open erection joints of the Phoenix truss, that a truss of this form could not be safely erected without providing for angular movement at every panel point of the compression member; hence the reason for the pins. It was doubtful, however, if ordinary pins could perform the intended function of permitting this angular movement because the movement is gradual as the load comes on and on some pins, when the cantilever arm has been extended out some distance, must be made under heavy stress. If movement were to be assured, it is probable, that the unit pressure on the pins would have to be much less than the bearing pressure allowed for the transmission of axial stress. This is provided for by large sleeves at the shoe-pins of the Official Design and of the Bridge as built.

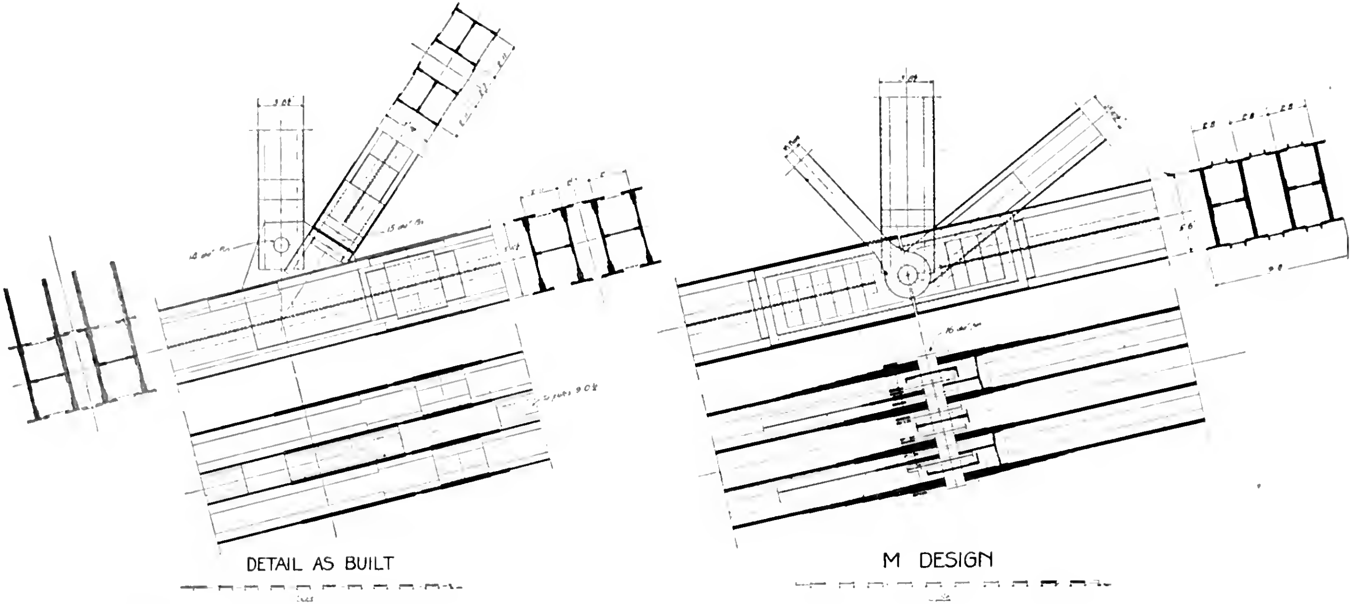


Figure 11

It will be seen that to provide pin bearing even for the stresses permitted by the specification, it was necessary to build up the chord webs of the "M" design to great thickness, and that in some instances all the reinforcing was placed on the outside of the web in order to make room for the long fork ends of the members connecting on the pin. This, which may be termed eccentric reinforcing, is of doubtful value even if the inner pin plates are made sufficiently long to assure that the rivets all act in shear.

The necessity of stopping the diaphragms towards the ends of the chord joints, thus making fork ends on the chords as well as on the web members, also seemed a very undesirable detail.

Added to the above considerations, we concurred with the Board of Engineers that these very large compression members should be as nearly as practicable continuous from end to end, so that the distribution of stress throughout the member should not be disturbed, and that the stresses would continue without being deflected past the splices and past the panel points where the member would receive a fresh increment of stress to be provided for by additional material. With pin joints it takes a great deal of reinforcing material to concentrate upon the pins the compression stresses carried by the outer edges of the web and the flanges, and these stresses must be again distributed over the section of the member after the joint is passed.

A question that may be asked is why the "K" bracing was not carried through the suspended truss, so many advantages having been found in its use in the cantilever arms and the suspended truss itself being of such very considerable length. "K" bracing was designed for the suspended span, and had we determined to erect this as a cantilever it would no doubt have been used, but it does not lend itself well to the parabolic form of top chord or to the low end heights which it seemed economical to use for this truss and, considering the method of its erection and the deformation diagrams, there seemed no objection to the form of truss used.

The discussion heretofore has referred to the designs submitted with the tenders and principally to Design "B" of the St. Lawrence Bridge Company recommended by the Advisory Board of Engineers for acceptance by the Government. That Design was for the specified lengh of 1758 feet centre to centre of piers, and its outline elevation necessarily differs from that of the present Bridge which has a span of 1800 feet centre to centre of piers.

Plate XVII. — Shows the general elevation of the Bridge as built.

The masonry of the old Phoenix Bridge was left in perfect condition after the accident, but the piers were too short to carry the projected span, it having been determined to increase the 67 ft. width of the Phoenix Bridge to 88 feet in the new Bridge. Mr. Vautelet had intended to make use of the old foundation of the pier on the South side, increasing its dimensions, however, both in width and length by sinking new caissons alongside, and to build a new pier on the North side of the River clear of and to the South of the old pier—the plan working out to a distance of 1758 feet centre to centre of piers with the centre line about 15 feet down stream from the centre line of the old bridge.

It was finally concluded that there might be great difficulty in sinking new caissons alongside the old foundations of the South Pier—indeed, it was thought by some that the proposed construction was quite impracticable. After Mr. Vautelet had resigned, the Board, in conjunction with the Advisory Engineers, recognizing the risk of at least serious delay from this cause, made a number of studies for using the foundations of the old piers. These studies were based on building up new masonry on the old foundations of the maximum length that the foundation caissons would permit, and reducing the weight upon the foundations by carrying a portion of the load on new piers to be built to the South of each of the present piers, thus giving four points of support for each cantilever instead of two as in the Official and the present design. Studies were also made for sinking three new pedestals on each side of the River and making use of the old foundations for the fourth pedestal.

Figure 12. — Shows the proposed design of the superstructure for one of those plans.

It was not found practicable to make the piers long enough for the width of bridge without too great intensity of pressure on the ends of the caissons, and the three new pedestals did not promise much economy. The plan to make use of any part of the old masonry or the old foundations was finally abandoned, and the Board recommended sinking new piers to the South and entirely clear of the old foundations but on the same centre line, thus restoring the span to 1800 feet.

In adjusting the "B" design to the new length of span the Advisory Board considered that the Bridge would have a better appearance if the low panels towards the ends of the cantilevers and of the suspended span were made shorter, in order that the inclination of the web members might be kept at about the same angle. In conference with the Board, the length of the suspended span was fixed at 640 feet and the cantilever arms at 580 feet. The span was divided into four more panels, one being added to each cantilever arm and two in the suspended span.

Mr. Vautelet had specified that the height of the steel work above the centre pier should not exceed 290 feet and had refused to permit any greater height. The Advisory Board saw no reason for this limitation and the inclined upper chords of the anchor and cantilever arms were continued until they intersected over the centre pier, the Board considering that this arrangement gave a better appearance, as it undoubtedly does on paper, than the two panels of flat chord over the piers. The intersection

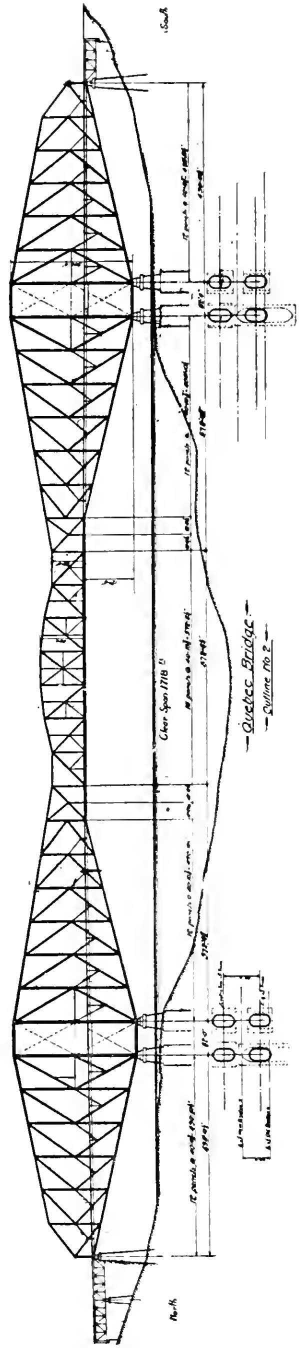


Figure 12

of the chords at this point made it necessary to extend the vertical posts to carry the shears from these chords and, having adopted this vertical post to omit the horizontal tie holding the heads of the first compression web members and replace it by inclined tie-bars, thus carrying the "K" system of bracing from the inclined posts at the end of both cantilever and anchor arms throughout all the panels to the main pier.

In making our competitive designs we were necessarily governed by considerations of economy, and endeavoured to adopt an outline that would result in the lightest structure consistent with the specifications and the other requirements sought, including good appearance.

The increase in span called for a heavier structure and the additional panels, as well as the changes in the pier panel, and also made considerable additions to the weight, but the Board considered that in a monumental structure of this character the extra weight was justified by the improved appearance. Personally, I am not at all convinced that the more economical arrangement of the "B" design would not have given an equally good appearance in the completed structure had it been adopted.

Plate XVIII. — Shows a cross section of the floor of the Official Design together with the live and dead loads above the railway stringers.

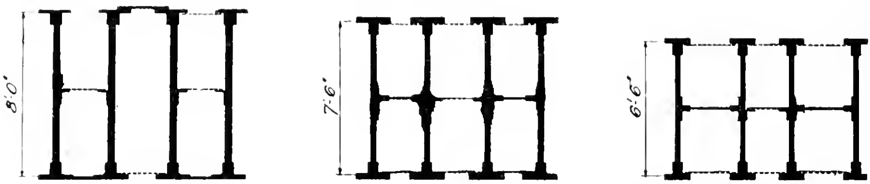
A cross section of the floor and the loads on which our Design "X" was based.

And a cross section of the floor of the bridge as built.

Throughout the consideration of the construction of the Official Design and of the alternative designs, we were faced with the problem of manufacturing and placing material having weight and dimensions far beyond any precedent, and it was constantly being forced on our attention that because of the span and the construction approaching the practicable limits it was of great importance to avoid any unnecessary weight either in live or in dead load.

A consideration of the heavy construction required to carry the highways specified, and the increase in the weight of the structure arising therefrom led us to make some approximate figures for a bridge to carry railway traffic only. These were so encouraging that they were followed up by complete designs omitting the highways, with the exception of a side-walk, which it was thought should be retained for the sake of inspection and such pedestrians as might wish to cross the bridge.

These designs were otherwise in strict accordance with the specification and were similar to the designs carrying highways. This plan saved 3100 lbs. per lin. ft. of superimposed load and 4,480 lbs. per lin. ft. in the weight of the roadway. The saving in the structure itself averaged 11,900 lbs of steel per lin. ft. and the finished structure was estimated to weigh only about 75% of that designed to carry the highways in accordance with the Board's specification.



2037 □"　　　1716 □"　　　1372 □"

OFFICIAL DESIGN No1　　　"B" DESIGN　　　"X" DESIGN

ST LAWRENCE BRIDGE Co

Figure 13

Figure 13. — Shows sections of the bottom chords next the main pier of the Official Design, the "B" design and the "X" design without the highways.

In addition to the economy in weight to be realized, there were many good reasons for advocating this change of design, as it brought the size of the members within more practicable limits for manufacture and erection and the small interests to be served by the highways seemed out of all proportion to the additional cost, and the risk and difficulties in construction that the highways as designed entailed. After the Advisory Board had recommended the acceptance of our tender for Design "B", designed to carry the full highway and electric car loads as specified and had modified the span and outline as above noted, we pointed out to them the economy and advantages to be gained by omitting the highways and, when the final contract was drawn, the specification and plans accompanying it omitted the provision for these highways.

In preparing our alternative design without the highways, it was not considered advisable to risk undue criticism by making any further change than necessary and the highways were simply left off, leaving the tracks as spaced in the Official Design. It was manifest, however, that advantages would accrue by separating the tracks as widely as the clearance of the sway bracing and the torsion of the structure would permit, these advantages being reduced stresses in the floor beams, a better staying of the top flanges of the floor beams and easier provision for traction stresses. The Board of Engineers readily accepted this change and the tracks were moved out to a width of 32'6" centres.

During the discussion of this change Mr. Monsarrat advocated a further change to the form of floor system used under his direction by the C. P. Ry. on some of its high viaducts. This floor, which is shown on Plate No. XVII consists in placing each track in a through plate girder bridge with steel stringers and floors beams of its own, and with exceptionally well braced and reinforced top flanges, so that in the event of derailment, the derailed rolling stock would be kept in this trough and have no opportunity to plunge down and wreck other members of the bridge.

Another change from the accepted design was in the use of eye-bars for the top chord. We followed the Official Design in the use of carbon and nickel steel, and tendered on all of our designs in this way, in order that our tenders should not be set aside through not complying with the requirements. We also gave alternative tenders on what we considered more economical proportions of carbon steel, but we tried to avoid using carbon steel where its use could be criticised on account of adding unnecessary weight to the bridge. While we tendered in some of our designs on using nickel steel eye-bars, experiments up to that time had not shown these bars to be entirely reliable, the quotations for them were exceedingly high compared to other material, and in our "B" design we estimated on plate nickel steel chords with riveted connections. Some of the members of the Board preferred eye-bar top chords, and after the contract was awarded we obtained quotations for carbon steel eye-bars, prepared estimates comparing the weights and cost of these with the material shown on the contract drawing, and under a supplementary contract substituted the eye-bar top chords in the final design of the bridge.

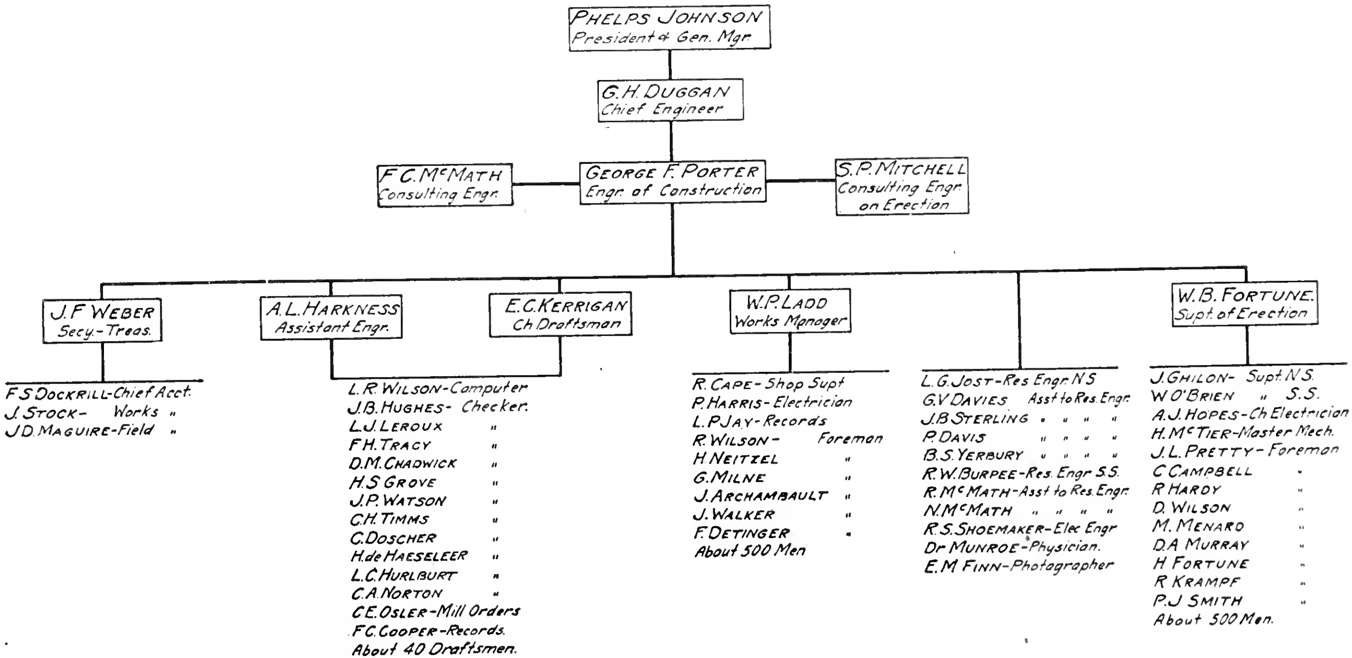
When discussing the changes in length of span and modifications of outline, the Advisory Board of Engineers also discussed some changes in the specification to bring its form more in accord with existing standard specifications without sensibly changing its requirements except, in the substitution of E-60 for E-50 locomotives; and when it was finally decided to build the bridge for railway traffic only, a new specification covering this condition and the other changes was drawn up.

The specifications finally issued by the Board of Engineers provided for a double railway track, but for no highways, except two sidewalks five feet wide. They will be given in full in the forthcoming paper.

The greatest care was taken to comply with the requirements of the specification, and to submit with our tenders all the information called for therein. Had there been no change in the length of span, our plans were ready for the preparation of shop drawings, but the changes already enumerated necessitated an entirely new set of computations and, while the details and the make-up of the members remained of the same type as originally designed, modifications were necessary in nearly every instance by reason of these changes.

Before tendering it was necessary, for purposes of estimating, to make approximate plans of shops and equipment, the erection travellers, falsework and other equipment required in the field, so that we might assure ourselves of the practicability of the work and make close estimates of the cost of the whole. These sketches were, however, not working drawings and as soon as the contract was let it was necessary to organize for carrying out the new design, the preparation of shop drawings, the manufacture and the erection of the structure. We felt that great perfection and a new standard of shop work would be required, that there was so much to be

ORGANIZATION OF ST. LAWRENCE BRIDGE CO. LTD



done, so many unusual problems to consider and so short a time in which to do it, that we must employ the best engineering and manufacturing advice that could be obtained.

Figure 14. — Shows the working organization of the St. Lawrence Bridge Company for the construction of the bridge.

Mr. Johnson as President kept an engineering as well as a business supervision of his charge.

Mr. McMath was available for consultation when required, all drawings were submitted to him and were examined by him personally. Mr. Willard Pope, Chief Engineer of the Canadian Bridge Company, rendered much valuable assistance, personally checking all strain sheets.

Mr. Porter was put in immediate charge of the Engineering force as Construction Engineer. To him belongs a very large share of the credit for the conduct of the work. In addition to his assistance in preparing the details for our tender designs, he acted as Resident Engineer throughout, overseeing the preparation of shop drawings and the detailed design of erection equipment at Montreal during the Winter, and during the summer residing at the site in responsible charge of the field operations. From the outset, the work under his direction was carried on so satisfactorily that much more responsibility than generally falls to the position was placed upon him with the utmost confidence.

The Canadian Bridge Company released Mr. W. P. Ladd, then Superintendent of their shops at Walkerville, as Works Manager of the new shops at Rockfield and it became his duty to take charge of the layout, purchase of equipment, organization and, indeed, everything in connection with the manufacture of the bridge and the erection equipment. The excellence of the shop work and the wonderful precision in the lengths and fitting of the various members have been commented on by every engineer who has viewed the work. This excellence contributed much to the facility with which the bridge was put together in the field, and we all realize that the work of Mr. Ladd and his staff was one of the most important factors in the operations.

We felt that the erection, involving as it did the lifting and placing of the heaviest pieces heretofore handled, and by far the largest tonnage in one span, required the best experience in heavy work and the best expert advice that could be obtained, and we were fortunate in securing the services of Mr. S. P. Mitchell as Consulting Engineer on erection, he being considered best qualified to supply the experience and advice needed. Mr. Mitchell devoted a large portion of his time to the consideration of the erection equipment, secured for us the services of Mr. W. B. Fortune, our General Superintendent of Erection, and assisted in organizing the field force.

Where so many experienced men have been employed and given their best thought to the work over a period of about six years, it is impossible to particularize further, but it will no doubt be appreciated that many helpful suggestions and much assistance were received from the Assistant Engineer, Mr. Harkness, the Chief Draftsman Mr. Kerrigan and his staff, the Superintendents and the Inspectors both in the shop and in the field. The accurate work of Mr. Jost, Mr. Burpee and the field engineers should not be overlooked.

Reference must be made to our harmonious relations with the Board of Engineers and the assistance received through the hearty co-operation of its Members. The experience and judgment of Mr. Modjeski, Mr. Charles MacDonald and the Advisory Engineers when discussing the changes in plan was of great value, and it goes without saying that anything with which Mr. Schneider had to do must bear some evidence of his knowledge and ripe judgment in all matters pertaining to steel construction. Notwithstanding Clause 5 of the Specification which threw the entire responsibility of the design, material and construction upon the Contractor, the Board of Engineers organized an experienced and highly efficient staff of engineers and inspectors, and our work was much facilitated by the co-operation of the Board and its staff. Every stress sheet and every calculation was investigated and checked, and every detail was checked to the last rivet. Material was inspected at the mills, in the shop, and in the field. Workmanship was most carefully inspected both in the shop and in the field, and all field engineering (lines, levels and measurements) was carefully checked. It was very reassuring to have this supervision and to feel that it was practically impossible for an error to escape unnoticed.

THE DESIGN, MANUFACTURE AND ERECTION OF THE SUPERSTRUCTURE

of

THE QUEBEC BRIDGE

by

PHELPS JOHNSON, M.E.I.C., G. H. DUGGAN, M.E.I.C., and GEORGE F. PORTER, M.E.I.C.

THE project of the long span required for the Quebec Bridge has been before the engineering world for a long time, it being over twenty years since the specifications and call for tenders issued by the Quebec Bridge and Railway Company, under date of 1st September 1898, brought the bridge prominently before the profession. The subject was at once taken up with great interest as it was realized that the magnitude of the work would call for the solution of new and untried problems, conditions in Canada not permitting the methods of construction used at the Forth Bridge—the only other bridge that at all approached the Quebec Bridge in magnitude.

Much has been written on the subject but it is scattered through the engineering papers of a long period, and the engineers responsible for the design and construction of the bridge as built feel that a description of the work should be on record in readily available form within the Transactions of the Engineering Institute of Canada, all the Engineers taking part in the construction being members of the Institute.

There may seem some plagiarism in what follows, some of the description and many of the drawings having been already published; but as all of the data for these articles was taken from the designs, drawings and notes prepared by the St. Lawrence Bridge Company, the Authors feel entitled to present it a new as original matter.

An introductory paper was read before the Society on January 10th, 1918, entitled " Notes on the work of the St. Lawrence Bridge Company in preparing the accepted design for the construction of the Quebec Bridge." That paper refers to the important features of the official design on which tenders were called and the difficulties forseen in erecting it; it discusses the design of very long span bridges in general terms, the advantages of the " K " bracing and outline the considerations leading to the design of the St. Lawrence Bridge Company recommended by an Advisory Board of Engineers for acceptance, and on which the contract was awarded. These subjects are therefore either not discussed or only touched on in this paper.

The introductory paper reviews the history of the undertaking; the disagreement among and the changes in the personnel of the Board of Engineers, the setting aside of the official design and the adoption of the Contractor's design.

These events may be briefly summarized chronologically as follows:

29th Aug. 1907. Bridge under construction by Phœnix Bridge Company failed.

31st Aug. 1907. Government appointed a Royal Commission to investigate the accident.

17th Aug. 1908. Government appointed a Board of Engineers, to "prepare plans and construct the bridge on these plans."

Mr. (Now Sir) Maurice FitzMaurice, C.M.G., M. Inst. C.E.

Mr. Ralph Modjeski, M. Am. Soc.C.E.

Mr. H. E. Vautelet, M.Can.Soc.C.E., Chairman and Chief Engineer.

Mr. Vautelet completed plans and specifications about the 1st of January 1910, but the other members of the Board did not fully approve of his design, believing that a more practicable design could be made, and consented to tenders being called upon the official design only on condition that bidders might submit tenders on their own plans if they so desired.

1st June 1910. Mr. FitzMaurice resigned.

17th June 1910. Advertisement inviting tenders on Mr. Vautelet's official design and on Contractor's alternative designs.

28th Sept. 1910. Mr. Chas MacDonald, Past Pres. Am.Soc.C.E., appointed to succeed Mr. FitzMaurice.

1st Oct. 1910. Tenders received and opened by the Government.

20th Jan. 1911. The Government appointed M. J. Butler, C.M.G., Past President, Can.Soc.C.E., and H. W. Hodge, M. Am. Soc. C.E., to assist the Board in selecting a design.

8th Feb. 1911. Messes. Modjeski, MacDonald, Butler and Hodge reported in favor of the St. Lawrence Bridge Company's design. Mr. Vautelet dissenting.

28th Feb. 1911. Mr. Vautelet resigned from the Board.

The remaining members of the Board then changed the span from 1758 feet to 1800 feet, and the masonry plans, modified the specifications and, in conjunction with the Contractors, modified the design but retained all the essential features of the design submitted with the tender.

4th April 1911. The Contract was signed, the design for the bridge was signed as part of the Contract, as were also the specifications.

When the Contract was signed Mr. MacDonald desired to be relieved of further detail work and Messrs. Butler and Hodge ceased to be members of the Board by the terms of their appointment, thus leaving Mr. Modjeski the sole remaining member of the Board that had decided on the plans and specifications.

It is pertinent to note that after this date when the contract was signed, no change could be made in the design or specifications except by agreement between the Contractors and the Board of Engineers. The calculations, all the working drawings and all the special methods employed in the construction were entirely developed by the staff of the St. Lawrence Bridge Company — the Board of Engineers thereafter acting in a supervising capacity only.

6th May 1911. Mr. C. N. Monsarrat, M.Can.Soc.C.E. was appointed to the position made vacant by Mr. Vautelet's resignation.

15th May 1911. Mr. C. C. Schneider, Past Pres. Am.Soc.C.E. was appointed to complete the Board of three.

8th Jan. 1916. Mr. C. C. Schneider died.

Feb. 1916. Mr. H. P. Borden, who had been on the Engineering Staff of the Board from the beginning was appointed to fill the vacancy created by the death of Mr. Schneider.

Reference was made in the earlier paper to the exceptional responsibilities for the DESIGN and ENGINEERING of the structure placed upon the Contractors in the clause of the Contract which required the Contractor.—

"To GUARANTEE the satisfactory erection and completion of
" the Bridge and to undertake the ENTIRE RESPONSIBILITY not

"only for the materials and construction of the bridge but also for "the DESIGN, CALCULATIONS, PLANS and SPECIFICA-"TIONS and for the sufficiency of the bridge for the loads therein "specified".

The Contractors welcomed this responsibility, having every confidence in their design and plans for construction and being thus relieved by the terms of the contract from the possibility of being asked to adopt plans or methods of construction of which they did not approve.

Reference was also made to the organization of the St. Lawrence Bridge Company. This Company was specially incorporated in the joint interests of the Dominion Bridge Company and The Canadian Bridge Company to combine their organizations and resources for the execution of the work. The working staff of the St. Lawrence Bridge Company is shown on Fig. 14. Further introduction is believed unnecessary.

The present paper describes the bridge as built and the methods of construction.

The paper is prefaced with a map of the vicinity, of the site, notes of the River conditions, tides, currents, etc., all of which may assist in a better appreciation of the conditions governing the delivery and storage of material and the erection as well as of the decision to make no provision for electric railways independent of the steam railway tracks or for vehicle traffic.

It has been stated that the principal consideration leading to the adoption of the "K" form of truss bracing was the difficulty of erecting all other forms tried and the comparative safety and facility of erecting the "K" form. The problem of placing and connecting the important members of the structure was constantly before the Designers and, even after the general form of the truss and the make-up of the members were determined, the erection of each member and its field connections were always carefully considered in making the working drawings. It is thought, therefore, that it will facilitate an understanding of the design and construction to preface the detailed description of the work with a general outline of the methods of erection adopted, giving a short argument for their adoption and for the elimination of other methods considered. Reference is also made to the traveller and other appliances so far as this equipment affected the stresses and the details of construction. A more extended description of all important erection equipment is given in the chapter on erection.

After the outline of erection methods there follow plans illustrating the completed structure with a description of the general design and details.

The manufacture and handling of the large members entering into the structure was beyond the capacity of any equipment existing in Canada at the time the work was undertaken, and it was necessary to build and equip special shops. The shops and their equipment are briefly described as well as some of the methods employed in the manufacture.

The chapter on erection touches on the general accommodations for the field forces, the preparations for storage and equipment for handling material at the site. It describes the equipment and the method of erecting the important members in the structure. While the latter description is tedious, it is felt that the record may be interesting to some — safe and easy erection having been the motif of the Designers in preparing the plans for the bridge.

Summarizing the above outline, the following is a brief index to the several chapters:

I General description of the site and conditions.

II Outline of the methods of erection and transportation with reference to their effect upon the design of the superstructure.

III Description of the structure as built, including the design.

IV Description of the shops and equipment with reference to the unusual methods of manufacture.

V Field operations and erection.

- (a) Camp and storage at site;
- (b) Erection Travellers;
- (c) Erection of approach spans;
- (d) Steel falsework and staging;
- (e) Erection of the north anchor arm;
- (f) Erection of main posts;
- (g) Erection of the cantilever arm;
- (h) Erection of the suspended span at Sillery;
- (i) Equipment for floating, mooring and hoisting the suspended span;
- (k) Floating and hoisting the suspended span in 1916;
- (l) Hoisting and coupling in place of the suspended span in 1917;
- (m) Final operations to complete;
- (n) Erection details;
- (o) Accuracy of work;

Several subjects which may be found useful for reference which are not necessary to an understanding of the structure as constructed, are treated in appendices.

APPENDIX "A"

In discussing the broad considerations which led to the adoption of many features of the present design, reference was made to tests carried out by the Royal Commission and by the Board of Engineers. Additional tests were called for in the original specification and the Engineers of the St. Lawrence Bridge Company made still further tests for their own information and guidance. The deductions from many of these tests find their expression in the specifications and in the design of the compression members; those relating to end details of tension members and riveted tension splices were carefully considered in the design of the details. A summary of these tests is given in Appendix "A".

APPENDIX "B".

The calculations for both primary and secondary stresses, being of a comparatively simple character, did not require the development of any new methods and naturally followed old and well established practice. It is thought, however, that the calculations were well arranged both for computation and reference and that it may be of interest to have a record of how these were carried out.

Mr. A. L. Harkness has written a memorandum on this subject.

Chapter I

GENERAL DESCRIPTION OF THE SITE AND CONDITIONS

Figure 15. — Is a photograph of the completed bridge.

The Quebec Bridge is now a link in the Canadian National Railway, which saves about 219 miles between Moncton and Winnipeg; the distance from Levis to Winnipeg by the N. T. Ry. being 1355 miles, while the distance via Montreal and the Ottawa River is 1574 miles.

Plate XIX. — Is a map of the vicinity of the Bridge site showing the connections of the railway lines crossing the bridge to the main lines of the C.N. Railway and the branches leading into the Cities of Quebec and Levis. It also shows the main highways in the vicinity of the bridge.

Plate XCIX. — Is a chart of the River from Sillery Cove on the East to a short distance above the Bridge on the West.

Plate XX. — Is a general elevation of the Bridge showing the profile of the River at the crossing. On the North side steep rocky cliffs extend for some distance on either side of the Bridge with only a very narrow beach at high water, and for about a third of a mile to the East of the Bridge high water comes to the base of the cliffs, cutting off the road connection. The shore between high and low water is flat and covered with large boulders. At about the point of low water, or a short distance outside, the bottom falls away rapidly. On the South side the conditions are somewhat different, the beach being much wider. The South main pier is dry at all stages of the tide, while on the North side there is about 25 feet on the outside of the pier at low water.

The City of Quebec is situated at the East end of a high narrow spur which is almost an island, the deep valley of the Cap Rouge River at this point cutting it off from the high banks to the Westward. The St. Charles River flows into the St. Lawrence just below the City of Quebec after traversing a course almost parallel to the St. Lawrence and the valley of the St. Charles joins that of the Cap Rouge River at a comparatively low elevation.

The main line of the Canadian National Railway crosses the Cap Rouge River on a high single track trestle 3,335 feet long, at an elevation of 154 ft. above mean high water. The railway then skirts the side hill of the Quebec Cape at almost a level grade until it reaches a point just to the East of the bridge where it begins to fall to Sillery Point, and from that point maintains its level to the projected terminal just below the Terrace at Quebec.

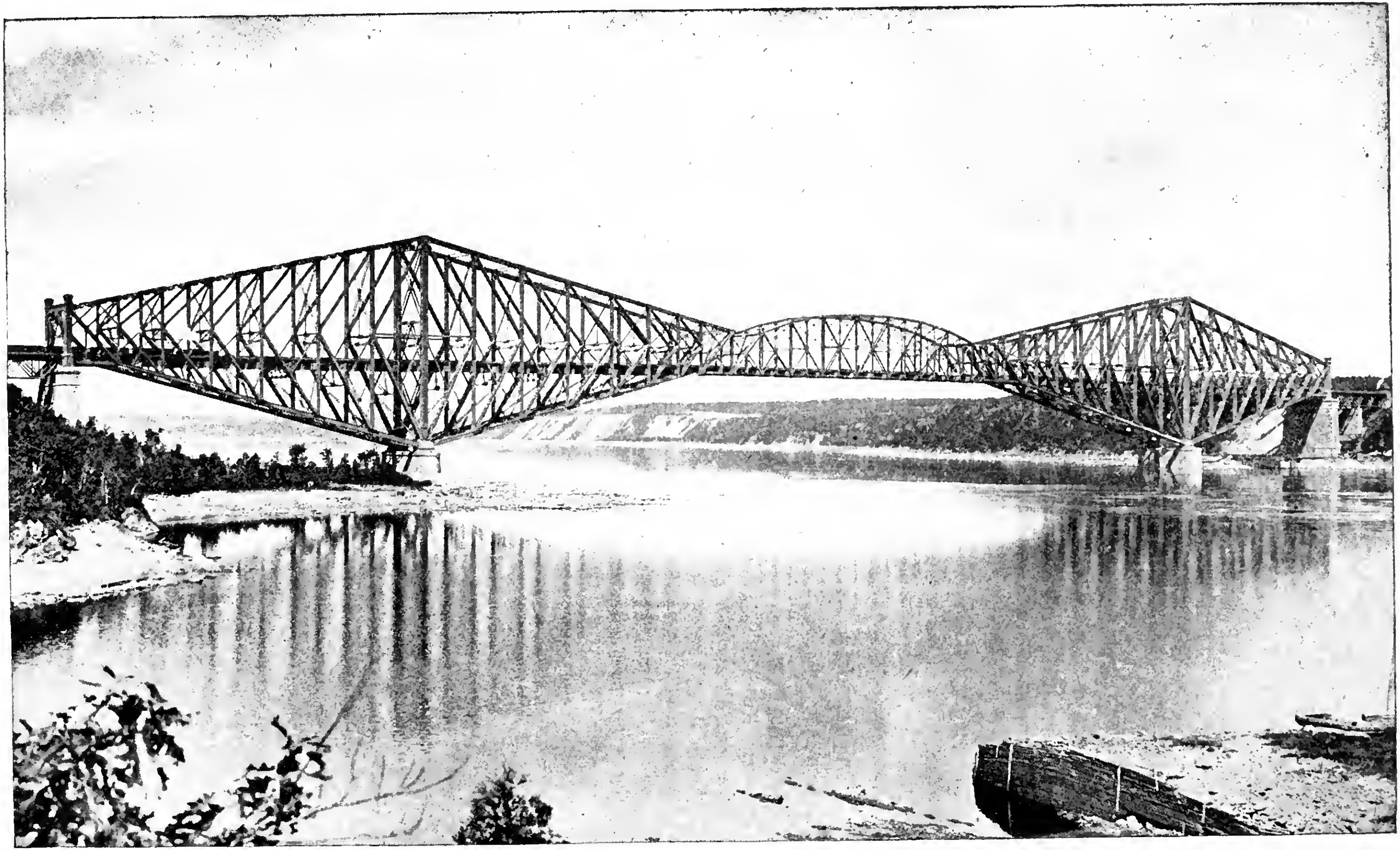


Figure 15 Photograph of Completed Bridge.

It may be noted that at the time the contract was let it was intended to establish the Quebec Terminals at this point, but subsequently arrangements were made with the Canadian Pacific Railway for joint use of the C.P.R. terminals on the St. Charles River, the connection being made up the Valley of the Cap Rouge River around the West end of the Quebec promontory.

A car ferry service during the construction of the Bridge was also established from a wharf about half a mile West of the end of the track opposite the Quebec Battlefields to the Intercolonial Railway wharf at Levis.

The Quebec Bridge is nearly at right angles to the line of railway described above, and is connected to it by curves in both directions forming a " Y " in which the material storage yards were established. This " Y " and the yards were on a wide bench about rail level, but there was a high narrow rock ridge near the edge of the cliff which had to be cut through to give entrance to the bridge. The bench to the Westward of the Bridge was levelled down with the intention of establishing a large railway yard and shops. A roundhouse and much of the yard was put down but the repair shops were later established on the St. Charles River at St. Malo. On the South side the rail level at the abutment is considerably above the bank and the railway is carried for some distance on a high embankment through which passes the shore road leading to Levis.

The Cities of Quebec and Levis are connected by an excellent ferry service, and ferries run occasionally from Sillery Point to New Liverpool. The shore road at the South end of the bridge was comparatively unimportant, the greater part of the highway traffic into Levis being from the roads leading from a more Southerly direction. On the North side the St. Louis Road, the main highway opposite the bridge, has little traffic originating to the West of the Bridge, owing to the steep grade up the bank of the Cap Rouge River and practically all of the traffic is destined for Quebec. Connections for highways on the bridge to the road on the South side of the River could have been readily made, but the connections on the North side presented a good many serious difficulties; the deep rock cut at the entrance to the bridge, the crossing of the railway yards and the connection to the St. Louis Road, which at this point is at a considerably higher elevation than the bridge floor.

The above considerations coupled with the greatly enhanced cost of the superstructure if highways were provided led to the decision to omit the highways.

The tides at Quebec are heavy. The normal Neap tides have a range of about 12 feet and the normal Spring tides of about 18 feet. Occasional tides have a larger range up to 21 feet. In ordinary conditions of weather the heights of low and high tide as well as the periods can be very closely predicted from the Tide Tables. The average period of flood tide is about

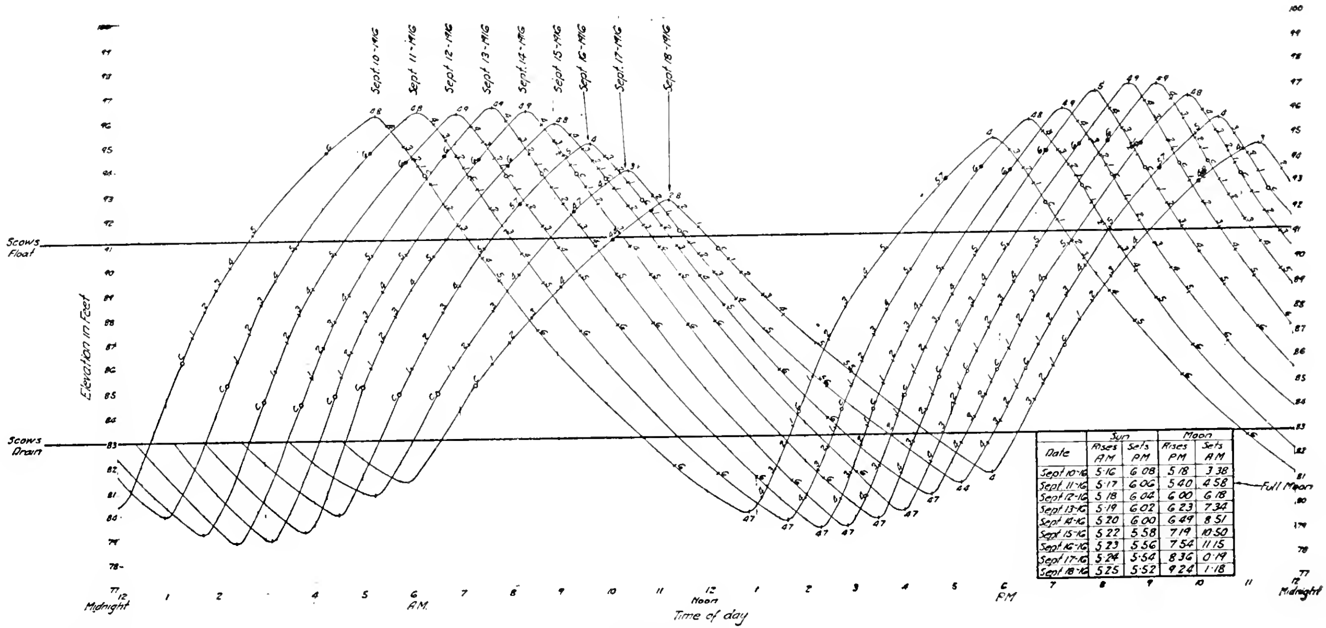


Figure 16

Tide Curves.

five hours and of ebb tide about seven hours. The high tide begins to fall about an hour before the change in direction of current and the low tide begins to rise also about an hour before the current changes in direction. At the Bridge site serious cross currents often occur, and the River men state that these are most troublesome at the end of the flood tide.

Figure 16. — Shows a series of tide curves, plotted from the Tide Tables, about the period selected for floating the suspended span in 1916. It well illustrates the general tide conditions.

To the West of the Bridge site there is a straight reach of the River from one to three miles wide for about twenty miles; the high banks continue on either side of this reach. To the East of the Bridge site the River is narrower and not so straight and the site is comparatively sheltered from the East, but from both directions a heavy sea makes at the bridge crossing, particularly when the tidal current is against the wind. Owing to the geography of the River outlined above, there is no shelter from a wind up or down the River, but strong cross winds are very infrequent.

The Board of Engineers established a wind gauge with an area of 37.2 sq. ft., 54 feet above mean high water on the Contractor's wharf near the North Pier, but it was rather too low to give accurate records of the pressures that would occur at a higher elevation against the structure, and as the anchor arm was built it was to some extent sheltered. Its use was therefore practically abandoned at the end of the year 1914. The maximum pressure recorded on this gauge was 10.6 pounds per sq. ft. in December, 1914. As the substructure was erected anemometers were established at the highest and most exposed points.

The foundations and all masonry work were constructed by Messrs. M. P. & J. T. Davis, Contractors.

Plate XXI. — Shows the general dimensions and layout of this masonry.

Chapter II

OUTLINE OF THE GENERAL SCHEME OF ERECTION IN SO FAR AS IT AFFECTED THE DESIGN OF THE SUPERSTRUCTURE:

Plate XXII. — Indicates the order in which the different parts of the structure were erected.

Following the order of designing, the erection of the suspended span was first considered. Although tenders had been submitted for erecting the official design by cantilevering, in order to fully comply with the specifications, there was no expectation that these tenders would be accepted. For the St. Lawrence Bridge Company's alternative designs, plans were made and estimates prepared for erecting and floating the suspended span at the high level which it would eventually have in the structure and also for erecting and floating it at a lower level and hoisting it to its place in the bridge after it reached the site. The high level scheme had many objections which became apparent after the drawings were prepared and the site investigated, but tenders were made on it on account of the prejudice known to exist against hoisting the suspended span into position, the method actually used.

The span was erected on steel falsework having corner towers sufficiently strong to carry the span. After the intermediate staging bents were released and removed, barges were floated under it and the span blocked on top of the barges which rested on wooden sills bedded in concrete. Valves in the bottom of the barges permitted the tide to flow in and out until it was desired to move the span, when the valves were closed at extreme low tide.

A favorable site for erecting the span was found at Sillery about 3½ miles to the Eastward of the bridge site. This site had a flat rock bottom, dry at most low tides, affording good foundations for the falsework and making it easy to prepare them. The Railway connecting the Material Yard at the Bridge with the Market Street Station at Quebec was about the elevation at which it was desired to erect the floor of the suspended span and swept around Sillery Cove in a curve, allowing an easy connection leading directly on to the falsework set for the erection of the span.

Plate XXIII. — Is a plan of Sillery Cove showing the position in which the span was erected.

From the time it was decided to erect the centre span on falsework and float it into position it was foreseen that the greatest risk in placing would be when the suspended span was being anchored between the cantilever arms in order that the suspension connection or the hoisting connection as the case might be could be made. There were several factors in this risk. The range of tide of about 18 feet at the time of placing caused comparatively rapid vertical movements of the water surface; the strong current reaching about 7 miles per hour only a short time before the turn of the tide; the cross currents at or near slack water, and the large wind surface of the span and falsework, all making it difficult and hazardous to move and place the span in the position required without fouling and damaging some of the permanent structure either of the cantilever arms or of the span.

Important considerations in favor of low floating were the high and very expensive falsework and the high centres of gravity and wind pressure for the elevated position of the suspended span. This raising of the centres of gravity and pressure required long pontoons to give stability, greater displacement in the pontoons to carry the weight of the falsework, and it would have been necessary to make the pontoons strong enough in themselves to carry the weight of the falsework and span to the foundations, as there seemed no practical way of floating the pontoons under after the span had been erected on the falsework. The extra length and draft of these pontoons would have added considerably to the difficulty of controlling them when moving the span and to the risk in placing it in position. Moreover, there was no suitable site to make foundations for these longer and deeper pontoons. The site at Sillery while excellent for the pontoons used, could not well have been adapted to anything requiring greater draft. The actual method and arrangements for hoisting the suspended span will be given in Chapter V.

The traveller which erected both the anchor and cantilever arms and set the falsework of the anchor arm, is shown on plates XIII and XIV. Its total weight was about 920 tons, almost equally distributed on front and rear trucks with both cranes placed at the rear end. When lifting its maximum load, the reaction under the forward posts was about 1,300,000 lbs. on each leg and for some conditions there was an up-lift at the rear end of 59,000 lbs. The traveller ran on rails laid on the top flanges of the permanent bridge stringers, and other stringers temporarily laid on the floor beams to suit the gauge of the traveller truck wheels, with temporary lateral bracing between the top flanges and sway bracing as required. The stresses resulting from the traveller load did not call for increases of sections in the permanent structure.

Briefly, the plan followed in erecting the anchor arm was: The main shoes were set accurately on the pier; the bottom chords were laid from the shoes towards the abutment in a perfectly straight line and the joints were completely riveted. After the bottom chords were fully riveted and made continuous from end to end they were jacked down to the position required to give the calculated camber in order that they might be straight when under load in the finished structure. The traveller set its own falsework as it proceeded towards the centre pier to erect the shoes; set the chords as it retreated towards the abutment and was then taken back to the centre pier to erect the lower half of the web bracing and complete the triangles resting on the bottom chord. The upper triangles were then started from the abutment and as the traveller advanced toward the pier it completed the erection behind it.

The above arrangement as carried out worked perfectly, but at first it seemed that it would be impracticable as the deformation diagrams showed that the upper triangles could not be connected without a considerable bend in some of the members, necessitating very heavy forcing.

The difficulty of connecting the upper triangles was overcome by slightly elongating the pin holes in some of the members. Tests were made, as referred to in Appendix A — to ascertain what effect the elongation of the hole might have upon the strength of the member, and it was found to be in no way detrimental.

All field splices in the compression web members were fully riveted as the work was assembled, thus avoiding any possibility of open joints. This made it necessary to slightly spring some of the compression members in order to match the pin holes when connecting an " M " joint or when completing a bottom chord panel. The springing of the compression diagonals was accomplished by means of wire tackles, but for the bottom chords of the cantilever arm it was found desirable to use a temporary platform on which the chords could rest while being assembled and riveted and from which they could be jacked to the required position. Generally speaking, the cantilever arms required very little special provision for erection conditions.

Chapter III

PLANS AND DESCRIPTION OF THE STEEL SUPERSTRUCTURE

Plate XX. — Shows the outline of the structure with the governing dimensions. The clear head room of 150 feet above high water required by navigation interests and the rail level on either side made it economical to use a falling grade of one percent from each end of the suspended span.

Specifications:

The specifications were determined by the Advisory Board of Engineers at the time the contract was signed.

The loads and unit stresses for which the bridge was designed are as follows:

Loads—The loadings for which the bridge will be calculated are as follows:—

A. *Train Load*—Two Class E60 Engines, followed or preceded, or followed and preceded by a train load of 5,000 lbs. per foot per track, on one or two tracks. Where empty cars weighing 900 lbs. per lineal foot of track in any part of a train produce in any member larger strains than the uniform load of 5,000 lbs., such empty cars shall be assumed.

B. *A Sidewalk Live Load* of 500 lbs. per lineal foot for each of two sidewalks.

C. *Dead Load*—The weight of all material remaining in the completed bridge, and a snow load of 500 lbs. per lineal foot of bridge.

The weight of railway floor above stringers to be assumed at 860 lbs. per lineal foot of each track. Timber to be assumed to weigh 5 lbs. per ft. B.M.

D. *Erection Loads*—The weight of loaded travellers, erection plant and materials.

E. *A Wind Load* normal to the bridge of 30 lbs. per square foot of the exposed surface of two trusses and one and one-half times the elevation of the floor and 120 lbs. per lineal foot of bridge on sidewalk fence (fixed load), and also 30 lbs. per square foot on travellers and falsework, etc., during erection.

F. *A Wind Load* on the exposed surface of the train of 300 lbs. per lineal foot applied nine feet above base of rail. (Moving load).

G. *A Wind Load* parallel with the bridge of 30 lbs. per square foot acting on one-half the area assumed for normal wind pressure. (See paragraph E).

H. *Tractive Force* assumed at 750 lbs. per lineal foot on one track.

I. *Temperature*—A variation of 150 degrees Fahr. in the uniform temperature of the whole structure.

J. *Temperature*—A difference of 50 degrees Fahr. between the temperature of steel and masonry.

K. *Temperature.* A difference of 25 degrees Fahr. between the temperature of a shaded chord and the average temperature of a chord exposed to the sun.

L. *Temperature.* Stresses due to a difference of temperature of 25 degrees Fahr. between the outer web exposed to the sun and the other webs of compression members.

M. *Impact* from railway load shall be assumed as 100% for stringers; 75% for floorbeams and truss members carrying one panel load or less; 50% for hangers carrying two panel loads and 20% for all main truss members. For trusses of approach spans

$$\text{Impact} = S \frac{300}{300 + L},$$ where L is the length of loaded track, and S = total stress from railway load.

Loads Used to Determine Section of Members—All the co-existing loads and stresses and the deformation shall determine the section of the different members with the following restrictions:—

Load "A" shall be used in all calculations where not otherwise provided;

Load "B" shall be used for floorbeams and sidewalk stringers, and members receiving their maximum strain from a length of moving load covering two panels or less;

Stresses produced by "L" shall be considered as secondary stresses, and loads "K" and "L" shall be assumed to co-exist with one-half wind loads "E" and "F."

UNIT STRESSES AND PROPORTIONING OF PARTS

Unit Stresses (Carbon Steel). All parts of the structure shall be proportioned so that the sum of the maximum stresses produced by the loadings specified, including impact, shall not exceed the following amounts in pounds per square inch.

	Lbs. per sq. in.
Tension:	
Eye-bars	20,000
Riveted members	18,000
Including secondary stresses	24,000
Compression:	
Short members with l/r 50 and under	14,000
Long members with l/r over 50	17,500-70 l/r
Including secondary stresses	18,000
Bearing:	
Shop rivets	22,000
Field rivets	20,000
Rollers, per lineal inch	600d
where d = diameter of roller in inches.	
Pins in eye-bars	22,000
Pins in riveted members	20,000
Bending:	
Pins	25,000
Steel castings	16,000
Shear:	
Shop rivets and pins	11,000
Field rivets	10,000
Webs of plate girders, gross section	10,000
Steel castings	11,000
Bearing on Masonry:	
Granite	800
Concrete	250

For nickel steel increase the unit stresses given for carbon steel by 40%.

Grip of Rivets—For rivets with a grip greater than four diameters reduce the units specified in Paragraph 20 by one per cent. (1%) for each one-sixteenth (1/16th) inch of additional grip, except in compression members having butt joints, but no rivet shall have a grip exceeding seven and one-half (7½) diameters.

Laterals and Sway Bracing—Take both systems in calculation of strains, disregarding reversal of strains.

For compression....................................16,000-70 l/r

Anchorage Masonry—Anchor piers shall show a co-efficient of safety of two for all primary stresses including impact.

Bending Stresses—All bending stresses in compression members produced by the weight of the member itself and by loads applied on the member shall be considered as primary stresses.

All such members shall be proportioned so that the greatest fibre stress due to this bending and axial strain together will not exceed the allowed units for the axial stress in that member.

Secondary Stresses—All stresses produced owing to the deformation of the steel work under any and all loads, either by the absence of pins at the joints or by the friction on pins opposing the turning of members shall be considered as secondary stresses.

Alternate Stresses—Members subject to alternate tension and compression shall be proportioned for either stresses and their section shall be made equal to the sum of the two sections. Rivets in connections and splices in all cases shall be proportioned for the sum of both stresses.

Splices in Tension Members. Tension members shall be given full splice in material and rivets.

Splices in compression Members—All splices in compression members shall be given full strength in material and a sufficient number of rivets for the axial stress.

Net Section at Pins—Pin-connected riveted tension members shall have a net section through the end pin hole at least thirty-three per cent. (33%) in excess of the net section of the body of the member and the net section back of the pin hole parallel with the axis of the member, shall not be less than eighty per cent. (80%) of the net section of the body of the member. The net section through the intermediate pin holes shall be increased over that of the member by the section cut out by the pin hole.

Latticing—The latticing of compression members to be proportioned for a cross shear per square inch of gross section of two per cent. (2%) of the unit stress for short struts of the same material; if the weight of the member produces additional shear this must also be provided for.

Single flat lattice bars shall have a thickness of at least 1-40 and double lattice bars at least 1-60 the distance between nearest end rivets. Their unit stress in compression shall be $8,600—63\frac{l}{t}$ for carbon and 40% more for nickel steel.

Lattice bars shall be so spaced that the portion of the flange included between their connections shall be as strong as the member as a whole.

Radius of Gyration of Compression Members. Minimum radius of gyration shall be one one-hundredth (1-100th) of the length of member for trusses, and one one-hundred-and-twentieth (1-120th) for lateral and sway bracing struts.

Materials to be Used—Approach spans, floorbeams, stringers, hand railings, stairways and all rivets shall be made of carbon steel. In case the main part of any member of the trusses is made of nickel steel, all the details and connections of such members shall also be nickel steel. In case the main part of any other member of the bridge is made of nickel steel, the details and connections may be made of carbon steel.

Minimum Thickness. No material shall have a thickness of less than one-half (½) inch, except floor bracing, lacing for sway and lateral bracing, stiffener angles, and sidewalk stringers which may be three-eight (⅜)inches.

In no case shall any material be less than three-eight (⅜) inches, except fillers.

Compression Members—The thickness of plates in compression members shall not be less than 1-24th of the distance between the lines of rivets connecting them to the flanges.

Forked Ends—When forked ends are used they shall be made of at least twice the sectional area of the member, and at least as strong as the body of the member.

MATERIALS

Rolled Carbon Steel

All structural steel shall be made in an open-hearth furnace.

Chemical Requirements—The ladle tests of steel as usually taken shall not contain more than the following proportions of the elements named.

	Acid.	Basic
Phosphorus	.06 per cent.	.04 per cent.
Sulphur	.04 per cent.	.04 per cent.
Manganese	.70 per cent.	.70 per cent.
	except rivet steel, .60 p.c.	
	No chromium to be used.	
Silicon	.10 per cent.	.10 per cent.

It is desired that the carbon contents be as small as possible to meet specifications.

Rivet Steel—The ladle tests of the carbon rivet steel shall not contain more than .03 of one per cent, of phosphorus, and not more than .03 of one per cent. of sulphur.

Physical Requirements—Specimens cut from the finished material shall show the following physical properties:—

MATERIAL	Ult. Strength lbs. per square inch	Minimum Yield Point lbs. per Square Inch	Minimum Elongation per cent. in 8 inches	Minimum Reduction per cent. of area
Shapes and Plates up to and including 1 in. thick...	62,000 to 70,000	35,000	$\frac{1,500,000}{\text{ultimate}}$	44 per cent.
Shapes and Plates over 1 in. thick....	62,000 to 70,000	33,000	22%; 20% for Sheared Plates	40 per cent.
Eye-bar Flats (unannealed)	66,000 to 74,000	35,000	22%	40 per cent.
Rivets........	48,000 to 56,000	28,000	$\frac{1,500,000}{\text{ultimate}}$	50 per cent.
Pins and Rollers (annealed)..	65,000 to 75,000	35,000	22% in 2″	35 per cent.

Yield to be determined by drop of the beam.

Speed of machine for testing samples to be such that material under tension will not elongate more than one inch in two minutes.

Bending Tests—Specimens cut from plates, bars and shapes two inches wide shall bend cold 180 degrees around a rod of a diameter equal to the thickness of the specimen; when at or above a red heat, 180 degrees flat.

Specimens cut from rivet rods shall bend 180 degrees flat when cold, or when at or above red heat. A test piece two inches long when heated to a bright cherry red shall flatten longitudinally under the hammer to a thickness of one-quarter (¼) inch without cracking on the edges.

Full sized sections of eye-bar material as rolled without annealing shall bend cold about a rod of diameter equal to twice the thickness of the bar. Angles of all thicknesses shall open cold to an included angle of 150° and close to an angle of 30°, without a sign of fracture.

All specimens in bending tests must show no signs of cracking on the outside of the bend.

Rolled Nickel Steel

All nickel steel shall be made in an open-hearth furnace.

Chemical Requirements—The ladle test shall contain not less than 3.25 per cent. of pure nickel, and not more than the following proportions of the elements named.

	Acid.	Basic.
Phosphorus....................	.06 per cent.	.04 per cent.
Sulphur........................	.04 per cent.	.04 per cent.
Manganese....................	.70 per cent.	.70 per cent.
	No chromium to be used.	
Silicon.........................	.10 per cent	.10 per cent.
Carbon........................	.45 per cent.	.45 per cent.

Physical Requirements—Nickel steel for plates, shapes and unannealed eye-bar flats must meet the following physical requirements in the finished material:—

MATERIAL	Ult. Strength lbs. per square inch	Minimum Yield Point lbs. per Square Inch	Minimum Elongation per cent. in 8 inches	Minimum Reduction per cent. of area
Plates and Shapes........	85,000 to 100,000	50,000	$\frac{1,600,000}{\text{ultimate}}$*	40 per cent.†
Eye-bar flats, unannealed ‡	95,000to 110,000	55,000	15%	25 per cent.
Pins (annealed)	90,000 to 105,000	55,000	$\frac{1,800,000}{\text{ultimate}}$ in 2″	35 per cent.

* For material thicker than one inch (1″), the required percentage of elongation shall be reduced by one for each increase in thickness of one quarter inch (¼″) or fraction thereof above one inch (1″), but in no case shall the minimum elongation required be less than 14%.

‡ Tests for information shall be made of annealed specimens cut from the rolled eye-bar flats.

† For material thicker than three-quarter inch (¾″), the required percentage of reduction of area shall be reduced by two for each increase in thickness of one-quarter inch (¼″) or fraction thereof above three-quarter inch (¾″).

Bending Tests—Specimens of nickel steel not less than 2″ wide and of the full thickness of the material as rolled shall bend cold 180° around rods of the diameters specified below for the various thicknesses, without fracture on the outside of the bend.

For material up to ½″ incl.	180°	around	D=1T.
" " over ½″ and up to 1½″ incl.	"	"	D=2T.
" " over 1½″	"	"	D=3T.

Angles of all thicknesses shall open cold to an included angle of 150° and close to an angle of 30°, without a sign of fracture.

Steel Castings

Steel for castings shall be made in an open-hearth furnace.

Chemical Requirements—The ladle test of steel for castings shall not contain more than the following proportions of the elements named:—

Phosphorus	.04 of one per cent.	for basic steel.
Phosphorus	.06 of one per cent.	for acid steel.
Sulphur	.05 "	"
Manganese	.75 "	"
Silicon	.35 "	"

Physical Tests—Test pieces taken from cupolas on the annealed castings shall show an ultimate strength of not less than 65,000 lbs. per square inch, an elastic limit of at least 35,000 pounds per square inch, and an elongation of not less than 20 per cent. in two inches. They shall bend without cracking 120 degrees around a rod twice the thickness of the test piece.

The final stress sheets and the working plans were necessarily made concurrently and after many trials to determine closely approximate sections and resulting weights, the deadweight of this bridge being so largely in excess of the live loads that errors in assumed weights were not permissible—particularly in the suspended span or towards the ends of the cantilevers where the deadweight has a large influence on the stresses in the cantilever and anchor arms. It is interesting to note that the shipping weights agreed within less than one percent of the estimated weights used in the final calculation of the stresses.

In making the working drawings the natural order of procedure, after the approximate designs had been prepared, was to take up first those self-contained units of the structure in which stresses did not result from the weight of other portions of the Bridge, but the deadweight of which had a marked effect upon the stresses and sections in the trusses of the structure. The description will generally follow the order of making the final design —namely, the floor system, the suspended span, the lateral systems, the cantilever structure, the shoes, and the anchorage; but the subjects must overlap in places and the logical order cannot always be strictly maintained.

Floor System:

Plate XXIV. — Shows the general arrangement of the floor system over the whole structure. It also shows a cross section of the suspended span and at two points in the cantilever structure. Cross sections showing the floor over the main and anchor piers have already been shown on Plate XX.

Plates XXV and XXVI. — Are stress and material sheets for the floor system.

Mention has already been made of the type of through floor adopted for the purpose of giving greater safety in case of derailment and for the sake of facilitating the erection.

The tracks were designed to be laid with 100 lb. A.S.C.E. rails on 8″ x 10″ wooden ties 12′ long spaced 12″ centres. Sixty-pound rails are also laid as guard rails. The ties rest on 24″ I stringers spaced 7′ centres headed into the floor beams of the through plate girder spans which are spaced about 14′ apart. The plate girder spans have lateral bracing in each panel and the webs of the stringers are diagonally braced to the girders to provide for traction and lateral stresses. The top flanges of the through girders are well stayed from the floor beams by plate brackets and are reinforced with heavy 15-inch channels for protection in case of derailment.

The sway bracing of the trusses above the floor is in the form of portal bracing with the inclined knee brace intersecting the plane of the truss well below the floor to avoid bending in the truss members. It was desirable to spread the tracks as far as practicable to reduce the moment and the deflection of the floor beams, but the spacing was limited to 32′6″ centre to centre by the necessary clearance of the portal bracing.

In spreading the tracks, consideration was given to the torque produced in the suspended span and the effect upon its sway bracing and lateral systems when diagonally opposite corners of the cantilevers were loaded by trains in positions to give the maximum difference in the deflection of the corners at the end of each cantilever. It was found that with the track spacing adopted the suspended span is sufficiently flexible to adapt itself to this twist without over strain in the cross bracing.

Plate XXVII. — Is a shop drawing of the single girder floorbeams.

The floorbeams were made as deep as practicable but owing to their great length the deflection is appreciable and special provision was necessary in the end connections to avoid bending the truss members to which they connect. With the exception of the floorbeams in the suspended span and those at panel points 1 and 2 of the cantilever and anchor arms, the floorbeams are hung on pins placed in their neutral axis, the pin holes being bushed to give a low unit pressure and to permit deflection of the floor beams without cutting the bearings.

The floorbeams at the main panel points of the cantilever structure are made of two girders, one placed on each side of the vertical tension members to which they are connected. The two girders are connected by diaphragms placed over the pins and at the centre of each track. The distribution of load upon the two girders is further provided for by connecting the abutting ends of the longitudinal track girders. All other floorbeams, including those in the suspended span, have single webs with the bottom flanges stayed to the track girders to avoid undue vibration.

Provision for Traction Stresses and Expansion Joints in the Floor System:

The fixed points of the cantilever system are the main piers and provision for changes in length arising from temperature and deformation under loads was necessary at the anchor piers and at the junction between the ends of the cantilevers and the suspended span—the possible variations, as shown on Plate XXVIII being 11 inches over the anchor piers and 14 inches at the junction of the suspended span. The floor is at varying distances from the neutral axis of the cantilever structure and when the whole structure deforms under load the floor does not exactly follow the longitudinal movements of the truss verticals to which it is connected. Moreover, being of lighter material it may be more rapidly affected by temperature changes than the heavier material of the trusses. Expansion joints were, therefore, provided at floorbeam 14 over the main piers and at the intermediate points, FB_7, in the anchor arms and FB_9 in the cantilever arms; also at the centre of the suspended span, thus dividing the floor of the cantilever structure between anchor piers into ten sections.

Where the floorbeams of the sub-panels are carried on posts, the ends of the floorbeams and the posts are stayed longitudinally by braces to the floor system, but in the more usual case where the floorbeams are carried by tension members from the " M " joints there is no provision for fixing the ends of the floorbeams or to take up longitudinal stresses at these points, and it is necessary to fix each section of floor in such a manner that the traction stresses are carried through the truss structure to the main piers. The floor sections are anchored against longitudinal movement in panels 5-6 of the suspended span, and in both cantilever and anchor arms in panel 1-2 and in the 5th panel from the main pier. On the suspended span and at the ends of the cantilever structure where the floor system is near the main lateral system, the floor is attached to the laterals of the bridge. Lattice girders, spanning the panel length in which the traction is taken, are placed on the centre line of each track with their top flanges connected to the lateral system of the track girders and their bottom flanges connected to the lateral system of the bridge. The traction stresses are thus transferred from the floor to the bottom chords of the trusses. The lattice girders have flanges of two 4 x 4 x ½ angles connected by lattice angles in the form of a Warren truss.

In panels 10-11 of the cantilever arm and 8-9 of the anchor arm, advantage was taken of the inclination of the sway bracing in the inclined compression members. Immediately below the floor girders deep lattice trusses are substituted for the regular sway bracing. The bottom flanges of the track girders are connected by brackets to these trusses and due to the inclination of the diagonal compression members, about 45°, the truss receives traction stresses from each floor girder and transmits them to the diagonal posts. The vertical component of the stress delivered to the traction truss is resisted by the track girder in bending, causing an increase or diminution of the bending moment from vertical loads depending upon the direction of traction. Provision for the additional stresses is made in the track girders and main floorbeams at these points.

Plate XXVIII. — Shows the expansion joints in detail.

Suspended Span:

Plate XXIX. — Is a stress and material diagram for the suspended span which shows the form of the cross sections of all the members.

Figure 17. — Giving a view of the span shows the general construction.

Plate XXX. — Is a shop drawing of the lower half of the end post.

Plate XXXI. — Is a shop drawing of Post U_6L_6, showing typical connections at top and bottom, the slot for the floorbeam connection and the bottom lateral connection.

The chords and end posts are constructed of three webs 45″ deep, connected on the top by cover plates 64 inches wide, also connected by diaphragms and the usual tie plates and lattice bars on the bottom flanges. The top chords were shipped and erected in half panel lengths, and spliced with material and rivets to take up the full stress after being erected. The connections at the main panel points are made on pins which also serve for the upper connection of the main web members. The end tension diagonals are eye-bars connected on pins, but all other web members consist of two built-up channels having their webs in planes parallel to the centre lines of the trusses in order that they could be connected at the ends by riveting to connection plates through which the chord pins passed.

The lower chords are nickel steel eye-bars packed parallel in one tier. The floor beams are passed through and connected to the inside channel of the vertical members and also connected to the outside channel, thus distributing the load to both sides of the verticals. The end connections are given a bevel equivalent to the deflection under one-half live load, and

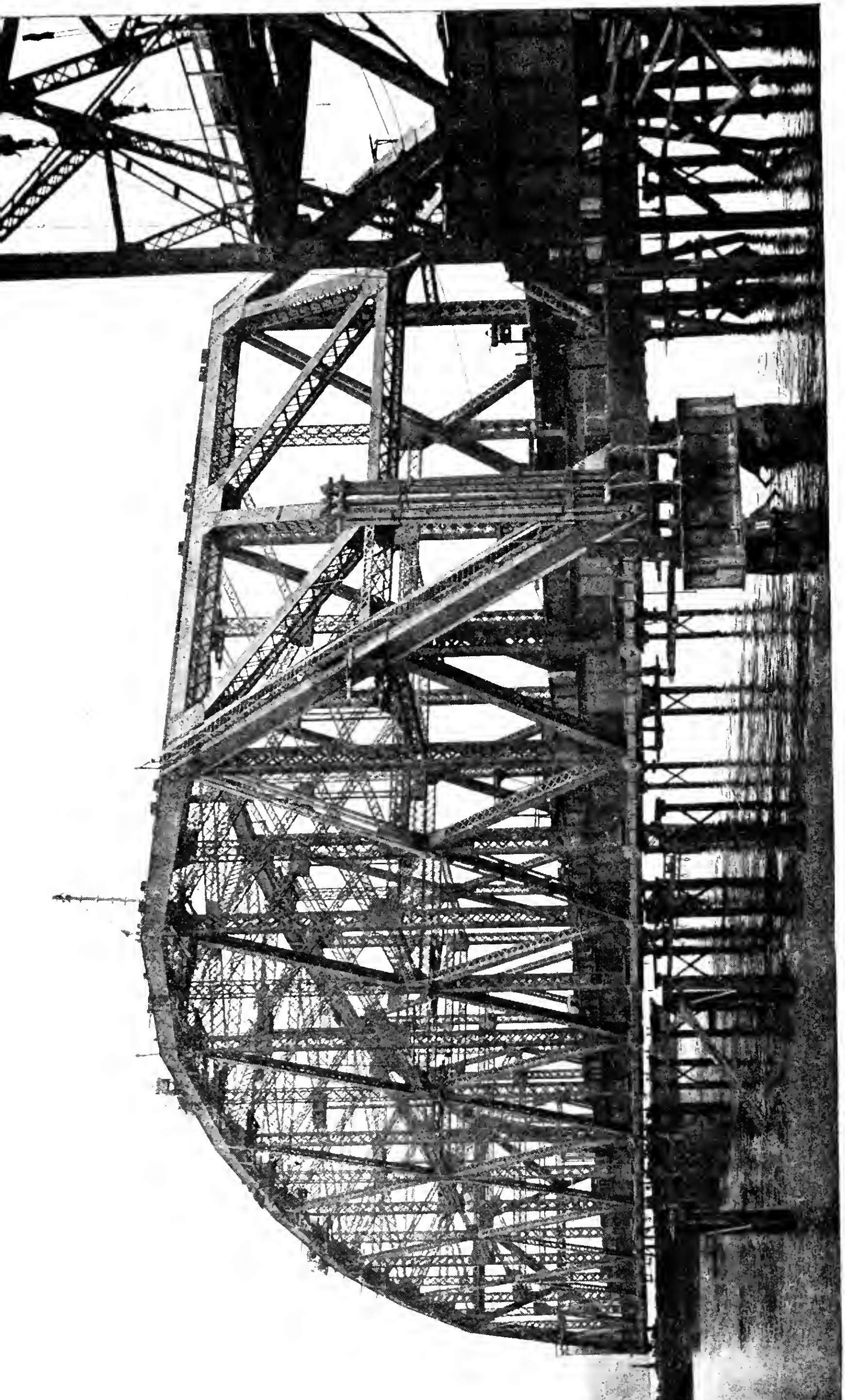


Figure 17 Photograph of Suspended Span at Sillery.

the verticals have a slight initial bend under dead load only but are straight and without bending stresses when the floor beam is under usual conditions of live load.

The lateral bracing is of box section with the top and bottom flanges connected to the floor beams and the verticals at equal distances from the centre line of the bottom chord, thus eliminating bending stresses in the truss members.

The suspended span is carried by eye-bars pin-connected at the intersection of the lower chords and the end post of the truss as shown on Fig. 17, but during erection it was carried by a bearing under the centre rib. The transfer of stresses from the outer ribs to the centre rib was accomplished by inserting heavy diaphragms and reinforcing the tie plates adjacent to the shoe.

The transverse horizontal loads acting on the suspended span are transferred to the lateral system of the cantilever by a heavy pin illustrated on Plate XXVIII. This pin is cantilevered out from a box girder forming an end strut for the suspended span and slides in a bronze bearing, spherical on the outer surface, so that it may be free to align itself with the pin under all conditions of vertical or horizontal flexure.

The eye-bars suspending the span from the end of the cantilever were made but 10″ wide to minimize the secondary stresses from bending, due to the expansion and contraction of the structure. Under certain combinations of loading and temperature—the deformations may cause the point L_0 of the suspended span to move 9″ towards the centre of the bridge or 5″ away from it in the other direction, giving a total range of 14″.

Experiments were made, as referred to in Appendix A to determine the resistance of pin friction (steel on steel dry) to angular movement, and the bending stresses caused by friction of 40% were provided for. The pin holes in the eye-bars are bushed with bronze to reduce the unit pressure and prevent cutting and it is probable that the actual bending stresses are very much less than those estimated.

Cantilever Structure:

Plates XXXII and XXXIII. — Show the axial stresses and the material of the cantilever and anchor arms.

Plates XXXIV and CXIII. — Show the secondary stresses throughout the cantilever and Anchor arms. These did not affect the sections of the members but required some extension of the pin plates at CL_{16} and AL_{14} and special treatment of the floorbeam connections at CL_2 AL_2, as indicated on the plates.

Plates XXXV and CXV. — Show the stresses and material in the sway bracing of the cantilever and Anchor arms. There is no top lateral bracing in the cantilever or anchor arms, the horizontal forces applied at the top chord being transferred through the sway bracing between the vertical posts to the " M " joints, from which points they are carried to the bottom lateral system through sway bracing in the plane of the inclined compression members. The sway bracing between the vertical members is of the ordinary type consisting of deep top and bottom struts connected by a single set of diagonals in each direction. The bottom strut is stayed by a vertical from its centre to the intersection point of the diagonals.

The wind shears at the " M " joints are carried by portal bracing to a point below the floorbeams where the diagonal bracing is resumed. as shown on the stress sheet. The knee braces of the portal are extended to an intersection with the diagonals below the floor system to avoid bending stresses in the compression members in the truss. Sway bracing of the " K " form is placed between the tension verticals of the truss below the floor to carry the transverse loads acting on the floor to the bottom lateral system.

The omission of the top laterals eliminates ambiguity and makes the wind stresses for the assumed pressure determinate throughout the structure. While the vertical forces due to wind displacement considerably increase the stresses in the truss members, the arrangement worked out more economically than if the top chord wind loads had been carried to the centre and from thence transferred through the sway bracing to the main piers. It, moreover, simplified the erection as the cantilever arm was stayed against wind forces by the permanent bracing—without the use of temporary material—at all stages of its erection.

The laterals were calculated as a double Warren system, but horizontal struts were introduced to assist in keeping the alignment during erection and to complete the " K " system of sway bracing in the vertical tension members. The lateral members and struts are all formed of double lattice girders the depth of the chord, latticed together on top and bottom flanges. The laterals are connected to the chords by heavy connection plates reinforced where necessary to transmit the longitudinal stresses from the connection to the centre of the truss. The pin plates connecting the truss members prevented the upper lateral connection plates from being carried across the upper flanges of the chords and the forces are distributed over the four ribs of the chord by wide tie plates at the ends of the lateral connections on both top and bottom flanges of the chords.

Plate VII.—Shows the cross section and outside dimensions of all members in the cantilever arm.

The main truss members of the cantilever structure are made of four ribs consisting of symmetrical I sections for compression members, and channel sections for tension members, built up of plates and angles. The compression ribs are connected in pairs by longitudinal diaphragms on the centre lines and lattice and tie plates connect the flanges thus forming two H sections, which were again connected in the field by tie plates on the flanges of all members and by an additional line of tie plates on the centre line of the larger members.

The tension webs were assembled in pairs with flanges turned in and connected by lattice and tie plates on the flanges only.

Each member was completely assembled in the shop for the purpose of finishing the ends, boring pin holes and drilling splicing material. Most of the pins connecting the web system were in two lengths. This construction practically divided each main truss into two complete trusses placed side by side and field connected together, which facilitated the transportation and erection of the large members. The butt joints of the compression members are in perfect contact throughout the structure, but when the work was designed it was thought impracticable to attain this accuracy and all compression joints are fully spliced by material and rivets, no reliance being placed on the bearing of the faced ends.

A description of the details would be too voluminous but a few typical drawings follow which show the general arrangement of the details and the assembly at the joints of the main members of the structure.

Plate XXXVI. — Shows the size of lattice bars for all members of the cantilever arm, and plate XXXVII the method of their determination.

The general make-up of the members is illustrated by the typical shop drawings shown on plates.

Plate XXXVIII. — Shows the detail drawing of the bottom chord at AL_{12}.

Plate XXXIX. — Shows the upper half of a diagonal compression member with the detail at AM_{10}.

Plate XL. — Is a vertical compression member which rests on the diagonal shown on Plate XXXIX.

Plate XLI. — Shows a typical joint of the bottom chord with the truss members and lateral bracing assembled. It will be noted that all the connections of the web members to the bottom chord were made on pins outside the chord permitting the webs of the chord and the compression web members to be in practically the same vertical planes which are parallel throughout. In elevation the chords are tapered from a depth of 82″ at the shoe to a depth of 45″ at the L_2 joint.

Plate XLII. — Shows the detail at Joint AL_2.

Plate XLIII. — Shows the assembled members at AM_{12}. It will be noted that the detail is built as an integral part of the compression diagonal $AM_{12}M_{13}$. This arrangement enabled the

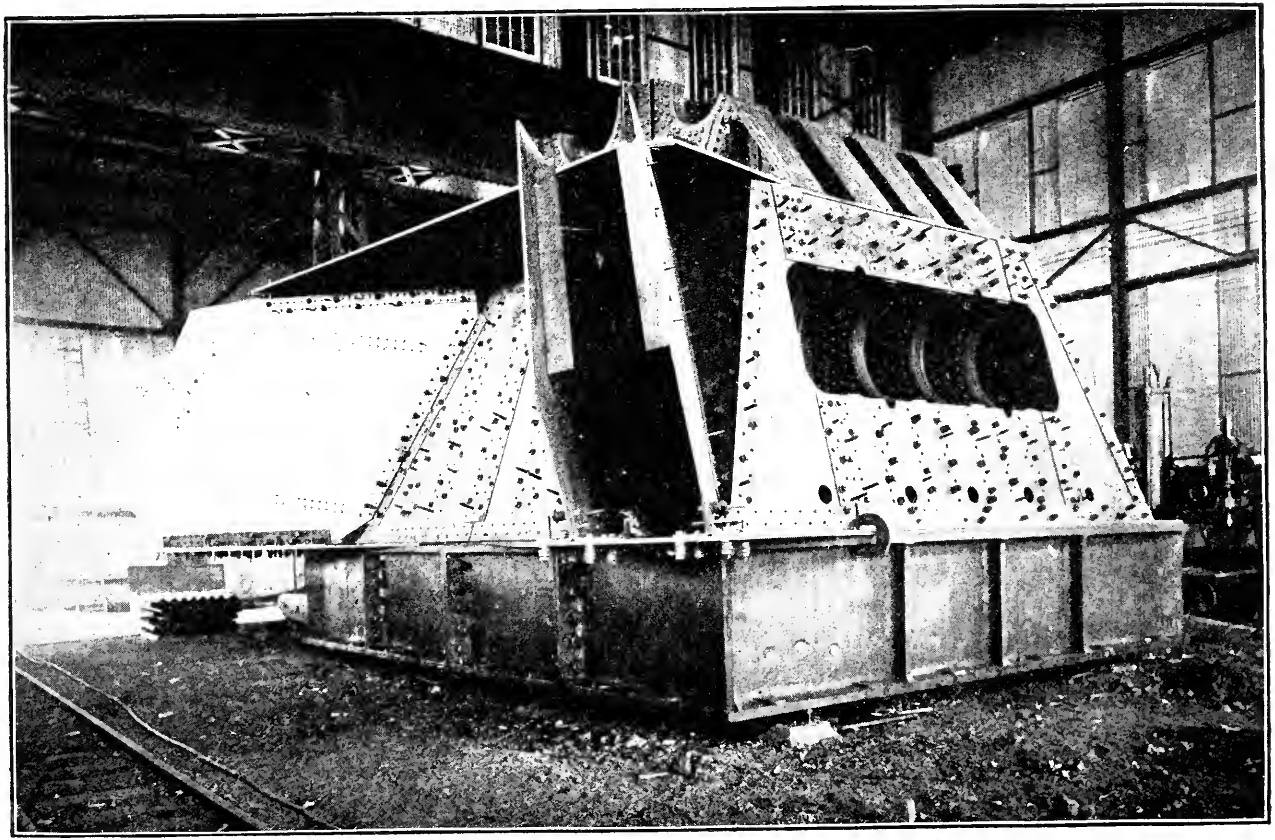


Figure 18 **Photograph of shoe assembled in the shop.**

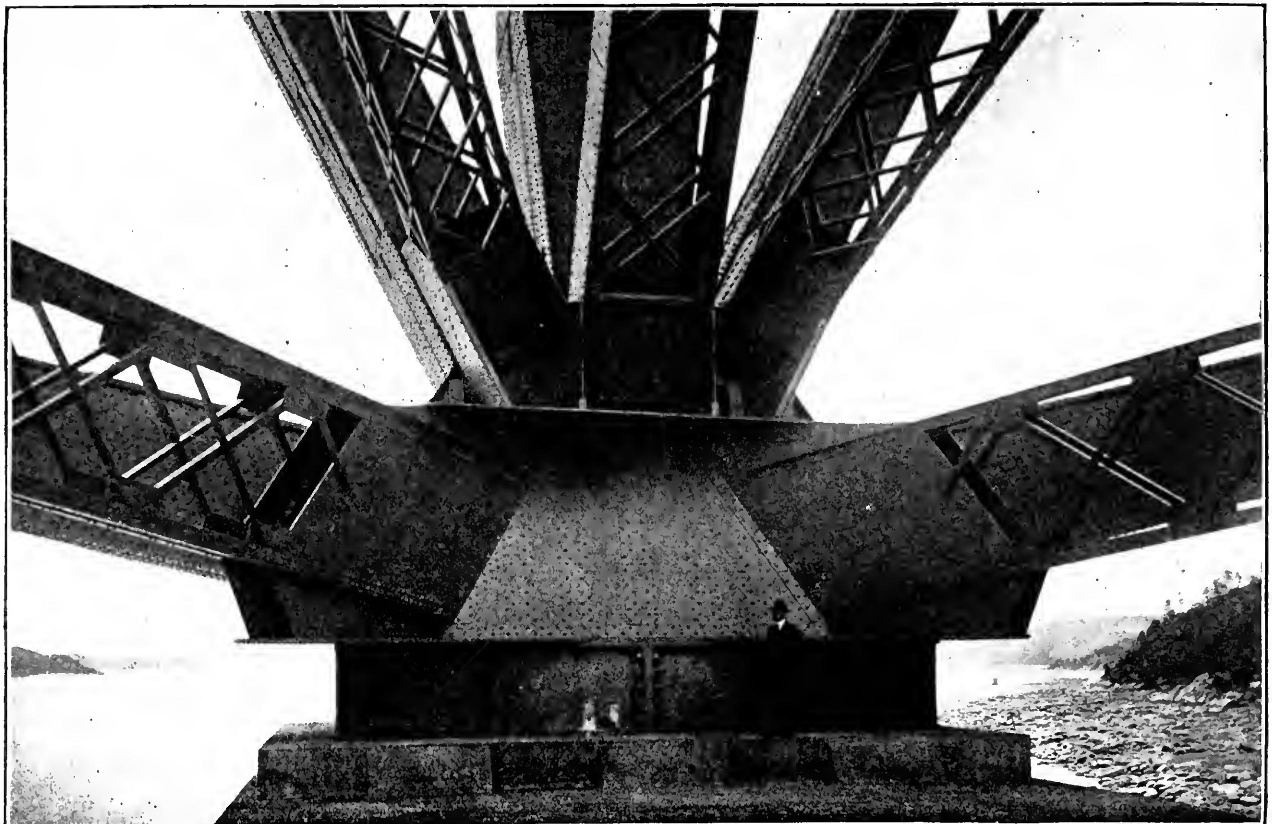


Figure 19 **Photograph of shoe assembled in the structure.**

material of the compression member to be carried through the joint and gave an independent connection for all the members assembling at this point. The combination of these heavy connection plates is quite complicated and cardboard models were made of all joints to facilitate the consideration of the distribution of stress amongst them.

Plate XLIV. — Shows a typical top chord panel. Eye-bars could not be obtained in full panel lengths at reasonable cost and difficulty was foreseen in handling such long bars. There was doubt as to the proper allowance for bending stresses caused by the weight of the bars. The bars were, therefore, made in two lengths connected by pins in the centre of the panels. They were packed parallel to the plane of the trusses in two tiers, to insure an equal distribution of stress throughout all the bars by reducing the length of the pins. To prevent deflection of the bars they were joined at the center. The connecting pins were carried by trusses spanning the length of a panel. The trusses with their lateral bracing formed a box frame in which the bars were packed before erection. This arrangement greatly simplified the erection and held the bars in place until the pins could be driven. At the top of the main post the bars were connected by a special detail the shop drawing of which is shown on Plate XLV.

Plate CXVII. — Shows the main posts and the bracing between them.

Plate XLVI. — Shows the joint AU_{14}, the top of the main post.

Plate XLVII. — Shows the detail at joint AM_{14} about the middle of the main post and show the adjustment provided in the horizontal member to insure the proper distribution of stress and provide a means of tipping the main post out of the vertical position in order to be able to connect the members in the anchor arm.

The form of the trusses, the arrangement of the joints and the methods of loading of the trusses assured the calculated distribution of stress throughout the structure. At the end of the cantilever the weight and loads carried by the suspended span are equally distributed over the four webs of each cantilever truss.

Plate XLVIII. — Shows the joint at CU_0 the end of the cantilever arm.

Deformation diagrams were studied concurrently with the preparation of the stress sheets, the design of the details and the erection, but the only special provision necessary for deformation or for secondary strains beyond the ordinary framing for camber and the elongated holes in the tension members of the anchor arm was in the suspension bars of the central span and in the pin joints at the main shoes.

Plate XLIX. — Shows the anchor arm deformation diagram. For convenience in making connections the anchor arm was erected with the point L_2 5 inches below its normal position. This and the framing lengths required that the vertical post over the main piers should be set with its top 14 inches shorewards, causing angular displacement in all members connecting at the shoe. As the cantilever was erected the post gradually approached a vertical position.

To provide for this angular displacement without causing bending stresses in the members connecting at the shoe and to eliminate the danger of pin cutting, the bearing unit pressure on the pins was very much reduced by the use of sleeves and as a further precaution the moving surfaces were lubricated with paraffin. By introducing sleeves between the pins and the bearing surfaces of the members to give a bearing over the whole length of the pins, the unit pressure was reduced to about 8600 lbs. per sq. in. under total load and 4000 lbs. per sq. in. for dead load and the thickness of the members bearing on the larger diameter of the sleeve is kept within reasonable limits, about 9 inches. The experiments noted in Appendix A show that under the above pressure and with paraffin lubrication the friction is reduced to 2%, but to be entirely on the safe side it was calculated at 40%.

Deformation Diagrams are shown on Figs. 5 and 8; also on Plate L. These show remarkable uniformity of curves and the absence of kinks in the members at all stages of the work.

Plate LI.—Shows the general construction of the main shoe.

Figure 18.—Shows the shoe being assembled in the shop.

Figure 19.—Is a view of the completed shoe in the structure showing the connection of the lateral bracing.

The maximum stresses calculated in the various members connecting at the shoe are as follows:—

Main Vertical Post, 26,600,000 lbs.; Cantilever Compression Diagonal, 7,820,000 lbs.; Anchor Arm Compression Diagonal, 7,810,000 lbs.; Cantilever Chord, 29,600,000 lbs.; Anchor Arm Chord, 24,100,000 lbs.

The vertical components of the above stresses are 55,000,000 lbs. and the horizontal components at the centre of the shoe 32,000,000 lbs.

The shoe proper is built up of four webs in the same plane as the webs of the connecting truss members. The vertical loads from the webs are distributed on the masonry by a grillage of cast steel box girders each about 6'8" wide and 20'10" long, giving a bearing area on the masonry of 26' 4" x x 20' 10". Each grillage casting has 4 longitudinal ribs and 8 cross diaphragms one under each of the main ribs of the shoe. The material is no where less than 2½" thick, increased as necessary to carry the concentrated loads under the ribs. The rough castings as they came from the foundry weighed about 40 tons and the shipping weight, after machining, of each casting averaged about 37 tons.

For convenience in obtaining material, handling in the shop and shipping, the main portion of the shoe was made in two storeys the lower one 9′ 6″ high and the upper one 5′ 5″, making the centre of the pin in the main post 15 feet from the base of the shoe and 19 feet from the masonry. The ribs of the lower portion were shipped in pairs and connected in the field, but the upper portion was shop riveted together and shipped as a unit.

Plate LI. — Shows the diaphragms between the ribs and the brackets opposite these diaphragms extending from the top of the shoe to the outer edge of the base plate resting on the grillage castings. The brackets were added to give lateral stability to the shoe and to carry the transverse wind force amounting to 1,300,000 lbs. to the base. While the grillage castings were calculated to distribute the vertical loads of the ribs over the full area of the base, the stiff brackets must carry considerable loads and these were made as rigid as practicable, special 12 x 12 bent plate angles being used for connecting them to the ribs and all faces being planed to give a perfectly true bearing. Horizontal plates were introduced between the two parts and at the bottom of the shoe to make convenient connections between the parts and to distribute the lateral stresses throughout the member. The ends of the shoe ribs and of the brackets are covered by face plates to add to the rigidity, make a better appearance and keep out the weather. Manholes were provided to all parts of the shoe to make it accessible for inspection. After erection the grillage castings were filled solid with concrete and the base of the shoe was also filled with concrete up to the bottom of manholes in transverse diaphragms and sloped to the drainage holes shown on the end elevation.

The grillage castings were machined with great care to exact dimensions on all faces, assembled on a carefully levelled table and the holes drilled for the bolts through the end flanges connecting the sections. Each rib of the lower story of the shoe was fabricated separately but the holes connecting it to the grillage casting and for the diaphragms and brackets were drilled after the ribs were assembled on the grillage casting. The connecting rivet holes in the upper portion of the shoe were also drilled after assembly. After assembly and match marking on the grillages, the shoes above the grillages were re-assembled on the bed of a large boring machine and all pin holes were finished at one setting of the shoe.

The pressure on the granite masonry from the vertical loads is 690 lbs. per sq. in. which is only slightly increased by the displacement due to transverse wind force, but the longitudinal wind force is much larger, being 6,200,000 lbs., and is transmitted to the masonry through the comparatively narrow grillage girders. After making due allowance for these girders being riveted to the base of the shoe and for friction, it is calculated that the toe pressure under the leeward edge of the grillage girders may reach 915 lbs. per sq. in. for all loads.

Friction brakes were placed at the ends of the suspended span in the sections of bottom chord between L_1 of the cantilever and L_0 of the suspended span to prevent any sudden movement of the span due to traction or from longitudinal wind force. The detail of these brakes is shown in plate LII. They are calculated for a resistance of 250,000 lbs. and are made adjustable so that this resistance may be maintained.

There is no condition of loading which produces a positive reaction at the anchor piers and the anchorage bars are always in tension. The transverse horizontal forces of the lateral system are transferred to the anchor system through a pin cantilevered from a heavy steel frame built into the masonry shown on Plate LIII.

Plate XXVIII. — Shows the detail of the sliding shoe engaging the top of this pin and the frame in which it slides.

Plate LIII. — Shows the general arrangement of the grillage girders in the anchor pier and the anchor bars at the end of the anchor arm. Steel stairways for inspection purposes are provided over the main piers and at the ends of the anchor and cantilever arms. The tops of the girders carrying the upper chord eyebars provide a footwalk over the cantilevers which is guarded by a pipe handrail on each side.

Chapter IV

DESCRIPTION OF THE SHOPS AND EQUIPMENT WITH REFERENCE TO THE USUAL METHODS OF MANUFACTURE

At the time the construction estimates were prepared there was no Canadian shop equipped with cranes and tools of sufficient capacity to manufacture the large members called for by the design; the shops of both the parent Companies were fully occupied with ordinary construction, making it undesirable to re-model either plant for manufacturing the Bridge and it was decided that special shops must be built for the work.

There was no crossing of the River below Montreal, and the high banks at the site, great range of tide, exposure to waves and ice conditions in the Winter made it impracticable to deliver by water. The site selected for he shops is at Lachine, near Montreal, on the main line of the Grand Trunk Railway, giving connection by the Grand Trunk Railway and the Intercolonial Railway with the South side of the River at the bridge site and a short spur connected the shop site with the Canadian Pacific Railway over which material could be shipped to the North side of the River, both connections providing excellent railway facilities both for receiving and shipping material.

Plate LIV. — Shows the general arrangement of tracks and connections to the trunk lines, the location of all buildings and the storage yards.

Plate LV. — Shows in fuller detail the plan of the principal shops, the position of the machine tools, the track arrangement of the interior of the shops and of the storage yards. It also shows the three elevations of the main shop.

Plate LVI. — Shows a transverse section through each end of the shop and the longitudinal section through the centre. The overhead crane arrangements for handling material are indicated in these sections.

The main portion of the shop was 160 feet wide and 660 feet long, with an extension 60 ft. wide along the South side for a distance of 440 feet from the East end and 30 feet wide for the remaining 220 feet. 400 feet of the West or receiving end of the main shop was divided into four bays of 100 feet each between columns. The 260 ft. of the East or finishing end was made higher with a row of columns 20ft. apart, practically dividing the shop into two aisles of 75 and 85 feet, each covered by travelling cranes. 100 feet of the East end of the 60 ft. extension was utilized as part of the main shop for boring eye-bars. The remainder of the 60 ft.

extension contained the pipe shop, forge shop, machine shop, tool room, power plant, heating plant and electric supplies. The straightening rolls occupied the 30 ft. extension at the West end.

In planning the construction of the shop it was decided to eliminate the risk of fire and possible consequent delay by making the shop entirely fire proof, and no wood whatever entered into its construction. The roof is of 3-inch reinforced concrete slab, resting on steel beams; metal sash is used for all windows and sky-lights; the walls are of brick below the windows and cement plaster on expanded metal above.

The template shop, shown on plate LV, is a steel frame building 60 x 176 ft. in plan. The curtain walls are of brick but the roof is of wood laid on steel frames, as it was considered useless to attempt to make this building fire proof, its contents being of so inflammable a nature and in such quantity that a fire in the contents might destroy a fire proof building. A sprinkler system was installed to make the fire risk to building and contents as small as possible.

The main shop was laid out with a view to conducting all longitudinal movement of material on narrow gauge tracks which extend to the storage yard at the West end. Transverse movement at the receiving end of the shop where the operations of straightening, shearing, drilling, edge planing, punching were carried on, was accomplished by means of transverse overhead cranes. After the material was fabricated it was moved East on the narrow guage tracks and assembled under heavy longitudinal cranes which were of sufficient capacity to lift the heaviest members after assembling. The South bay where the large wall planers and boring machine were placed was equipped with two 70-ton overhead cranes and one 35-ton, and all the large compression members were assembled, riveted and finished on this side of the shop. The North bay was equipped with two 35-ton cranes and was used for assembling the long tension members, the lighter compression members, floorbeams and the smaller material that could be handled by the lighter cranes. The shipping shed was equipped with one 70-ton two trolley crane and one 30-ton crane. In addition to being used for loading material, it was also used for assembling large members and drilling the splice connections. A heavy 10′ x 10′ Loudon Bros. planer was installed at the North end for planing the 40-ton grillage castings, the shoe ribs and other work of a similar character. This planer was later fitted with a boring bar to supplement the Newton borers in the main shop.

The site chosen for the shop was under-laid by rock at a depth of about four feet, giving excellent foundations for the large machines and the skidways to carry the heavy members.

The shops were laid out after careful consideration of the length of members to be handled, their weight, the operations to be performed and the quantity to be manufactured each month.

The Plate Straightening Rolls were designed to straighten plates 120 inches wide and were of sufficient strength to straighten nickel steel plates 84″ wide and 1¼″ thick.

Three edge planers were installed, two with a capacity of planing plates 2″ thick and 45′ long, taking a 5/32″ cut at a speed of 40 ft. per minute and one with a capacity of planing plates 20 ft. long. The Plate shear had a capacity of shearing nickel steel plates 84″ wide and 1¼″ thick. The Angle Shears had a capacity to cut nickel steel angles 8 x 8 x 1¼″ at an angle of 45°.

Most of the rivet holes were drilled in the solid and for this purpose 40 radial drills were installed. All drills had a 6′ reach. The columns of 14 drills were placed along the centre of the second 100 ft. bay and were anchored to the masonry. The remaining 26 drills were mounted on heavy trucks which ran on special tracks to which the trucks could be anchored after the drill was set in the desired position.

Punches, spacing tables, angle mills, lattice mills, and other machinery of a similar kind were standard for heavy bridge work.

Compressed Air at 100 lbs. was used for operating the riveting machines all of which were of the Hanna or Murphy toggle type of yoke riveter and maintained a uniform pressure on the rivet for the last ¾″ of the stroke. There was considerable variety in the capacity and reach of the machines used. For the long 1⅛″ rivets the machines were of 100 tons capacity some with a reach of 6′3″, while for the short ⅞″ rivets machines of 70 tons capacity some having only 48″ reach were used. The heavy riveters were mounted on wall cranes which travelled along a track on each side of the central columns at the riveting bays of the shop.

Three 60″ rotary planers were installed all on turntables, two of them on the south side of the shop being mounted on a heavy bed permitting both ends of a member 90 ft. long to be faced at the same time. One was mounted at the northeast corner of the shop and equipped with a squaring table, for finishing the ends of the smaller members.

For facing the ends of the heavy compression members a double headed vertical and horizontal planer was installed at the east end of the shop mounted on a bed permitting both ends of the heavy compression members to be finished at the same time. These machines had a vertical and horizontal stroke of 126″ and could be set to face both ends of a member 65′ long. Care was taken to maintain a nearly uniform temperature while a member was being planed and the operation of facing the individual member continued day and night until finished.

Compression members of ordinary size and tension members were finished with the double headed rotary and single rotary as before mentioned. For boring the very large pinholes in the shoe and heavy compression members a special boring machine was installed at the south-east corner of the main shop. The boring bar of this machine had a sufficient range of motion to bore all the holes in any piece without resetting the work after the piece was bolted to the bed, the boring bar having a horizontal movement of 22′ and a maximum height from the top of the bed to the centre of the bar of 15′. The cutting head travelled through a distance of 14′. The large pinholes in the shoes and bottom chord 45″ in diameter were bored by this machine.

In the north bay of the Shop two Newton horizontal borers were installed for boring the holes in the tension members and those in the lighter compression members. These machines were used on holes up to 14″ in diameter and 90′ centres.

A double-headed eyebar boring machine was installed in the east end of the 60 ft. lean-to, each head set on a heavy bed of sufficient length to bore holes 70 ft. apart centre to centre. The large pin holes in the chords and connecting plates were swept out 1″ smaller than the finished hole in the individual plates before assembling. For this purpose there was provided two manhole borers each having a capacity to cut holes from 10″ to 45″ in diameter and equipped to cut elliptical manholes.

The best quality of workmanship was demanded. All sheared edges were planed; punching was not allowed in nickel steel and only in carbon steel not over 11/16th in. thick. All drilled holes were drilled after assembling the members and punched holes were reamed ¼ of an inch larger than the diameter of the punch after assembling. Rivet holes were countersunk about 1/16 of an inch to remove the burrs.

Some difficulty was found in drilling the members after the material was assembled together, in a manner that would avoid drill chips or an excess of compound working in between the several parts. This was successfully overcome in the following manner: in the thick material, of which most of the members was composed, stitch holes for bolting were selected which would give an area of unfastened plate not over about 14 inches square. These holes were drilled ⅛ of an inch smaller than the finished hole and after the other holes were drilled and bolted they were reamed to the proper size. It was found impracticable to draw the plates close enough together by ordinary bolting and after the pieces were assembled the member was moved down to the riveting trestles where a pressure of about 80 tons was applied through a riveter alongside of each bolt as it was being tightened up. The ordinary carbon steel assembling bolts did not stand the stress of being properly set up and Mayari steel was early substituted for this purpose.

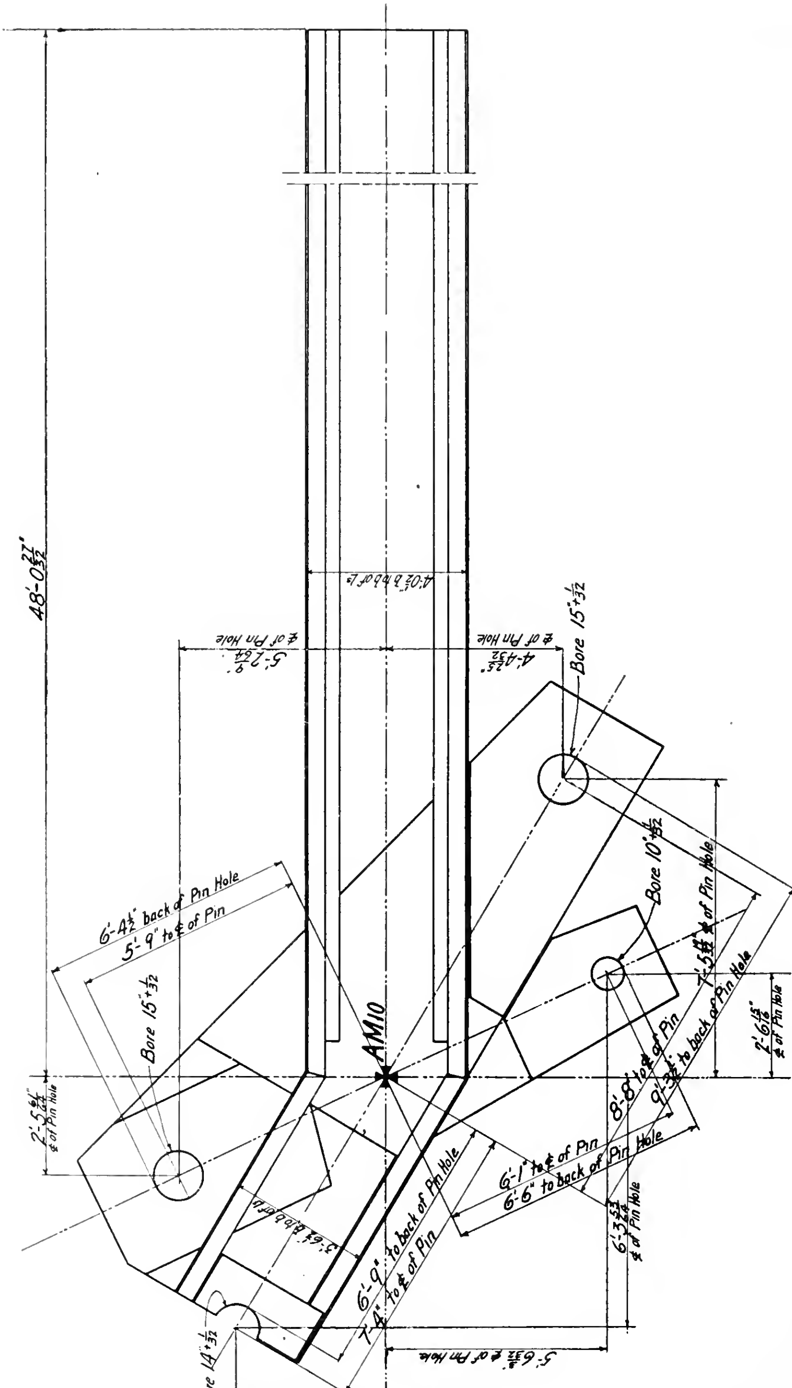

AM10
Bore 15" + 1/32
6'-4 1/2" back of Pin Hole
5'-9" to ℄ of Pin
Bore 10" + 1/32
6'-1" to ℄ of Pin
6'-6" to back of Pin Hole
6'-9" to back of Pin Hole
7'-4" to ℄ of Pin
Bore 14" + 1/32

Figure 20

All lattice bars were drilled and planed to jig. In the early part of the work the holes were reamed after assembling, but it was found that the heavy members were kept better in line by accurately drilling the member and the lattice bars separately and this practice was generally followed throughout the work. The ends of all built webs of both compression and tension members were planed on the rotary planers after riveting but before assembling into the member, principally to relieve the large planers used for finishing the assembled member.

The Contractors appreciated from the outset that very perfect workmanship would be necessary, it being of the utmost importance to the operations in the field with work of this magnitude to have all members finished so exactly that connections could be made without difficulty.

Extreme care was exercised in setting members for machining to keep the member absolutely level both longitudinally and transversely. A centre line was marked on each outside web and an Engineer's level was set up off one corner of the member so that a sight could be taken along the longitudinal centre line and across the end. When that end was levelled exactly, the level was moved to the other diagonal corner to see that no wind existed. After the member was exactly level it was securely bolted to the machine bed. The transverse level was further checked by running a tool across the member at each end before starting to machine it.

The truss web members are all connected by pins with the exception of the diagonals in the suspended span, but the lateral and sway bracing was made with riveted connections throughout. All field riveted connections were reamed to steel templates, heavily bushed to prevent wear. These templates as well as those for the pin holes were checked by pinning the templates together on the floor of the template shop and great care was taken in applying them to the members.

The pin holes in the tension members were on the centre line of the member and required only the ordinary precautions in laying them off and boring them, but the pin holes at the " M " joints of the diagonal compression members and those of the bottom chords are in pin plates outside the members and great care was necessary in laying out these holes on the finished member.

Figure 20. — Shows a typical "M" joint and the measurements used for laying out these holes on the finished member. Before assembling the webs of the member together the holes were marked off in the connection plates from templates and swept out in a manhole borer about 1½ inches smaller in diameter than the finished hole. In laying out the finished boring; after the member was truly levelled on the bed, the intersection point was measured from the faced end previously finished and the centre of each pin hole was then laid out by measuring the distance from the intersection point to the vertical passing through the hole and laying off the

ordinate. After the pin holes were scribed, careful check measurements were made by measuring the distance from the back of each pin hole to the intersection point — the back of the pin hole always being taken as that side on which the pin bore. That is to say, in compression members the check distance was from the intersection point to the inner side of the pin hole, while in tension members it was to the far side of the pin hole. All of the distances and angles for laying off the pin holes were calculated in the drawing office and shown on the drawings.

The compression joints were spliced with material and rivets sufficient to carry the full calculated stress. The holes for the connecting rivets were drilled after the joint was assembled in the shop and the parts drawn into close contact.

All measurements in shop and field were made with steel tapes which were frequently checked with the standard tape, kept for that purpose, and corrected if necessary.

Chapter V

FIELD OPERATIONS AND ERECTION

Transportation by Railway limited the size of some details, such as the end connection plates, shoe ribs, etc. The maximum weights and in many instances the maximum lengths that could be safely transported determined the limit of shop assembly. Sketches were prepared and submitted to the Railways which showed the weights of the heavier members, their dimensions including connection plates and all data necessary for the design of special rolling stock necessary to carry the exceptional members. The railways provided steel cars of 150,000 lbs. capacity with specially framed bodies to provide for the concentrated loadings and with openings in the floor to pass the projecting connection plates on the members.

Plate LVII. — Show typical loading diagrams.

For convenience in handling the large members in the shop, in loading, unloading and in erection, the centre of gravity of each member was marked in the shop. In many cases special connections for the hoists were bolted on and only removed after the member was in place in the bridge. These connections were of various forms to suit the details of the member to be handled; some of them are shown on Plate LVIII. The positions for the connections were determined in the Drawing Office by calculation and verified in the shop.

(a) *Camp and storage at site.*

Crane runways were erected at the bridge site storage yards, each having two trolley cranes of 70 tons capacity, 30 ft. lift and 83 ft. span. The distance between columns in the runways was fifty (50) feet and the total length of each runway 500 feet. The columns were staggered and members longer than the crane span were handled through the runway by moving the member transversely when passing a column.

A convenient site for the North storage yard was found within the " Y " tracks close to the North end of the bridge, the yard thus connecting directly to the bridge and to Sillery Point where the suspended span was erected. At the yard site there were also a number of Railway sidings which were convenient for the storage of the lighter material that could be handled by locomotive cranes. On the South side the nearest point available for the crane runway was about a mile from the end of the bridge. The location of the storage yards is shown on Plate XIX.

The equipment for transferring material from the storage yards to the erectors consisted of:

Two 4-wheel yard locomotives weighing 56,000 lbs. each;

Four Bay City Locomotive Cranes each with a capacity of 30 tons at 13 feet radius;

Eight steel frame flat cars each of 80,000 lbs. capacity;

Ten trucks for transporting the heavy members to the erection traveller.

At the North yard, well equipped repair shops were established for blacksmiths, machinists, pipe men and electricians; also store houses for small material, erection gear and paint.

Bins for rivets and bolts, storage tanks for fuel oil required for heating rivets as well as facilities for watering and fuelling locomotives were established on both sides.

An office was placed near the South end of the bridge with accommodation for the South Shore force.

The working forces and all buildings were connected by telephone to the Main Office.

With the exception of the steam railway equipment, all of the power used in the erection of the bridge was electric. A.C. current was purchased at line voltage of 22,000 on the North side and 11,000 volts on the South side, and transformed to 2,200 volts for the synchronous motors driving the air compressors and motor generator sets which transformed to D.C. current for use in the traveller motors.

Steel frame fire-proof power houses were built on both sides of the River. That on the North side contained four electrically driven air compressors each of 530 cu. ft. capacity, and two motor generator sets each of 250 K.W. capacity. The South side power house contained three 530 cu. ft. air compressors and two motor generator sets.

The offices and dwellings for the Construction Engineer, the Superintendent of Erection and their staffs were established at the North end of the bridge, as well as a substantially built camp with sleeping accommodation for over two hundred men and boarding facilities for a much larger number. The camp included a hospital, police station and all the accessories of an isolated camp — it being about half a mile from the main road and three and one-half miles from the electric car line leading to Quebec. It was provided with water and sewerage systems, cold storage plant, electric light and fire protection. A road was graded to connect it with the main Quebec Road.

Plate LIX. — Shows the general layout of the camp in its relation to the North end of the bridge. The five bunk houses were each 24 x 53 ft. in plan; the dining hall 47 x 58 ft; the office building a two storey structure, 28 x 53 ft. with a one storey wing 20 x 20.

Two gasoline motor boats were employed for ferrying men and light materials to and from the South side. As soon as the work permitted, electric passenger elevators were installed on each side of the River connecting the shore level with the floor of the bridge.

(b) *Erection Travellers:*

All of the material of the bridge proper between anchor piers, including the steel falsework, was erected by means of two inside tower travellers, one on each side of the River, having the main hoists suspended from trollies which in turn were mounted on electric travelling cranes, enabling the hoist to be placed any where within the range of these cranes.

The general design and principal dimensions of the travellers are shown on the following drawings:

Plate	XIII	Side elevation and top plan
"	XIV	Front elevation
"	LX	Stresses and material
"	LXI	Deformation Diagram

The considerations leading to the adoption of this new type of traveller were fully discussed in the introductory paper, Pages 29, 30, 41 and 42. The original design was somewhat enlarged in scope, it being found desirable to add to the tender design the four swinging booms for handling the sway bracing, falsework and other light material; also the 5-ton auxiliary hoists at the ends of the travelling cranes for handling the working cages. These additions, with the machinery, added so considerably to the weight of the traveller as first designed that nickel steel was adopted wherever an appreciable saving in weight could be made by its use as in the top cantilever trusses and the structural material of the travelling cranes.

The Hoisting equipment of each traveller consisted of four 60-ton hoists hung from trollies, two on each side. Four pairs of 5-ton hoists also hung from trollies on the ends of the travelling cranes and four 20-ton hoists on the ends of the 70-ft. derrick booms.

The weight of each traveller in working order was about 920 tons approximately made up as follows:

Runway Trucks	65 tons
Bottom supporting trusses and bracing	120 "
Main tower frame floors and bracing	170 "
Wood in floors, etc	30 "
Top crane runway trusses and bracing	165 "
Elevator Shaft and stairway	19 "
Main travelling cranes	210 "
Auxiliary Gantry Cranes	36 "
Derrick Booms and Tackle	50 "
Wire and manilla rope except for main cranes	8 "
Electric wiring	3 "
Derrick hoisting engines	32 "
Miscellanous machinery on operating floor 3	12 "
	920 tons

The bottom story of the tower was made 89 feet long to give a long base to reduce the reactions and lighten the rear anchorage against uplift, but the length of the tower proper above the lower sway struts was limited to 37 feet to permit the sway bracing to be placed between the vertical posts last erected before the traveller was moved to a new position. This limitation in width caused heavy stresses. The resulting deflections were therefore large and as safety demanded there should be no grade in the crane runway which would tend to run the cranes off the end of the traveller, the trusses were so framed that under the worst conditions of loading the runway rail would be level. This, as will be seen, required an additional height of about 6-inches at the ends. See Plate LXI.

The top trusses were designed to form a runway of sufficient length to permit the hoists to stand a little beyond the vertical of the furthest ahead members to be erected. The bottom chords and the web members were built of four angles and a plate in an " H " section. The top chord was a box section with a cover plate,— reinforced to carry the rail,— on the top flange and latticed on the bottom flanges. All connections were riveted. The trusses were connected to the tower by 10″ diameter pins.

The width of the tower was limited by the required clearance for erecting members between it and the truss members already in place and, further, by the economy to be found in using the outside girders carrying the permanent track for supporting the rail under the inside truck wheels the traveller being supported when moving on a double line of rails on each side of the bridge. The outer line of runway rails was supported on the main stringer girders provided for the panels of the permanent track but not yet erected and temporarily placed under the traveller 6′6″ outside the permanent stringers. These were braced to the permanent girders with temporary laterals in the plane of the top flanges and with frames opposite the sub-floor beams in each panel. Special girders were supplied for the outer rail support where the panel length was shortened in the end panels of the anchor and cantilever arms, there being no regular main stringers available of the proper length. The rails were 85 lbs. fastened by clips bolted to the tops of the girders, field holes being left for this purpose.

Plates XIII, XIV and LX.—Show the general construction of the trucks supporting the traveller. The centre of the forward trucks was placed 9'2" from the centre of the front column and the longitudinal distance between trucks was 66'11". The cast steel wheels were 33 inches in diameter, bronze bushed, running on 6" axles. When the traveller was being moved the two top travelling cranes were locked at the extreme rear of the upper runway and in this condition the load on one front truck was 486,000 lbs., while that on one rear truck was 450,000 lbs. In order to distribute the load equally on the six wheels of each truck, the reactions were taken by heavy spiral springs four to each axle. Each spring carried a load of about 21,000 lbs. while the traveller was in motion but was designed to take a load of 30,000 lbs. before it would compress to its solid height. The cross beams of the forward truck were connected by 6" pins to an equalizing beam placed between the webs of the truss chord.

When lifting with the cranes in the forward position the reaction on a front post was for some conditions about 1,300,000 lbs. and as the trucks were only designed for about half of this load the posts were extended through the bottom chords, in order that they might be shimmed to a solid bearing when the traveller was in operation. Pedestals were provided to carry the post reaction directly to the floor beam and as a matter of safety the columns were bolted to the pedestals by 2" diameter bolts, the pedestals having previously been securely bolted and braced to the floor system. The traveller was anchored to the floor girders at the rear against uplift and shims were placed to avoid the possibility of overloading the rear truck springs when the cranes were lifting towards that end.

The rail on the anchor arm is on a 1% grade, but due to the framing for camber this grade was accentuated in the erection of the cantilever arm. The grade also varied from panel to panel as the erection of the cantilever arm proceeded and means were provided for keeping the upper crane track level by an adjustment in the rear trucks. The load at each of these trucks was suspended through a 5" diameter bolt, the length of which could be varied by nuts. When necessary to adjust the level of the traveller, both cranes on top were moved to the forward end, materially decreasing the rear reaction and permitting the adjustment to be easily made.

For moving the traveller along the bridge floor 1¼" wire ropes were connected to the runway girders on each side of the traveller. These ropes passed through sheaves between the chords of the lower trusses and were connected to six part $1\frac{3}{4}$ manilla rope tackles which were hung on the rear face of the tower from the brackets carrying the swinging derrick booms. The two tackles were rove with a single line the ends of which were led to spools on the outside derrick hoists, thus equalizing the pull on both sides. This arrangement gave a speed of about 10 feet per minute and gave close and satisfactory control of the traveller movement.

Plate LXII. — Shows the arrangement of the cranes on top.

Each of the main cranes on top of the traveller was built of two single web plate girders 113′ 4″ long, 5′5″ deep and spaced $8\frac{1}{2}$ feet on centres, connected by top and bottom laterals and brace frames for that portion between the runways, 54 feet. The working range of the 60-ton hoists was from 7 ft. to 21 ft. outside the crane runways. To lessen the movement of the cantilever extension the trolleys carrying the heavy hoisting machines were placed inside and as near the centre as the necessary travel would permit.

The upper block of each main hoist was a trolley carrying six 24″ sheaves connected to the main trolley carrying the hoisting machinery by a spacing strut to control its movement and take the pull of the fall lines leading to the drum. The sheave trolley was carried on an auxiliary track inside the main crane girders which was supported by girders three feet deep headed into a diaphragm between the main girders over the truck and into another diaphragm 24′6″ further out. The auxiliary girders which were spaced 3′11″ apart were connected to the main girders by brace frames and by quarter inch plates on the top flanges, thus providing efficient lateral bracing for the cantilever extension of the main girders and for the auxiliary girders.

The lower block carried five corresponding sheaves. With its connecting shackle it weighed about three tons. A wire cable $\frac{7}{8}$″ diameter was rove through these sheaves making a ten part tackle, both ends of the line being carried to the winding drums, greatly reducing the friction loss in the blocks and securing a more uniform distribution of the load on the different parts of the line. With the usual single hoisting line the friction load on the hoisting part when lifting or on the becket end when lowering would be doubled and a heavier line would have become necessary.

The maximum lift of the hoist was about 330 feet, requiring 3500 feet of cable for each hoist. This was wound on two drums 36″ diameter and 31″ wide requiring five layers of cable for the blocks in the upper position. The winding drums were mounted on the same shaft which thus carried the entire bending load, but in order to give greater strength in the driving gears and pinions each drum was separately driven by a heavy cast steel gear next the frame. One drum was keyed to the shaft to cause rotation of the shaft in its bearings, but the other was free on the shaft to prevent unequal loads on the driving pinions.

The controller system was of the magnetic switch type arranged for dynamic braking with a maximum lowering speed of 12 ft. per minute. The armature shaft had an automatic disc brake capable of holding the maximum load. Special fuses were provided which would blow when the load reached its maximum, thus guarding against an attempt to lift loads greater than calculated for. As a further precaution a hand operated brake, independent of all other safety devices, of sufficient capacity to hold the maximum load was placed on the extension of the back gear shaft of the motor. To prevent over-winding a limit stop was installed which applied the brake by interrrupting the current when the block reached the limiting position.

The hoist and sheave carriage had a traversing motion of 14 feet. A 5-H.P. motor was geared to a rack rail to give a speed of 10 feet per minute under maximum loading. The traversing mechanism was made strong enough to resist a thrust of 30,000 lbs. due to fleeting loads from the centre of the traveller, but was not capable of moving against this pressure.

In addition to the two main hoists carried by each travelling crane there were two light hoists of 5-tons capacity at each end. These hoists were supported by gantry frames which spanned the sheave carriages and were carried on rails over the centre of the main girder. Each gantry supported two trolley tracks having a travel of 20 feet normal to the axis of the main crane. The over-hang was not symmetrical, being two feet from one support and nine feet from the other. Locking wheels underneath the top flange of the eye-beams were provided to keep the gantry in place and take the uplift from the maximum load at the end of the 9 ft. arm. The hoisting blocks were single sheave rove with ⅝ diameter wire rope wound on drums placed near the centre of the main cranes. The drums on these hoists were 16″ diameter geared to give a rope speed under full load of 150 feet per minute. The trolleys were racked in and out by hand wheels.

The runway rails carrying the cranes were 100 lbs. A.S.C.E. section planed flat on top. Each crane was carried on four two-wheel trucks with wheels 4′ centres giving a total wheel base on the runway of 12′6″. The wheels were steel, 30″ diameter, 3″ tread. The crane was driven by a 16 H.P. motor placed at the centre of and between the main girders operating a 3½ diameter squaring shaft leading to each side. The motor was geared to give a longitudinal travel of 25 ft. per minute.

The two derrick booms at the rear of the tower were 70 feet long and hinged on brackets extended diagonally to permit material to be taken from the tracks at the rear and passed around the tower. The forward booms were on brackets extending in front of the tower. A second bracket a storey lower down was provided at each front corner to carry the booms when extended to 90′ for placing the anchor arm staging. The extension was made by inserting a 20′ section.

The booms were built of four 6 x 6 x $\frac{3}{8}$ angles latticed, the outside dimensions at the centre being 3′ x 2′6″. At the upper end two sets of sheaves were provided, one for the four part $\frac{3}{4}$ steel rope tackle of the lifting blocks and one set for the six part $\frac{3}{4}$ inch diameter tackle used for the topping lift. At the lower end the booms were pin connected to the pedestal, which was cast solid with a double bull wheel 4 feet diameter rigged for revolving the booms through an angle of 230°.

Electric hoists were placed on the second floor of the traveller about 18 feet above the track for operating the derrick booms. The hoist drums were 15″ diam. 31″ long, designed for a rope pull of 10,000 lbs. with eight layers of $\frac{3}{4}$ in. wire cable on a drum, power being supplied by a 51 H.P. motor, 295 R.P.M., geared to give a lifting speed of 37 feet per minute under full load, controlled with a drum type controller. Each hoist had in addition four spools 12 in. diameter and 15 in. long for manilla rope. The spools were loose on the shafts operated by jaw clutches. The bull wheel was revolved by a 5 H.P. Motor, 700 R.P.M. placed on the third floor, geared to give a rope speed of 25 feet per minute with a pull of 6,000 lbs. on the rope. The motor was furnished with a reversible controller and was fitted with a mechanically operated brake on the armature shaft with which the machine could be locked when not in use.

All hoisting ropes were of flexible plough steel, 6 strands with hemp centres, 19 wires per strand. All gears were of steel with cut teeth.

Direct current at 220 volts was used in all motors.

The main and auxiliary hoists on the cranes were equipped with full magnetic control and dynamic braking. Controllers for all the operations of the cranes were placed on the working platform immediately over the railroad tracks so that the operator could at all times see what was being done and get orders directly from the Foreman in charge. The controllers for the derrick operations, both hoisting and swinging, were placed near the hoisting machines, where the operators had a good view of the working space on the bridge floor.

Iron contact bars were used on top of the towers to deliver current to the electric cranes, 144 being required of $1\frac{1}{2}$″ x $\frac{3}{8}$ section. The connections from the contact shoes to the motors were made by copper wire as well as the connection from the contact bars to the controllers.

Figure 21.—Illustrates the contact bars as well as the general appearance of the hoists.

Except for racking the auxiliary Gantry hoists, operators were not required on the upper level but the machinery was constantly under

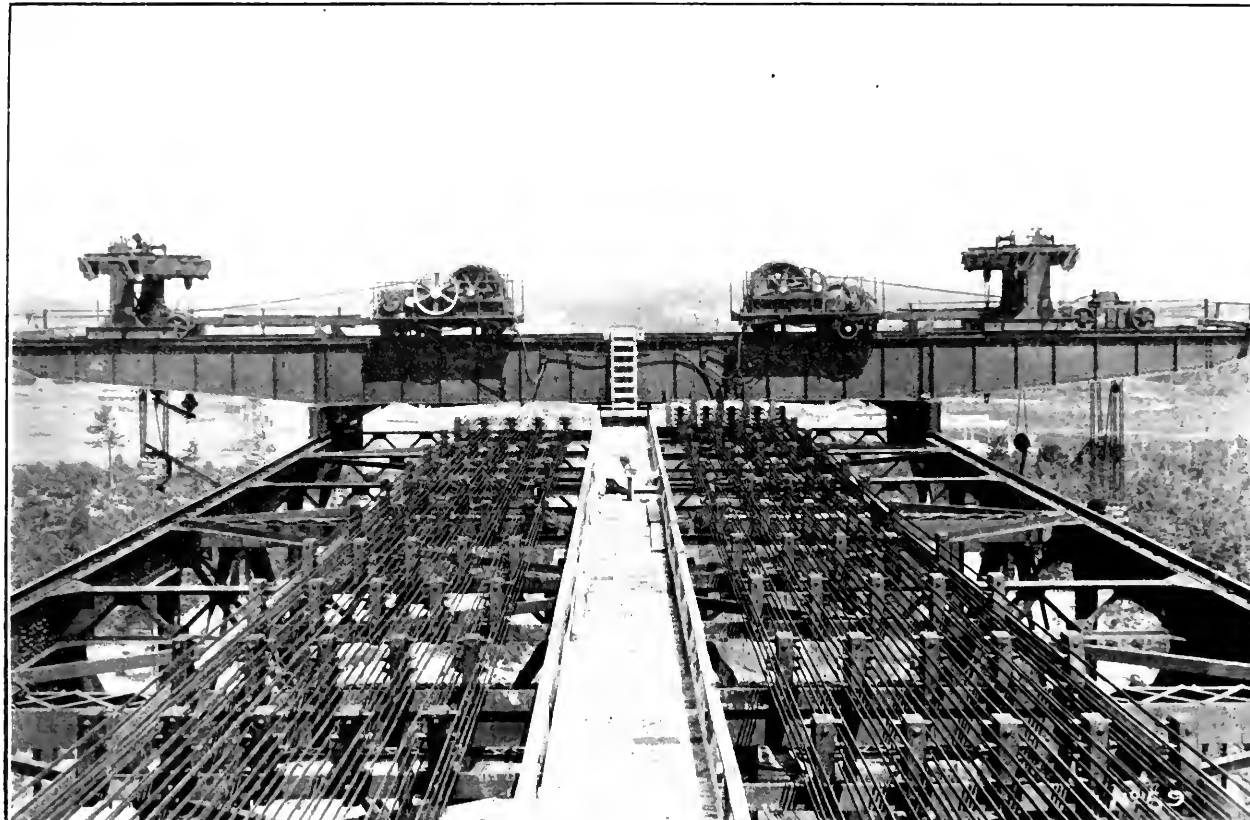


Figure 21 **Photograph of Traveller Crane and hoists.**

inspection and at least one man was kept on each machine to oil, inspect and apply the emergency brakes should occasion arise. The distance from the working platform to the crane girders was about 175 feet and for convenience an electric elevator was installed, operated by a 15 H.P. motor, geared for a rope speed of 150 feet per minute. The controller was arranged for dynamic control and placed in the car. The elevator shaft was encircled by wooden stairs.

Field telephones were liberally distributed over the traveller and working parts and placed wherever communication could be facilitated thereby.

The switchboards, magnetic control panels, resistance coils, etc., were installed in a fire proof building placed directly under the rear platform. The building was mounted on wheels running on the top flanges of the inside permanent track girders and was coupled to the traveller. The feed wires were wound on reels mounted on a platform attached to the switchboard house. The platform was also useful to carry shackles, rope, small tools and other extra equipment.

In using the traveller for erection of material special cantilever track stringers were placed to extend the working tracks about 20 feet in front of the forward floorbeam. Material was taken from the storage yard to the work on two special trucks. The forward truck was run out on the cantilever track and that end of the member lifted by the outside crane until the member was carried by this crane and the rear truck. The member was then run forward until the rear crane could be connected and the member lifted clear of the rear trucks.

Special fleeting tackles operated by the forward derricks were provided to keep the main crane blocks centred over the railway tracks. Truss members were moved forward in pairs, one on each track, and fleeted one against the other. When the members were lifted from the trucks the cranes were moved forward until the members could be moved transversely past the forward legs of the traveller. The fleeting tackles were then slacked until the falls hung vertically. When in this position the fleeting tackle was cast off and the members moved into the positions required by means of the crane and trolley motors under the control of the operator. If the truss members were too long to be fleeted past the forward end of the traveller in a horizontal position, the forward end was raised until the member cleared.

Plate LXIII. — Shows the manner of rigging one of these tackles.

Transverse members such as floor beams which could not be handled in pairs were loaded on trucks in such a manner that the centre could be carried beyond the forward legs and hoisted by the two 60-ton blocks from the forward crane, which were coupled together and connected to a special device. The lifting hook was provided with a ball-bearing

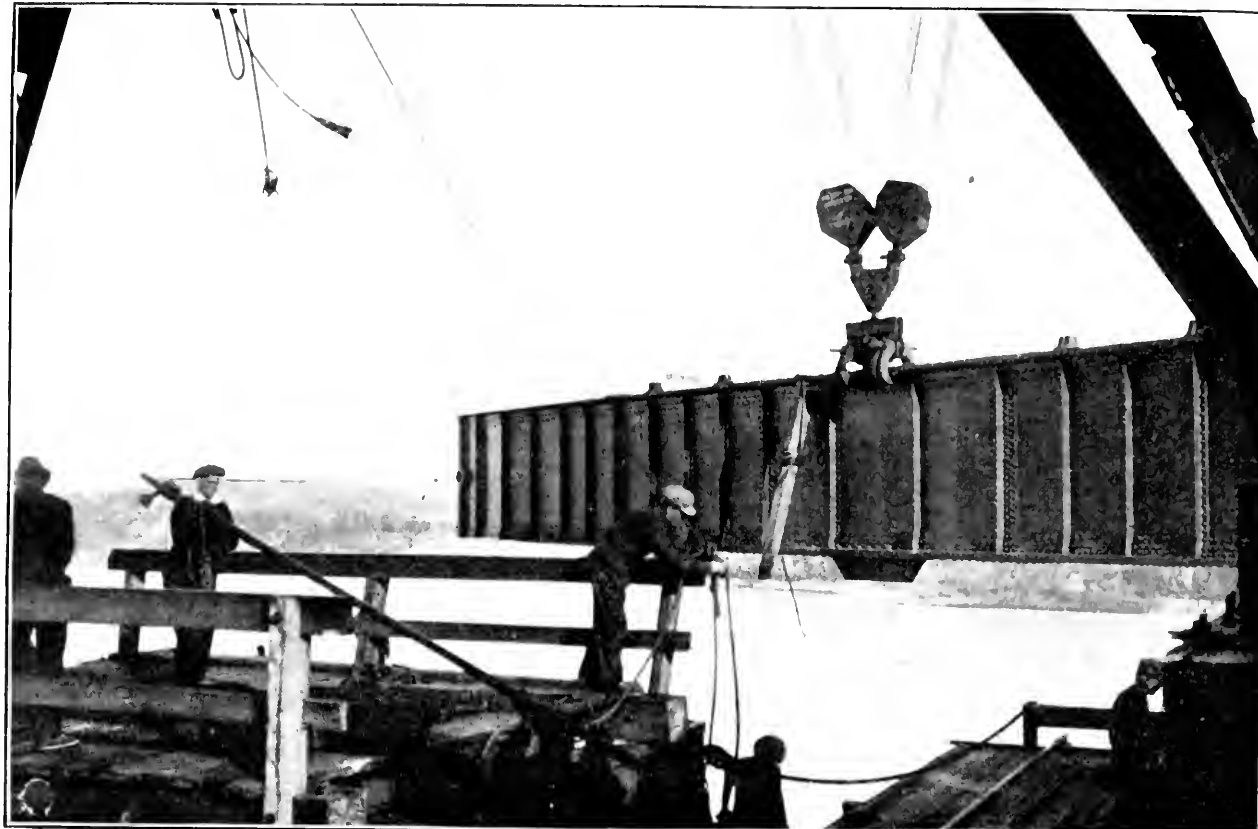


Figure 22 Photograph showing swivel for turning floorbeams.

swivel to permit the floor beams to be easily turned when suspended (See Fig. 22). Laterals, sway bracing and other light transverse members were handled directly with the derrick, sling chains being used for connecting to them in the customary way.

The lower supporting trusses of the traveller were erected with locomotive cranes and the upper stories by an ordinary wooden guy derrick operated by a two drum steam hoist. The derrick mast was placed in the centre of the tower, special provision being made for suspending it as it was moved up. The crane girders on top were raised in two pieces and spliced at the centre with finished bolts when in place.

The traveller on the North side was erected on shore during the winter of 1913-14. To insure a level foundation and that the traveller should be in the proper position to run on to the Bridge, four spans of plate girders were set on concrete foundations at the end of the approach spans. The girders were later used to carry the outside tracks for the traveller at the ends of the cantilevers as well as in other places during the progress of the work.

(c) *Erection of Approach Spans:*

The 107 and 151 ft. spans at the North end of the bridge were erected during the Summer of 1913 by locomotive cranes. Timber staging was used under the short end span and steel staging for the longer span reaching to the anchor pier.

It was necessary to move the traveller from the position in which it had been erected on shore across these spans in order that it might reach the anchor pier — the position where it was first put in use. When the spans were completed, temporary brackets were attached to the outside trusses to support the girders carrying the outer rails of the traveller tracks.

Plate LXIV. — Shows the method of supporting the temporary girders.

The approach spans were two independent single track structures and the traveller loads were applied off the centre line of each span. To prevent the eccentric loading causing serious lateral stresses and to distribute some of the traveller load on the inside trusses, the two spans were connected together by temporary struts opposite the brackets. Special clamps were also used to connect to the girder under panel K 3. This girder had been used as the top of the staging bent during the erection of the approach span.

(d) *Steel Falsework and Staging:*

The loads on the staging due to the weight of the structure and the erection equipment were so heavy that timber could not be used and steel construction on concrete foundations was necessary throughout.

The staging was arranged in two distinct parts. The first part called "Inside Staging" was designed to carry the floor and the traveller. The second part called "Outside Staging" was placed on separate foundations directly under the main trusses and was designed to carry the entire weight of the anchor arm including the floor.

It was planned to use the inside staging of the North shore for the same purpose on the South shore. It was necessary to leave the outside staging under the trusses of the North shore anchor arm until the cantilever arm had been built out so as to fully balance the weight of the anchor arm. Two sets of outside staging were therefore provided for the bridge but only one set of inside staging, to be used first on the North shore and then altered as necessary for use on the South shore.

Plate LXV. — Shows the layout of foundations for both inside and outside staging with their relation to each other, also the arrangement of anchor bolts.

Inside Staging:

Plate LXVI. — Shows the general outline of the staging in elevation and section together with the stresses and material in all members.

It will be noted that vertical posts were used in the upper storey of the bents, the posts being under the centres of the traveller legs, but the posts were battered below a plane which cleared the lateral bracing in the bottom chords of the trusses. This arrangement seemed to be the only way of providing room for the laterals and sway bracing with safe clearance while at the same time adopting a uniform construction for the bents and gaining a sufficient base for stability.

The clearances between the staging and the bracing of the trusses were in many places so close and at so many different elevations that to avoid any possibility of error it was considered prudent to construct a wooden model of the staging and of the anchor arm on a scale of ¼ inch to the foot. A model wooden traveller was also built and the complete model used for instructing the foremen in the order of erection and the method of handling the members.

Figure 23. — Shows this model.

Bents 1 and 3, 7 and 9, 11 and 13 were connected by longitudinal bracing to form towers but bent 5 was braced longitudinally by struts connecting to the other towers. It will also be observed that temporary struts were used between 9 and 11 to stay bent 11 during erection.

Bents 1 to 11 rested on concrete pedestals but bent 13 was located between the caissons of the old and new piers and it was necessary to carry it on heavy box girders which spanned from the old pier to a concrete seat on the new caisson, These girders were designed to be used later for supporting and lifting the ends of the suspended span.

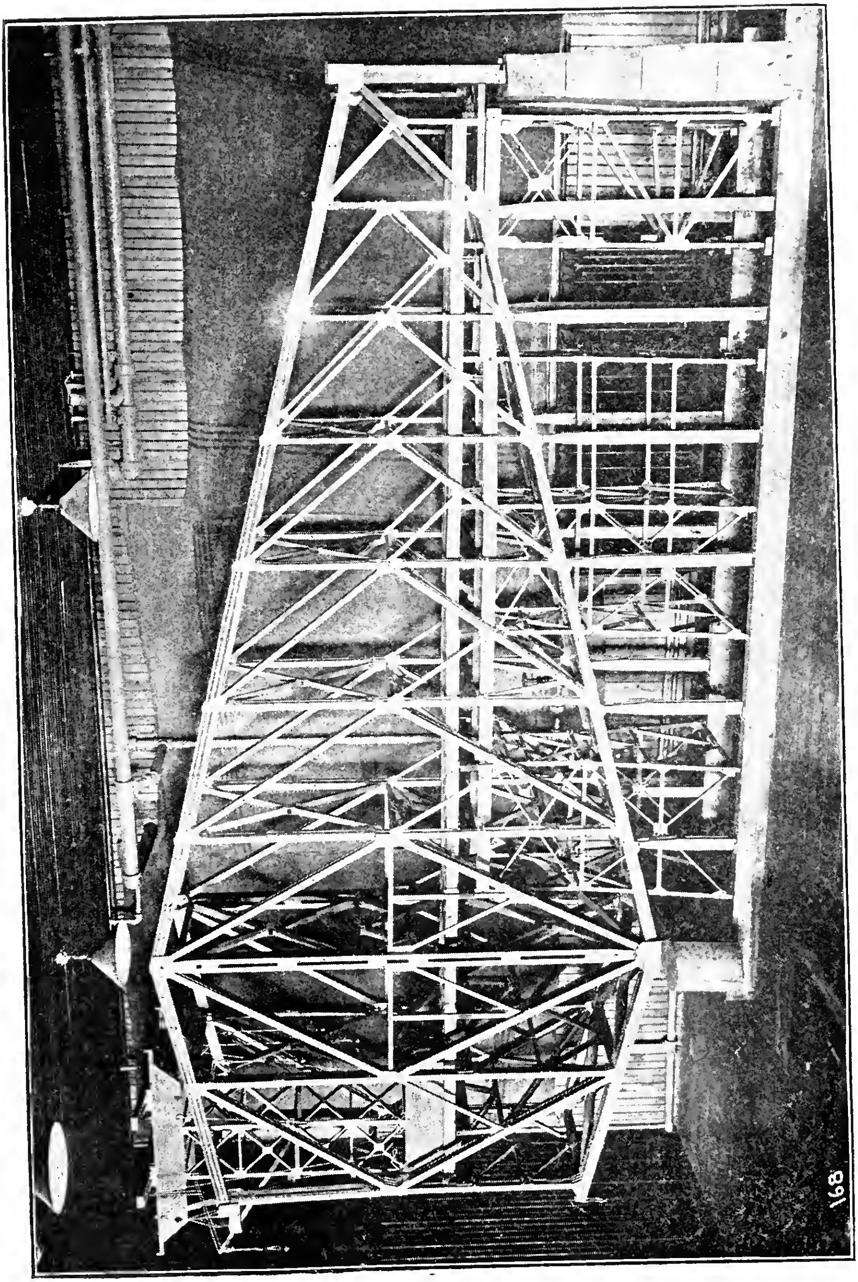


Figure 23 Photograph of Model.

The vertical columns of the bents were extended to a height that would permit the columns of the traveller to be shimmed directly over them. Brackets were attached to the columns of the bents to support a longitudinal girder on each side. These girders were spaced 5′ apart and were braced together in pairs. The permanent floor beams of the bridge rested on these girders but sufficiently removed from their final position to permit the vertical truss members to be erected without interference, the connection of the floor beams to the trusses not being made until the truss was completed, as will be later explained.

The floor stringers and floor system of the bridge were laid on the floor beams, special provision being made for the temporary shifting of the points of supports and where, as in the end panel, the stringers were notched out for the floor beams, short bents were used to carry the stringers. The outer temporary stringers supporting the traveller tracks were also placed as required and braced to the outer permanent stringers.

Plate LXVII. — Shows the general arrangement for the first two panels which are typical of the construction throughout.

Plate LXVIII. — Shows the location and displacement provided for.

To allow the floor beams to be brought to a position for connecting before the truss had taken its final form, clearance was provided above the tops of the longitudinal girders and steel shims were inserted between the floor beams and these girders. Shims were also placed between the floor beams and track girders supporting the floor and traveller track to provide adjustment for the track grade and to give ample clearance above and below the floor beams when adjusting for attachment to the trusses.

The inside staging was designed for use again on the South shore after the North anchor arm had been erected. The ground elevation was generally higher on the South side and short sections were used in the lower columns, the removal of which adjusted the bents to the heights of the South shore foundations. Sway brace connections were provided for both conditions on the same gusset plates. The sway bracing was fabricated the proper length for use on the North side and was cut off with a hack saw in the field to the required length for the South side, both sets of connecting holes having been punched when the members were fabricated. The top and bottom sections were used a third time to form part of the staging for the suspended span during erection and were detailed with this purpose in view.

Outside Staging:

Plate LXIX. — Shows the general arrangement of the outside staging and

Plate LXX. — Is a stress and material sheet of this staging. Each bent was a self-supporting tower built of four columns 6 ft. centres well braced on all four faces and having horizontal bracing. Each column was anchored to the masonry with two $2\frac{1}{2}$ inch bolts.

Temporary shoes were bolted to the bottom chord at each panel point, the ribs of the shoe bearing directly on the chord ribs. These shoes rested on pins carried in the tops of the outside staging towers — the connecting pins being in two lengths each only long enough to take the bearing of two ribs of the chord.

Floor beams provided for the cantilever arm were temporarily used between bents 6 and 8, 8 and 10, and 10 and 12, to carry timber bents for supporting the bottom chords at the mid-panel splices. These floor beams were braced together in pairs and supported on brackets fastened to the columns of the staging.

The columns were connected in pairs near the bottom by heavy box girders of sufficient capacity to transfer the load to the hydraulic jacks, used for adjusting the height of the bents. The bents were set to provide for camber as already noted, the point L_2 being 5 inches below its final position. The further adjustment required after the chords were riveted was provided by about 4 inches of steel shims placed under each leg. The shims were divided on the line of the anchor bolts, half holes being punched so that they could be easily taken out or put in place. The anchor bolts were made long enough for vertical adjustment and somewhat heavier than required by the vertical up-lift to guard against any possible displacement during the operation of jacking.

The North traveller was moved into position to begin erecting the inside staging on the 20th May 1914 and the staging and floor for the North anchor arm were completely erected by the 20th of July with the traveller in position to place the main pedestals.

(c) *Erection of North Anchor Arm:*

The scheme of erection starting from the base formed by the pedestals on the main piers and projecting both anchor and cantilever arms from this base required that the shoes should be set exactly as to elevation, distance apart and alignment:

The bridge seats in the main piers had been dressed with extreme care to a perfectly plane surface at exact elevation. Copper plugs were leaded into the masonry on which were marked the intersection of the longitudinal axis of each truss and the transverse axis of the pier with the four edges of the cast steel base of the shoe. Each of the bases was 20 feet wide, 26′4″ long and 4 ft. high, made up of four separate castings. When assembling the shoes and bases at the shop great care was exercised to maintain the pins in their exact relation to the edges of the castings and marks were placed on the castings which corresponded to the marks on the

Figure 24 Photograph of 30" pin and 45" sleeves being placed.

plugs set in the masonry. The surfaces of the masonry and of the castings were so nearly perfect that it was considered the bedding could be best made with only a thin wash of cement.

The pedestals were fastened to the masonry by twenty-eight (28) anchor bolts and sixteen (16) dowels, all 3″ diameter. The holes for these were drilled in the top of the masonry with a 4″ Calyx drill to a depth of 5 feet. The bolts and dowels were set to a steel template and grouted in with neat cement before the castings were set in place. When the castings had been set and carefully checked for position, the space between the anchor bolt and the casting in each hole was filled with lead and all nuts turned down tight. The dowels projected into the castings through holes cored in the bottom. After the shoe was assembled the castings were filled solid with concrete. Having placed the base castings the main portions of the shoes were assembled and the connecting holes brought to match. Before proceeding with the erection of the truss sufficient rivets connecting shoes and bases were driven to insure against any displacement while handling the heavy pins and chords which connected to them.

The pins and sleeves connecting the anchor arm bottom chord were placed in position in the manner illustrated on Fig. 24.

Erection of Bottom Chords:

Owing to their great weight these heavy members were shipped from the shop in quarters, being divided at mid length and mid width. Field splices in each chord had been arranged to keep the weights within the capacity of the traveller and of transportation.

The total weight of one panel of chords 12-14, including the details was 392 tons, but by shipping it in four parts the maximum weight of a single piece was 92.5 tons. A temporary bracket was attached to the shoe under each rib to support the ends of the half panel, L 13-14, abutting against the pin and to hold them in close contact with the connecting sleeve. The other end was supported on a bracket extended from bent 13 of the inner staging leg, a temporary wooden shoe being bolted to each rib of the chord section and adjusted to the correct height by shims. The plates for the centre splice were placed in position but only loosely bolted.

The two sections 12-13 comprising the second half of the panel including the detail at 12 were then placed resting on column 12 and supported at the inner end by the splice plates. When making this connection the end of the chord of bent 13 was kept slightly below its normal position to insure the upper edges of the joint being in contact. These edges were drawn together by means of heavy bolts set up against buck angles bolted to the top flanges of the abutting pieces. The joint was then raised by jacks placed under each rib until the surfaces of the adjoining pieces were in close contact throughout when the splice plates were thoroughly drifted and bolted for riveting, the shims being adjusted to hold the member in position. The bottom laterals connecting from panel point 12 to the shoes were connected and the alignment of each chord checked.

For erecting the first section of chord $L_{10}L_{12}$ the traveller was placed at bent 11. The inner end of the section was supported by the splice plates while the outer end was carried on a timber bent on top of the temporary floor beams spanning between staging bents 10 and 12.

Riveting was not started until this panel had been completely assembled and adjusted to insure that no displacement would take place at the first splice; thereafter riveting generally followed about a panel behind the assembling.

Panels 6-8-10 were handled and supported in the same manner as 10-12, the bottom laterals keeping the work in line.

In panel 4-6 the chord being lighter, the centre splice was riveted in the shop before shipment and a middle support was unnecessary.

It has been explained under the description of the design that each truss was laid out as to independent half trusses for the sake of ease in erection and to assure a proper distribution of stress in all parts of each member. Until the joint of panel 4 was reached the chords were handled in half sections as above and connected only by tie plates, care being taken, however, that the two ends were always in the same plane. The tie plates connecting the two half trusses were not riveted until the chord was completely assembled from end to end.

The detail at L_2 being very heavy and of complicated construction, was made in one unit without longitudinal division and was field spliced to the bottom chord 2-4. The section of 2-4 between field splices was thus without the heavy details required for panel connections and the area of the member being comparatively small it was possible to handle the whole section in one piece. After opposite chord sections were in position the splices were adjusted and the laterals connected. It was found in each instance that the abutting ends were in actual and close contact over their entire surfaces. When the bottom chords and lateral system were completed, the erection was stopped until the bottom chord splices were completely riveted and supporting towers for 4, 6, 8, 10 were jacked down to the elevation shown in the Diagram, Fig. 2, of Plate XLIX which corresponded to the calculated deformations of the panel points produced by the framing for no load conditions.

While the riveting and adjusting of the bottom chord was proceeding the traveller was being returned to bent 13 from which position was started the erection of the web members comprising the lower triangles of the truss.

Erection of Lower Web Triangles:

The erection was planned to avoid the use of temporary staging above the bottom chord with the exception of a short strut used to support $M_{13}L_{14}$ and a strut to stiffen the tension eye bars at M_2L_2.

Plate LXXI. — Shows the position of the traveller and gives the order of procedure for erecting the first panel $M._{12}$-$L._{14}$. The vertical member $M_{12}L_{12}$ was first erected. It was handled full length but as its weight was about 116 tons it was divided longitudinally in two parts in the same manner as the chord sections.

The inside half of the member was first erected in order that it might be braced, to stand alone, by timbers clamped on each side and connected to the projecting ends of the floor beams. The outside half was only placed after the sway bracing had been erected and connected to the inside half, and was then permanently connected by the tie plates and the connections to carry the pins supporting the floor beams.

Plate LXXII. — Shows the temporary supports and brackets used in placing $M_{13}L_{14}$. The 42 ft. plate girder used as a strut was one of those built for a platform on which to erect the traveller on shore. It was set upon jacks and braced normal to the chord. At L_{14} a temporary tension connection was made with riveted steel brackets to provide against a possible up-lift from wind force. The member $M_{12}M_{13}$ was also erected in two halves, each weighing about 72 tons. The splices at M_{13} were only bolted until the pins at M_{12} were driven. After the splice was adjusted to a true bearing by means of the jacks and shims under the ends of the temporary struts it was fully riveted before the erection was proceeded with.

The vertical $M_{10}L_{10}$ was erected and temporarily braced in a similar manner to $M_{12}L_{12}$.

Plate LXXIII. — Shows temporary wire rope supporting tackles which were first made use of in this panel. The drawing shows the method of attachment and the manner in which they were used in a number of panels. The tackles were provided with a special stopping-off device attached to the floor beams by which the load could be held in any position. A tackle was provided for each longitudinal half of a truss, making four in all.

After the erection of $M_{10}L_{10}$, two of these tackles were attached to the ends of the pins at M_{10}. The inner half of the member $M_{11}L_{12}$ was stood on its pin at L_{12} and temporarily supported by one of the tackles. Possible uplift was provided against by making a temporary tension connection to the chord. After the sway bracing was connected for lateral stability the outer half of the member was placed and held up in a similar manner and the tie plates connecting the two halves bolted in place.

The sub-member $M_{11}M_{12}$ was then put in place, the pin holes being brought to register by the supporting tackles which were released as soon as the pins were driven.

The upper half of the diagonal $M_{10}M_{11}$ was then erected; first the inner and then the outer half, and supported by the adjusting tackles attached near M_{10}. The alignment was adjusted by the tackles and the splice riveted up. After the splice was completely riveted the member was sprung by the tackles as necessary to adjust the distance between pins L_{10} and M_{12} and the pins at the latter point were driven. The lower triangles of the remaining panels were erected in the same way.

Plate LXXIV. — Shows the method of erecting M_2L_2. This member was made of eight eye-bars with temporary steel columns, to take compression during erection, placed between them in such a manner that they could be readily removed after having served their purpose. One column was provided for each half truss and the four bars clamped to it with the proper spacing, as shown on Figure 4. Plate LXXIV. They were then handled in a horizontal position to place and the pin L_2 driven after which they were revolved to a vertical position and temporarily guyed in place.

M_3L_4 was placed and held in position with the rear crane while the pin at M_3 was driven. Temporary wind connections were made to the bottom chord and the sway bracing placed.

The diagonal M_2M_3 was then placed and held back by tackle while the splices were being riveted up. It was afterwards adjusted and the pin driven at M_2 when the temporary guys were removed.

The diagonal M_1L_2 was erected in one piece without splice and connected to the sub-hanger M_1L_1.

The compression web members were temporarily connected at the bottom to the chords to resist wind forces.

While the traveller was in this position the anchor bars were placed. The lower lift of bars below A_2 were connected to the anchor girders and braced while the masonry was being built and a well was left for the bars above A_2. These bars were made in four lengths and the lengths arranged to bring the L_0 pin in a position where it might be supported on the masonry, thus carrying the weight of the string of bars while the upper connection at U_0 was being made.

The bars above the L_0 were toggled to provide for the position of L_2 being 5 inches below the normal, for facilitating the connection at U_0 and to control the straightening of the bars as they took their load with the extension of the cantilever.

Plate LXXV. — Shows the arrangement of the toggle and the false-work for supporting it.

An eyebeam grillage was placed across the well in the anchor pier to support the pins at L_0 and a timber tower was erected to hold the bars in position above the pier.

The bars below the pin L_0 were kept parallel by means of plate links connecting the pins. There were 24 of these bars in each corner arranged in groups of six on one pin. Three lengths of each group were lowered to the ground and coupled. When assembled in this way they were hoisted by the rear crane and dropped into position through the grillage on which the point L_0 was afterwards supported. The elevation of this point was adjusted to allow the two lengths to hang with a little slack to avoid the possibility of stress due to contraction in cold weather. The adjusting arrangement for the toggle was then laid on the timber tower and the upper portion of the tower completed. The upper lengths of bars were placed in groups of six and connected at M_0.

The post M_2U_2 was erected and held in a vertical position by the forward crane until the sub-diagonal M_1U_2 was placed and connected at each end. The supporting tackles were then attached to M_2U_2 as shown on Plate LXXV to support M_2U_0 which, with its U detail, was placed in one piece. The splice at M_1 was adjusted by means of the supporting tackles to a true bearing throughout and completely riveted before further connections were made.

U_4M_4 was next erected in one piece, and held in position by temporary brackets and bolts on the shore side and a wire rope guy adjusted with a steamboat ratchet on the other side. See Plate LXXIV.

The tension diagonals M_2U_4 were erected and pins driven at each end; the pin hole at U_4 being elongated to allow a small adjustment in the distance between U_2 and U_4.

The top chord panel U_2U_4 was then erected. The bars were packed in two groups in their supporting trusses at the yard before being brought on to the bridge — one pair of trusses and a group of bars for each half of the main truss.

Brackets were placed on each side of the vertical posts to support the trusses carrying the bars, and the trusses were bolted to these brackets, oblong holes were used to avoid any chance of stress being carried through the supporting trusses. Connections were provided in the frames for holding the ends of the bars in a straight line at the proper spacing.

Top chord panel U_0U_2 was placed in a similar manner and pins driven at U_0 and U_2, the adjustment of the distance between U_0 and U_2 being made by the temporary holding tackle supporting M_1U_0.

The toggle in the anchor bars was then adjusted to register the pin holes in the upper ends of the bars with those in the diagonal. After this connection was made the timber tower was removed and the portal bracing placed.

The erection of the succeeding upper triangles to the point U_{10} was similar except where the members were too heavy to erect in one piece, as in the case of the vertical post U_8M_8 which was divided longitudinally, each half being held by wire rope guys and temporary brackets.

The top chord panel U_6U_8 was too heavy to be handled in two parts complete and only the bars inside the supporting trusses were packed in the yard, the bars on the outside being sent out singly and slipped on the ends of the pins. The oblong holes in the eye-bars, before alluded to, made this operation comparatively easy, the camber being so adjusted that the top chord panels were one inch short.

The traveller was then moved to bent 11 and the sway bracing placed in $U_{10}M_{10}$ when the work of raising steel on the North shore was suspended for the season of 1914.

The floor beams from F_3 to F_8 were then connected to the main trusses and the weight of floor transferred from the inside staging to the trusses, thus releasing a portion of the staging which was removed and shipped to the South side during the remainder of the season 1914.

South Approach Span:

The South approach span was erected during the season of 1914 on timber staging with an ordinary bridge erection car.

The South traveller was erected over the anchor pier in position to start setting the staging and was ready to begin erection of the South anchor arm on May 20th, 1915. The erection of the South anchor arm was similar to that of the North and will not be referred to again.

Continuance of North Anchor Arm Erection:

The work of raising steel was resumed on the North side April 15th, 1915. The length of the vertical post at 12 being about 121 feet, it could not readily be brought through the traveller and passed to the outside. Each half, moreover, weighed about 65 tons, exceeding the capacity of one main hoist on the traveller. A field splice was introduced 53′7″ from the bottom end and the lower portion handled in one piece. The upper section was handled in two parts.

The diagonal tension members $M_{10}U_{12}$ were 137′3″ centres of pins, and for convenience in transportation and erection a field splice was made at about one-third the length. To erect these members the inside part was fleeted through the traveller in two pieces laid down on the floor beams outside the traveller and the splice riveted. The lower end of

$M_{12}U_{12}$ was then placed in position, guyed to M_{14} and connected by temporary brackets and bolts on the shore side as shown in Fig. 1, Plate LXXIV.

The diagonal $M_{10}U_{12}$ was then raised, the pin driven at M_{10} and rested on the top of the lower portion of $M_{12}U_{12}$ already in position to which it was connected, thus giving a firm support for the vertical member.

The upper outside half of the vertical member was then placed and spliced to the lower portion, when the outside tension member, which had been spliced on the floor, was raised complete and pins driven at M_{10} and U_{12}, after which the inside half of the vertical member was fleeted out and suspended by the forward crane. The rear crane was connected to the upper end of the diagonal which had been supported on the lower portion of the vertical member. This diagonal was then swung up until the upper portion of the inside vertical member could be placed and the splice connected, when the diagonal was revolved about pin at M_{10} until the upper holes matched those in the vertical, permitting the connecting pin at U_{12} to be driven. The vertical member was then lined up and the splice riveted. The top chord $U_{10}U_{12}$ was erected in the same manner as U_8U_{10}, the outside bars being placed after pin U_{12} was driven.

(f) *Erection of Main Posts:*

After the completion of panel $A_{10}A_{12}$ the traveller was moved to bent 13 to erect the main vertical posts. The bottom sections of the posts were tapered to carry the loads to the lower pin, being 8′11″ wide at the top and 5′9¾″ at the bottom. They were handled in two parts of two webs each weighing 60 tons, or the full capacity of one main hoist. The pieces were placed on the sleeves and pins through which they bore on the shoe, carefully plumbed by a transit and held securely in position by temporary brackets shown on Plate LXXII.

The next section above consisted of four main parts or corner posts and four minor parts made up of the lattice connecting the main sections. Each main section was 54 feet long and weighed 50 tons. The two sections next to the anchor arm were placed first and supported in a vertical position by temporary struts, one for each piece, connecting to the compression diagonal (See Plate LXXII). The minor parts containing the lattice were then placed and bolted to the two sections erected, when the two remaining main sections were erected and connected to the lattice parts, the tie plates connecting the four sections being afterwards bolted on.

The third section was similar to the second and was handled in a similar manner. After erecting it the lower sway bracing was placed between the columns, the floor beam erected and the main floor system laid to F_{14}.

The middle sections M_{14} were 36'6" long and carried the details for connecting the sub-members. After this section was completely assembled the sub-diagonal $M_{13}M_{14}$ was erected and the pin driven at M_{14}, but the lower end at M_{13} was not permanently connected until later.

The horizontal member $M_{12}M_{14}$ was provided with a screw connection and a heavy ratchet near M_{14} so that the length could be adjusted as the work progressed and it was connected at both ends.

Sections 5 and 6 were similar to Sections 2 and 3 and were handled in the same manner but no longitudinal or transverse bracing was required other than the lattice and tie plates connecting the four main parts of each section, which were quite sufficient to give lateral support.

The top section was similar to the bottom section but inverted and was bored for a 30 inch pin without sleeves.

The top link U_{14} was made of six ribs and divided for erection into two parts of three ribs each, each part weighing 70 tons. The side elevation was 16 ft. square and the width of the cover plates connecting the three ribs was 5'5" each. Each half was carried through the traveller on the pit cars which were used for shipping it from the shop and raised by the two main hoists on one side of the traveller for which special connections were provided on the member. The 30-inch pins were placed in the links before lifting, making it only necessary to raise the member to its position, the connection being made at the top when the pin was dropped into the half pin hole in the top of the post.

Plate LXXIV.—Shows arrangements for holding the top link in position.

The three ribs were connected by transverse diaphragms which were directly over the outer edges of the posts. Angles were connected to the bottom of the diaphragms and to the top of the post with a space between for the adjusting shims required to keep the top link in place and prevent rocking on the pin. After the link was properly adjusted the connecting angles were bolted together. As a further measure of security, a tension connection was made between the outer edge of the top link and the cantilever side of the post to balance the weight of the anchor arm top chord and tension diagonals.

The allowance for deformation required that the top of the main post should be 14 inches shorewards from the vertical, as shown on Plate XLIX, but to maintain and check the alignment during erection it was erected in a vertical position and not tipped back until all splices and connections had been thoroughly bolted, when it was easily adjusted by means of the ratchet placed in $M_{12}M_{14}$. Before tipping the post

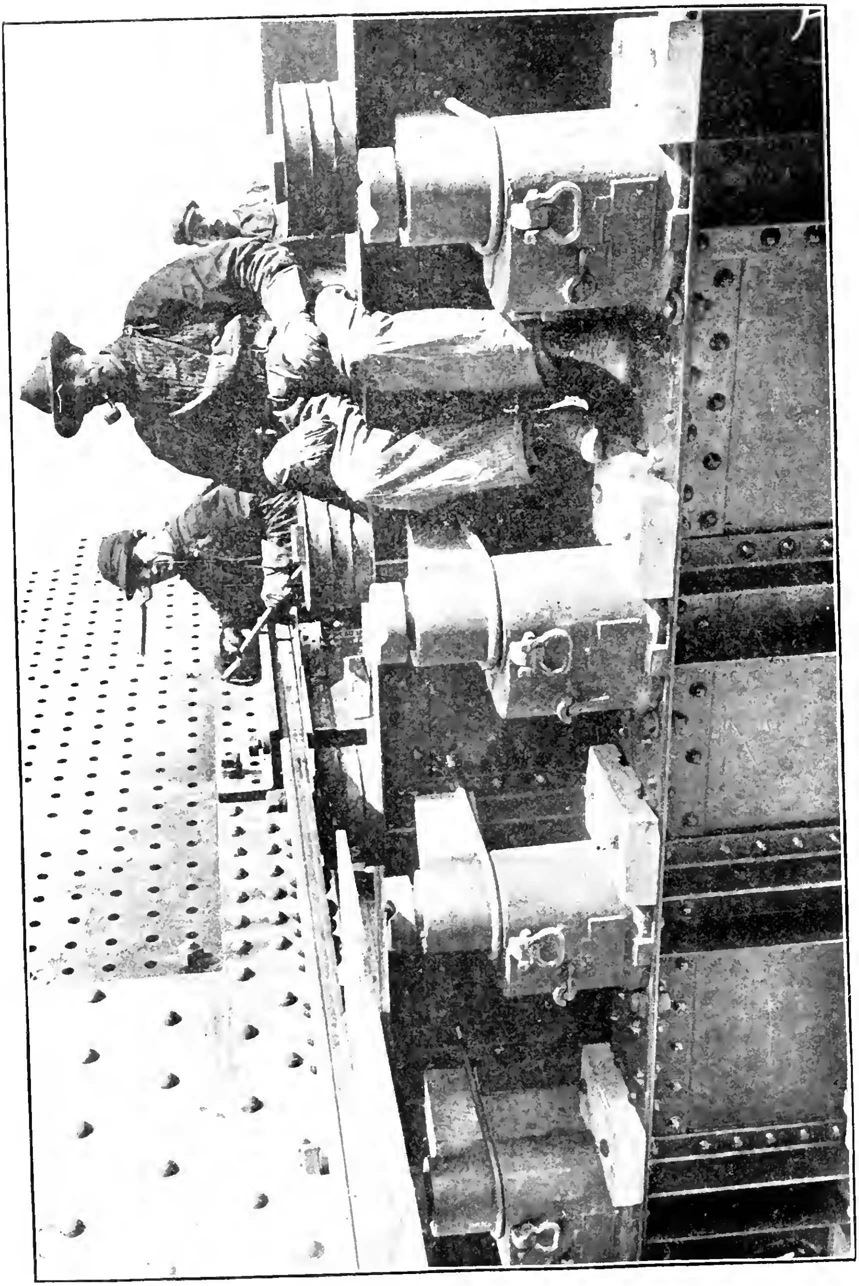


Figure 25 Photograph of 60-ton jacks on the erection bridge.

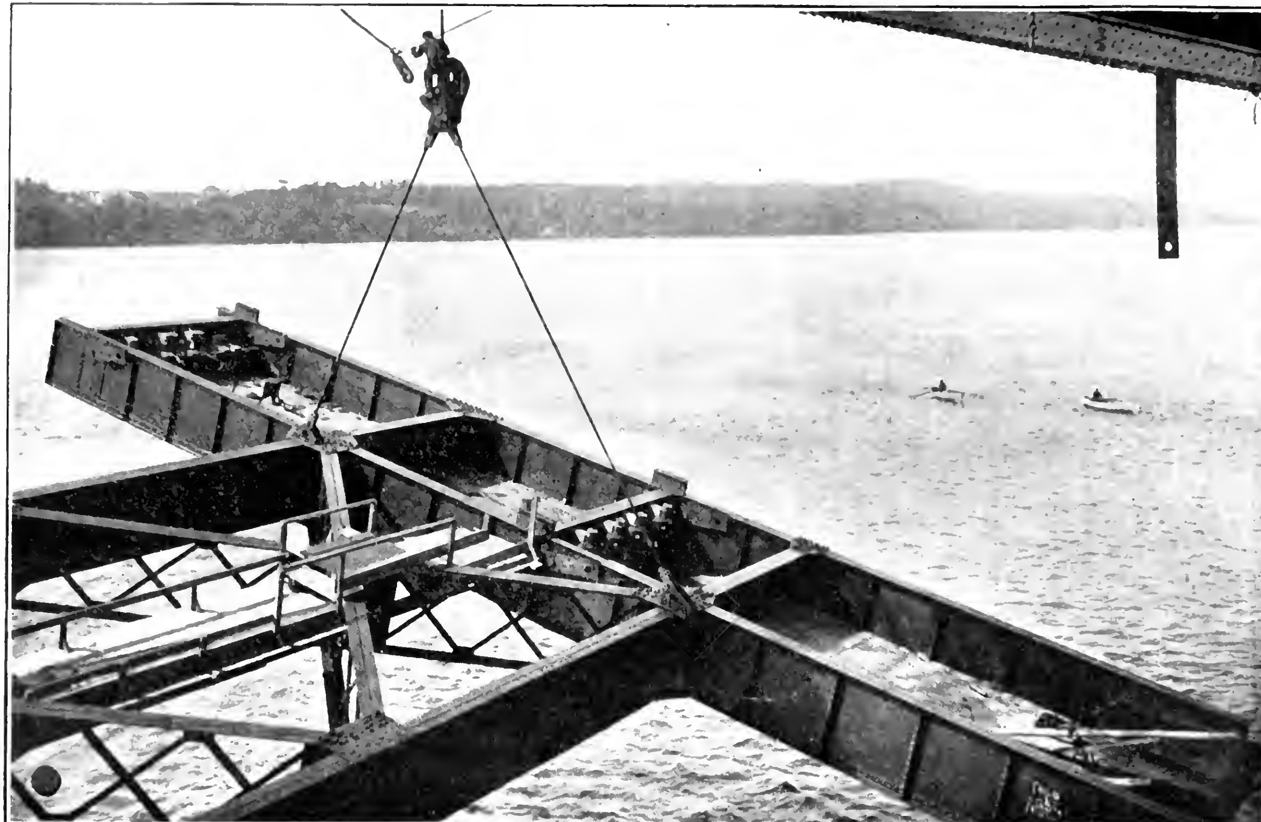


Figure 26 **Photograph showing erection bridge as it is being moved.**

back the temporary brackets at the bottom and the struts connecting to the second section were necessarily removed. When in position the pin at M_{13} was driven.

The main posts having been set to the calculated inclination, the tension diagonals $M_{12}U_{14}$ were erected and connected, the procedure being similar to that employed in placing $M_{10}U_{12}$. The top chord panel was then placed in the same manner as $U_{10}U_{12}$ and the panel completed.

(g) *The Erection of the Cantilever Arm:*

The design of the truss permitted each panel and sub-panel of the cantilever arm to be made self-supporting as the work progressed but, as already noted, equipment was provided of only sufficient capacity to lift one quarter of the weight of the heaviest chord panels. This necessitated some form of temporary support for the sections of bottom chords until the splices could be made and the full panel connected up.

Plate LXXVI. — Shows a movable platform provided for this purpose.

Plate LXXVII. — Shows the links and the manner of their attachment for supporting this platform in the different panels and Plate LXXVIII shows the manner of handling the platform from panel to panel.

Plate CXVI. — Shows the stresses in the platform and in the supporting links.

The platform consisted of two longitudinal girders for each truss, spaced 15 ft. apart and connected with cross girders and lateral bracing; the platforms on each side being further connected by cross struts and lateral bracing,making a unit of the whole construction. The supporting chains were made of plate links, 12¼″ by 1″ and 1½ inch of various lengths and provided with a number of pin holes to enable the different lengths required to be readily made up. The links were packed in pairs and special bales were provided for handling them. They were connected at the upper end to heavy box girder saddles which were temporarily fixed in convenient places to erected parts of the bridge and moved forward as the work progressed. The longitudinal thrust due to the inclination of the links was taken by a hinge thrusting against a bracket connected to the shoe in the first panel and in the succeeding panels against brackets connected to the under side of the bottom chord. The space between the two girders on each side was planked to make a convenient working platform.

A 60-ton jack was placed under each rib of the chord at the splice in the centre of the panel and a 100-ton jack under each rib near the end of the panel. These were used for adjusting the alignment and afterwards for adjusting the height of the end to allow the pin to be driven completing the panel. See Figs. 25 and 26.

The platform was erected by first assembling on the main pier that portion from the heel to the outside of the cross bracing, about 55 feet, swinging this into position, and after the traveller was moved to M_{15} placing with the traveller the outer sections, G_5, 28 ft. long.

After placing the inner section of the platform, as shown on Fig. 1, Plate LXXVIII, the pins and sleeves connecting the bottom chord to the shoe were placed in the same manner as the pins on the anchor side, and the two half sections, $L_{15}L_{16}$, of the bottom chord were lowered to position and supported at their outer ends on the cross girders of the platform. The half panel of bottom laterals was then placed, the outer ends being blocked up on the cross girders of the erection platform. The lower ends of the compression diagonal, $M_{15}L_{16}$, were then placed, the inside half first, and supported by the wire rope supporting tackle used in similar cases on the anchor arm.

Plate LXXIX. — Shows the manner of using these tackles in all the panels.

The sub-member, $M_{15}M_{16}$, was placed with the derrick booms and connected, thus supporting the compression diagonal and releasing the holding up tackles. The sway bracing $M_{15}L_{16}$, the subpost $M_{15}L_{16}$, the floor beam F_{15} were then placed and the floor was completed to F_{15}.

To give lateral stability temporary sway bracing was placed below the floor beam F_{15} and the floor laterals were supplemented with extra bracing.

Plate LXXX. — Shows the extra bracing, also the special pedestal used to carry the traveller reaction to the floor beam and the method of anchoring the traveller.

The material erected having been thoroughly bolted the traveller was moved forward to F_{15}.

Plate LXXXI. — Shows the members erected with the traveller standing at F_{15}.

The erection platform was then completed by placing the outer 28 feet and supporting this outer end by chains connected to saddle near M_{15} on the compression diagonal.

Chord section $L_{14}L_{15}$ was then placed in two parts, carefully aligned and the middle splice riveted.

The upper half of the compression diagonal $M_{14}M_{15}$ was placed next and supported by holding tackles. The outer half was erected first and thoroughly bolted to provide lateral support. When the inner half was erected the tie plates were connected and sway bracing placed below the floor level. The alignment with the lower half was carefully made with the holding tackle and the splice riveted up, when the sub-member $M_{14}M_{16}$ and the vertical $M_{15}F_{15}$ supporting this member in the centre were connected. The member $M_{14}M_{16}$ had a sliding connection at 16 so designed that the member could take no stress and would adjust itself to the varying distance between M_{16} and M_{14} as the work progressed.

The long diagonals M_{14} and U_{16} were fleeted through the traveller in two lengths for each half truss, assembled on the deck and riveted, then raised in one length to position and the pin at U_{16} driven. To make the M_{14} connection it was necessary to spring the compression diagonal 2.68 inches from a straight line. This was accomplished by means of the holding up tackles supporting it and required an additional pull of about 24,000 lbs. The hole in the tension member at M_{14} was oblong to simplify the adjustment of the distance between the connecting holes.

The vertical tension member $M_{14}L_{14}$ was handled next in two parts and full length. Each half was fleeted through the traveller and supported from the upper pin hole by the outer crane in its extreme forward position. This allowed the member to be hung vertically about 7 ft. beyond its position in the truss. The lower end was then drawn in to connect with the bottom chord at L_{14} and the pin driven. A runner was placed at the outer end by which the member was revolved about the lower pin until the holes at the upper end registered. The upper holes were oblong to facilitate the adjustment of the distance between M_{14} and L_{14} which was accomplished by means of the hydraulic jacks on the erection platform. The sway bracing, the floor beam F_{14} and the traveller track were then placed and the floor laid to F_{14}. The track girders only reached to a point half way between the two floor beams at F_{14} and a temporary extension bracket was bolted to the ends of the track girders to distribute the load on both beams.

The vertical post $U_{14}M_{14}$ was placed next. The member was handled in three parts; first the lower part with four ribs complete was set on the pins M_{14} and secured against lateral movement by the temporary anchor bolts on the outside and on the inside by plates bolted to the detail of the diagonal. The outside half of the upper end was then placed, the splice bolted and guy lines attached. Afterwards the inside half was erected and guyed in a similar manner and the tie plates connecting the two halves bolted on.

The top chord panel $U_{14}U_{16}$ was erected, the pins driven at U_{16} and the supporting trusses bolted at both ends, thus holding the vertical post in position and allowing the guy lines to be released. The sway bracing between the main posts, excepting the top strut, was then placed and the inside link supporting the outer end of the erection bridge was disconnected to allow the bottom laterals to be completed to L_{14}.

The chord pins at U_{14} could not be driven until the chord $U_{12}U_{14}$ was erected and as the support of portion of the succeeding panel caused stresses in $U_{14}U_{16}$ a temporary connection was necessary.

Plate LXXXII. — Shows links provided for this purpose. The links rested on the top chord supporting trusses and were attached by temporary connections bolted to the tops of the posts. The panel length was adjusted for driving the pins by means of the hydraulic jacks. The links were so arranged that they could readily be moved forward from panel to panel as the work progressed.

The erection of the succeeding panels to the panel point C_4, was a repetition of the foregoing operations, but the shorter lengths and the lighter sections towards the end of the cantilever arm permitted a simplification in some cases.

Plate LXXXIII. — Shows the members erected with the traveller standing at F_{14}.

Plate LXXXIV. — Shows the members erected with the traveller standing at F_{13}.

Panel C_2C_4 was erected with the traveller standing at CF_4, the overhang being sufficient to cover the entire panel.

Plate LXXXV. — Shows the members erected with the traveller standing at 2.

Panel C_0C_2, 65 ft. long, was erected with the traveller at CF_2 in the following manner: The bottom chord L_1L_2 was connected by its pin L_2 and supported at the outer end by the erection platform. See Plate LXXVIII. The sub-vertical M_1L_1 and floorbeam F_1 were then placed and the floor, including the lateral and traction connections, completed to F_1 — all being supported by the erection platform.

The sub-member M_1U_2 was then connected by its pin at U_2 and allowed to hang vertically alongside U_2M_2, M_1L_2 (weighing 67 tons) was erected and connected at L_2 when the member M_1U_2 was swung out and connected at M_1, thus supporting M_1L_2.

U_0M_1 (weighing 46 tons) was then carried out in a nearly vertical position by the forward crane and connected by a temporary hinge on the bottom of this member to a similar temporary connection on the top of M_1L_2, when the supporting hoist was slacked off and the member allowed to revolve on the hinge until in alignment with the lower part. It was then held in position by the holding up tackles until the splice was riveted.

The top chord panel U_0U_2 was then placed and the pins driven at U_2, when the adjusting links were slacked, transferring their load to the top chord U_2U_4.

The adjusting links were shifted to U_0U_2 to spring the member $U_0M_1L_2$ until the pins at U_0 could be driven. The distance necessary to spring the member to make this connection was 1.03 inches, requiring a pull on the adjusting links of 53,000 lbs.

The pin at M_1 was then driven, the jacks on the outer end of the platform under L_1 being used to make this adjustment. The erection of the cantilever arm was completed by placing the sway bracing U_0L_2 and hanging the eye-bars U_0M_0.

The erection platform was then dismantled and shipped to the south side to be used in the erection of the south cantilever arm.

The traveller was also dismantled and the lower portion sent to Sillery to be used for erecting the suspended span as later described.

The North cantilever was completed November 15th, 1915, when the work of dismantling the traveller was commenced, but before the forward derricks were taken off they were used for placing the upper girders from which was later suspended the apparatus for raising the suspended span.

The following table shows the progress of erection of the cantilever arm:—

Traveller at Panel Point		Date		Days Required Each Move	Days Required Full Panel	Approx. tons of Steel Erected
Main Post	14–16	June	5	....	....	....
	15	"	17	12 subpanel	....	950
	14	July	13	26	38	1,993
	13	"	19	6 subpanel	....	828
	12	Aug.	11	23	29	1,732
	11	"	16	5 subpanel	....	452
	10	Sept.	2	17	22	1,450
	9	"	8	6 subpanel	....	447
	8	"	18	10	16	968
	7	"	20	2 subpanel	....	493
	6	"	30	10	12	747
	5	Oct.	4	4 subpanel	....	288
	4	"	9	5	9	538
Last Move	2	"	25	16	11	610
	0	....		....	....	636

The total dead weight of the cantilever arm was approximately 26,000,000 lbs., and of the anchor arm 24,000,000 lbs., and when the cantilever arm was completed with the traveller in its last position there was an uplift on the anchor Pier of about 720,000 lbs. As the erection of the cantilever arm proceeded the elongated holes in the tension members of the anchor arm gradually came to a bearing and, as the weight of the cantilever arm balanced the anchor arm, the load on the outside staging was released.

The first lifting movement was observed in panels 10, 8 and 6 when the traveller was standing at panel 11 of the cantilever arm just after the bottom chord section $L_{10}L_{11}$ had been put in place.

Table II shows the vertical movement of the north anchor arm, in inches, as the erection of the cantilever progressed.

			Panel	Points		
	12	10	8	6	4	2
Aug. 19.......	0	1/8	3/16	3/16	0	0
" 24.......	1/16	5/16	3/8	5/16	1/16	0
Sept. 1........	1/16	5/16	3/8	7/16	3/16	1/16
" 3.......	1/16	5/16	7/16	7/16	3/16	0
" 10.......	3/16	1/2	11/16	3/4	1/2	1/4
" 15.......	3/16	1/2	11/16	7/8	9/16	5/16
Oct. 1.......	5/16	3/4	1-1/16	1(1/8	1	7/16
" 8.......	3/8	13/16	1-5/16	1-7/16	1-5/16	11/16

When the cantilever arm was fully assembled and the traveller removed the cantilever and anchor arms practically balanced over the main pier, but the staging at AL_2 was left in to prevent any possible negative reaction on the anchor bars due to wind or moving loads. The toggles on the anchor bars were slacked off and removed and the towers at L_2 were jacked until the gauges placed on the jacks measured a reaction of about 200,000 lbs. when shims were driven under the tower bases in this position.

The vertical motion of the trusses due to the balancing of the cantilever arm was very small at AL_{12}, and it was only after the suspended span was connected that the staging under this point could easily be released. As it was not required for use elsewhere it was left in place.

The intermediate towers were taken out, shipped to the shops and cut into short columns to be used at Sillery this work being done during the winter of 1915-16, for use in the early spring.

The south anchor arm was erected during the season of 1915. The work was started May 20th and finished, including the erection of the main post, on November 8th.

When the floor system of the south anchor arm had been connected to the trusses releasing the inside staging, part of it was shipped to Sillery to be used again in the erection of the suspended span.

The erection of the south cantilever arm was started in the spring of 1916, and completed on July 31st. The apparatus for lifting the suspended span was then installed before the traveller was removed on August 29th.

(h)—*Erection of the Suspended Span at Sillery:*

Plate LXXXVI. — Shows a general plan of the Sillery site and the position of the falsework carrying the span in relation to the double track main line connecting the site with the north shore material yard about three miles away and with the car ferry to the east by which connection could also be obtained to the south shore material yard. Due to the direct connections with the storage yards and the small amount of traffic on the main line, only limited siding capacity was required.

A trestle was built to connect the falsework for carrying the span with one of the sidings laid down. The trestle was constructed with the 84 ft. plate girder spans used on the inside staging of the anchor arm and timber towers. It was floored with ordinary track ties, but having a 14 ft. tie at frequent intervals projecting to carry a foot-walk, air pipes and electric wires.

Plate LXXXVII. — Shows the staging for carrying the suspended span during erection. The foundations for this and for the connecting trestle were prepared during the season of 1915. The bottom was a practically level shale rock foundation, and most of the footings were made by simply chipping the rock to a little below the required level, the correct elevation being obtained by levelling up to grade with cement mortar. Anchor bolts were grouted into the shale which was moderately hard below the exposed surface. The site was only dry for a few hours at low tide, and, while the work was easily done, it extended over a considerable period on account of the short working time.

An office and small power house was built during the season of 1915, in which were installed one of the 250 K.W. motor generator sets used at the north end, to supply direct current to the traveller motors, and one of the air compressors to supply compressed air for riveting.

The towers at the four corners (Plate LXXXVIII) were designed as pedestals to carry the full weight of the span when swung and any lateral forces that might occur from wind or temperature. The heavy box girders forming the top of these towers and providing support for the span were those used for the purpose of supporting bent 13 under the anchor arm.

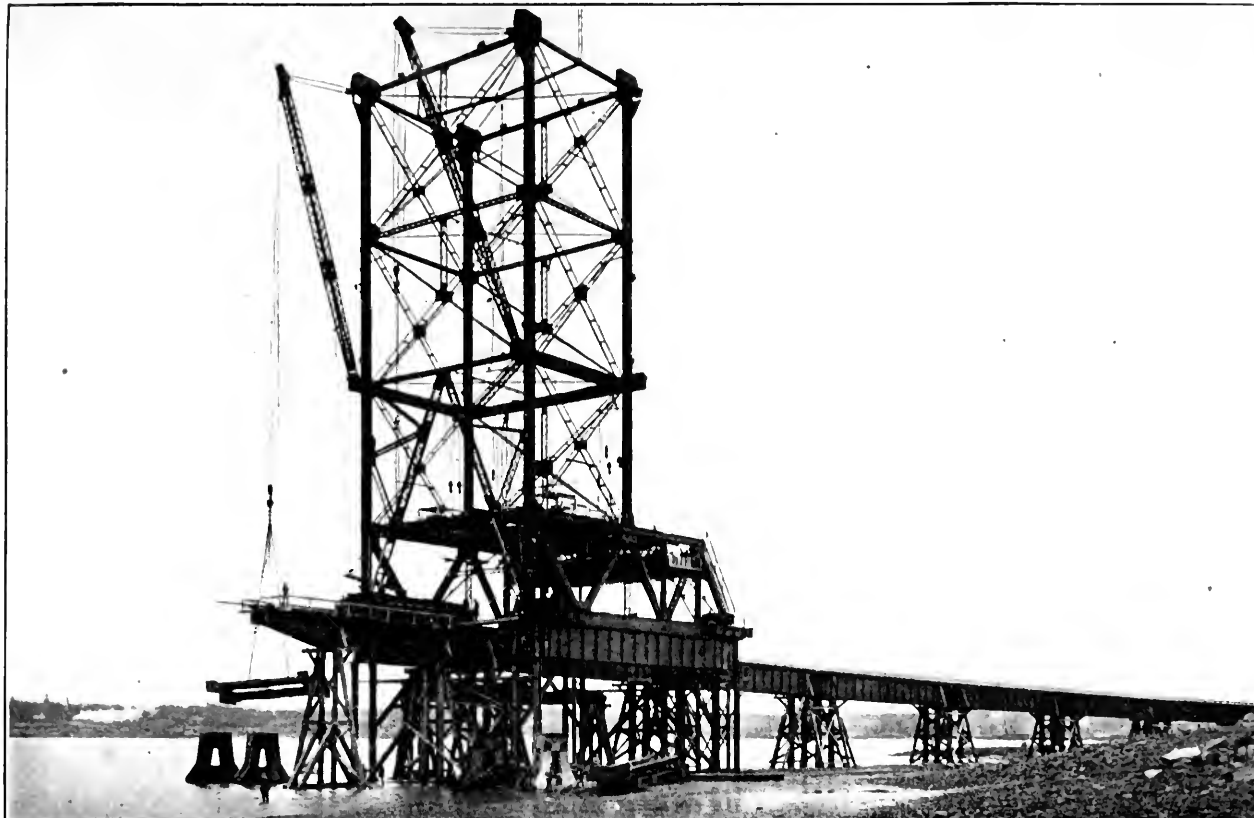


Figure 27 **Photograph showing traveller placing the first bent of the suspended span staging.**

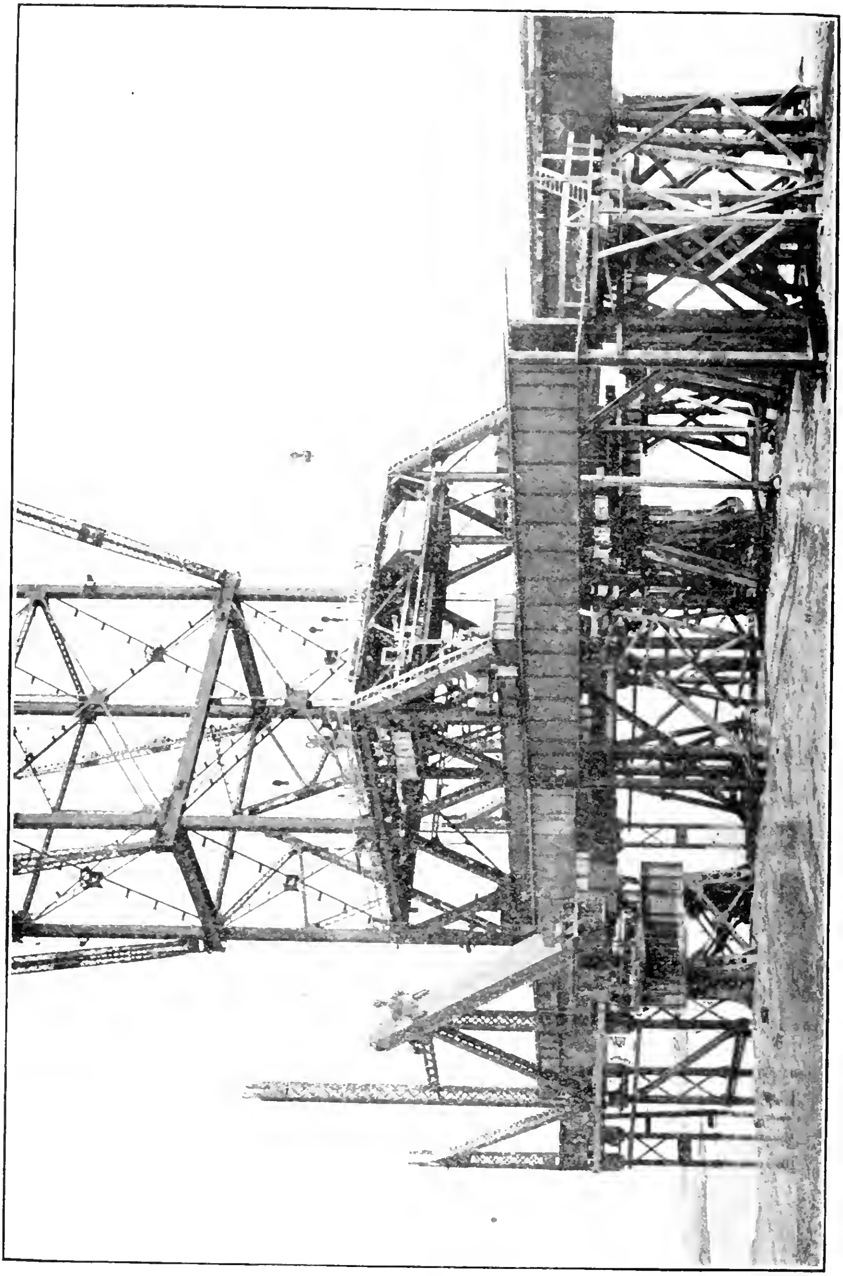


Figure 28 Photograph showing the lower half of the first three panels of the suspended span erected.

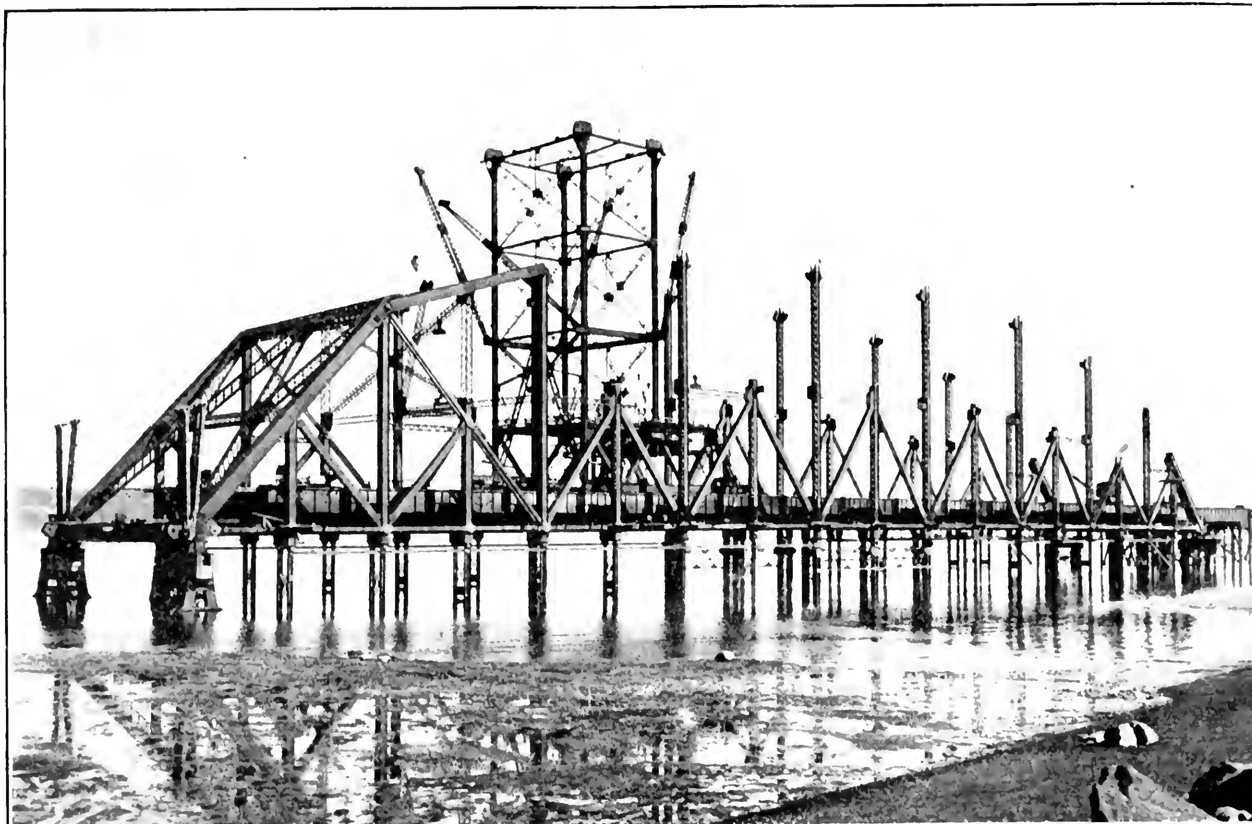


Figure 29 Photograph taken during the erection of the suspended span.

The timber blocking between the under side of the span and the top of the columns was built up in a special unit for each position and prepared in advance to facilitate placing it rapidly as required. See Plate LXXXVII.

Sand jacks for swinging the span were placed under all main panel points 2, 4, 6, 8, 10, 12, 14 and 16, those at 8 and 10, being of extra size.

The depth of the suspended span at the centre being only 110 feet and the weight of the heaviest member to be handled 66 tons, it was possible to use the derrick booms (increased to 40 tons capacity) for erecting the span and the cranes and crane-runways were not required.

The tackles on the derrick booms were increased in capacity by the addition of sheaves to the blocks but the booms themselves as originally made were of ample strength. The derrick hoists and swinging gear used when erecting the cantilevers were again used at this location.

Figure 27. — Shows the traveller assembled and setting the first bents of staging.

As it was necessary to remove the traveller from the span before floating it, the whole operation was facilitated and time saved by erecting the traveller on a special tower outside the span from which it could start to set the staging as it proceeded across the span and to which it could be returned when the erection was complete.

From its position on the temporary tower the traveller placed the corner towers at the point L_0 staging bents at 1 and 2 and the longitudinal bracing connecting them. Floorbeam FB_1 was then placed and stayed by the lateral bracing which was of a box form 3′ 9″ deep, the verticals connecting to the floorbeam were erected and the temporary floor laid, so that the traveller could be moved to bent "1." In this position it erected the bottom chords, FB_2 and the lower half of the diagonal truss members. The succeeding panels were erected in the same manner as the traveller was moved across the span, the upper half of the diagonal truss members, the top chords, the top lateral bracing and the sway bracing being left for the traveller to erect as it was returned panel by panel to the west end of the span.

Figures 28 and 29. — Show the span being erected.

The first member of the span was placed on May 22, 1916, and the last pin was driven July 21st, 1916. The riveting was begun as soon as the traveller was sufficiently advanced and closely followed the erection.

The top chords had a full bearing on the pins at U_2, U_4, U_6 and U_8 and were without hinge plates. The camber blocking was set to make the top chord panels a half inch longer than the shop lengths and the pin holes in the bottom chord bars were elongated half an inch except the four end panels which were in compression when floating. In placing the floor beams the distance between panel points was made 1/8th of an inch short of the shop length of bars, which greatly facilitated the driving of pins and allowed for any change in the length of panels due to temperature.

Plate LXXXIX. — Shows the deformation of the truss and the displacement of the panel points for the different conditions while being erected.

In swinging the span the blocking at the sub-panel points 1, 3, 5, 7 and 9 was first removed and afterwards the blocking on the inside columns of the staging at points 2, 4, 6 and 8 when the weight was carried on the sand jacks at these points. The sand was then removed from jacks at 2, 4, 6, leaving the entire weight supported at 0 and 8. During this operation the top chord joints at U_2, U_4 and U_6 closed and the opening at U_8 increased. The slack in the bottom chord between 0 and 8 was taken up causing a horizontal displacement at 0, 2, 4 and 6. This displacement was provided for in the construction of the sand jacks and in the detail at L_0. When the sand was removed from the jacks at the points 8 there was both horizontal and angular displacement at L_0. The movements are set forth on Plate LXXXIX, but the arrangements for meeting them will be referred to more fully when describing the loss of the suspended span in 1916, which was due to the failure of a portion of this joint.

After the span was supported on the four corners the four points L_8 were all brought to exactly the same level and the holes connecting L_8—M_8 were reamed and riveted thus insuring an equal distribution of the live load stresses between the diagonal members in the centre panel. All other web members had been riveted while the span rested on the false-work, but the top chord and end post splices were completely riveted when the span rested only on the four corners. The laterals and sway bracing were thoroughly bolted but not riveted before floating.

After swinging the span all but the centre bents of the falsework were removed to permit the scows to be fixed in position for floating it.

(i)—*Equipment for Floating, Mooring and Hoisting the Suspended Span:*

Plate XC. —Shows the position of the scows and the method of loading them. The span in condition for floating weighed approximately 5,300 tons. All the scows were blocked to the same elevation and were thus equally loaded under the panel points L_1, L_2, L_3 at each end. The load was distributed over a length of 25 feet at each end of the scows through a grillage made of four longitudinal floor girders to be used later as part of the floor of the suspended span which were well braced together and rested through wooden blocking on steel cross bulkheads. This grillage was in turn loaded by an eye-beam grillage made of eight track stringers on which the floor beam was carried by tightly driven steel shims. Each scow as placed was sunk on a foundation prepared for it by opening the valves in the bottom and was securely blocked in position. The scows rested while at Sillery on timbers bolted to concrete foundations placed vertically under the longitudinal trusses of the scows and accurately levelled.

The three scows at each end were connected together by four girders made of the inside staging posts, the girders and connections being calculated to meet the wave conditions shown on the diagram (Plate XCI). The cross girders were connected to the scows by means of channels fastened to the cross frames and extended up above the top of the girders to carry saddles. The girders were tightly wedged in place between these saddles and the top walling. See Plate XC. In addition to this connection the scows were well lashed together around the side bollards and diagonal wire ropes were run from corner to corner to take up any possible longitudinal shear between the scows.

Each scow was tested for tightness as it was placed by closing the valves and allowing the tide to exert the lifting effect of the full depth of the scow, a load about 50% in excess of that it would normally be called upon to carry when floating the span. The tests were made before the cross girders were connected to avoid any danger of straining these through compression of the blocking under test.

Plate XCI. — Shows a general plan of the scows, the various conditions of loading assumed, the stresses and the material. The load of the span was carried by the stringer grillage directly to the steel bulkheads which in turn transferred it to the three longitudinal trusses placed 10′ 6″ apart. These trusses were estimated to carry all the longitudinal stresses, no reliance being placed upon the planking which was regarded only as a skin to keep out the water.

With a view to making the scows saleable for commercial work after serving their purpose in erecting the bridge the framing was calculated to meet the five conditions of loading shown on the bottom of the Plate. The width of the scows was fixed at 31′ 11″, it being desired to give them the full width of the panel with only sufficient clearance to insure placing them accurately. Six 8″ valves were placed in the bottom of each scow and operated by gas pipe keys extending through the deck. These were left open while the scows were in place at Sillery to allow the tide to flow in and out without exerting any lifting effect until it was desired to float the span when the valves were closed.

The draft of the loaded scow was limited by the requirement of having the valves in the bottom sufficiently above the level of low tide to be sure that the scows would be thoroughly drained before it was necessary to close the valves for floating the span. From the tide tables it will be seen that the elevation of 83 meets this condition for four or five days at each spring tide. The shoal outside Sillery Cove over which it was necessary to float the span had a maximum elevation of 82 and from these two considerations the top of the foundations on which the scows rested was fixed at an elevation of 83.

When moving the span it was desired to float the scows about two hours before high tide to give ample time to reach the bridge site and make the connections before the reversal of the current. A draft of 8′ 2″ or a surface elevation of 91.2 met this condition. The length of scow required to give the necessary displacement on this draft also gave sufficient end stability against wind forces or possible wave action from any seas likely to be encountered—a study of such records as were available indicating that the wave length was about 40 feet with a maximum depth of 4 feet from crest to hollow. The scows were made 11′ 6″ deep, giving 3′ 4″ of freeboard under probable conditions of floating. The freeboard was sufficient to provide for the possible contingency of one scow being injured and two scows at an end having to carry all the load. The trusses were also investigated for this possible although unlikely contingency.

The steelwork for the scows was manufactured at the Lachine shops, but they were assembled and planked at a Sorel shipyard.

Before describing the floating and hoisting of the span to its final elevation the arrangements for anchoring it in position at the bridge site and the apparatus for hoisting it will be taken up. The hoisting arrangements used in 1917 only differed in a few details suggested by the experience of 1916 from those used in that year, and the description will generally be limited to the apparatus used in 1917. The anchoring arrangements were the same on both occasions.

Much consideration was given to securing the floating span in position at the bridge site. The strong and varying current, together with the cross currents which existed at certain stages of the tide, particularly near the turn, made it necessary to have ample clearance in placing the span to avoid the danger of the steelwork of the span fouling the anchoring or hoisting gear. Having placed the span in position it was necessary to hold it vertically under its permanent place in the bridge against any probable wind under varying conditions of current and elevation of surface while the hoisting chains were being attached and until the scows had floated out leaving the load suspended by the chains.

Plans for anchoring to the ground were considered both for connecting the cables to the supporting scows and for connecting the cables to independent scows anchored on either side of the span which would act as docks between which the span might be moored. The water is, however, about 200 ft. deep, the range of tide about 20 ft. and the wind surface about 11,000 sq. ft., making it impracticable to place anchors of sufficient holding power up and down stream. Neither did it seem practicable to hold the span in exact position by means of lines running to the shore on account of the great length required, the pull of the current upon the lines and the elasticity of any arrangement depending on submerged cables.

Plate XCII. — Shows the plan finally adopted, a heavy cantilever frame hung from floorbeam 1 of each cantilever arm. The frame was calculated to resist a pull from the current of 65,000 lbs. and a wind of 18 lbs. per sq. ft. acting on the span, giving a total horizontal force of 300,000. It was built of inside staging legs placed 54 feet apart centres, braced together and stiffened in the longitudinal direction by a truss. The frame was connected to the floorbeam at the upper end by hinges to allow it to be swung out of the way while the span was being guided into position. It was held in the longitudinal position desired by two heavy wire rope tackles leading to electric winches, all of which were obtained from the 60-ton hoists of the erection traveller.

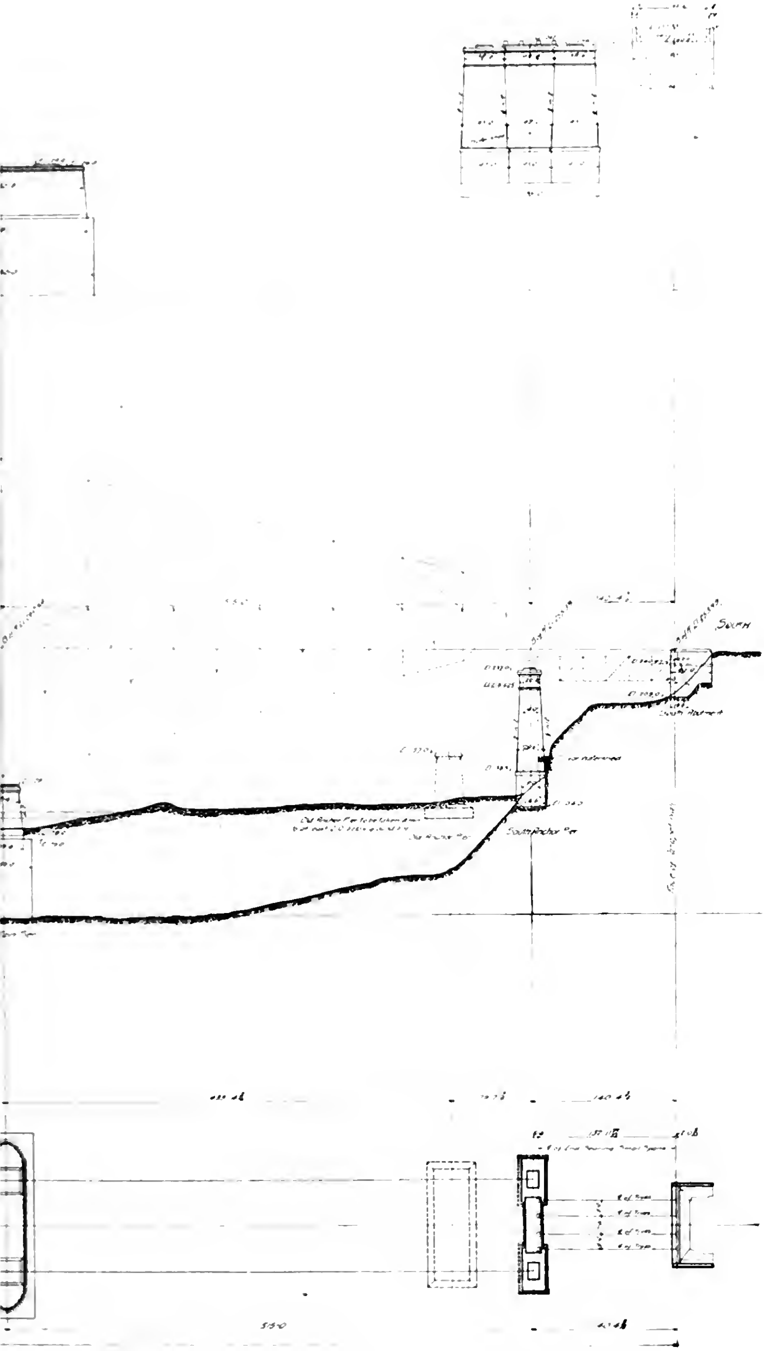
At each lower corner of the frame there were independent sheaves for two 1¼″ plough steel anchor cables hinged to revolve in a vertical plane through an angle of nearly 180 degrees. The anchor cables were attached to the span by loops slipped over steel bollards bolted to the end of the span and were led around the sheaves to eight part steel tackles, the falls of which were carried to electric winding engines previously used for the derricks on the traveller but later placed near the ends of the cantilevers. The lower blocks of the anchor tackle were fitted with guides running on a wire jack stay to prevent twisting or fouling.

A working platform was provided at the lower end for carrying lines and for general convenience when anchoring. Connections were also fitted for tackles to hold the span against horizontal forces as it was being raised.

Plate XCII. — Also shows the rigging and tackle for pulling the hoisting chains out of the way when placing the span — the dotted line indicating the curve of the chains when so drawn back.

Plate XCIII. — Shows the general arrangement of the apparatus at one of the four corners for hoisting the suspended span. The upper side elevation shows the shoe of the suspended span in its lowest position and the lower elevation the shoe raised to a point where the permanent connecting pins were driven. The permanent suspension eye-bars supporting the suspended span from the end of the cantilever arm were in two lengths, the top half being erected and hung from the end of the cantilever while the lower sections of the bars were pinned to the end post of the suspended span and raised with it, the final connection being made at the centre.

The whole hoisting apparatus was suspended by means of pins from a heavy girder carried on a rocker bearing on the centre line of the truss, thus securing adjustment in all directions for the hoisting appliances. The


South
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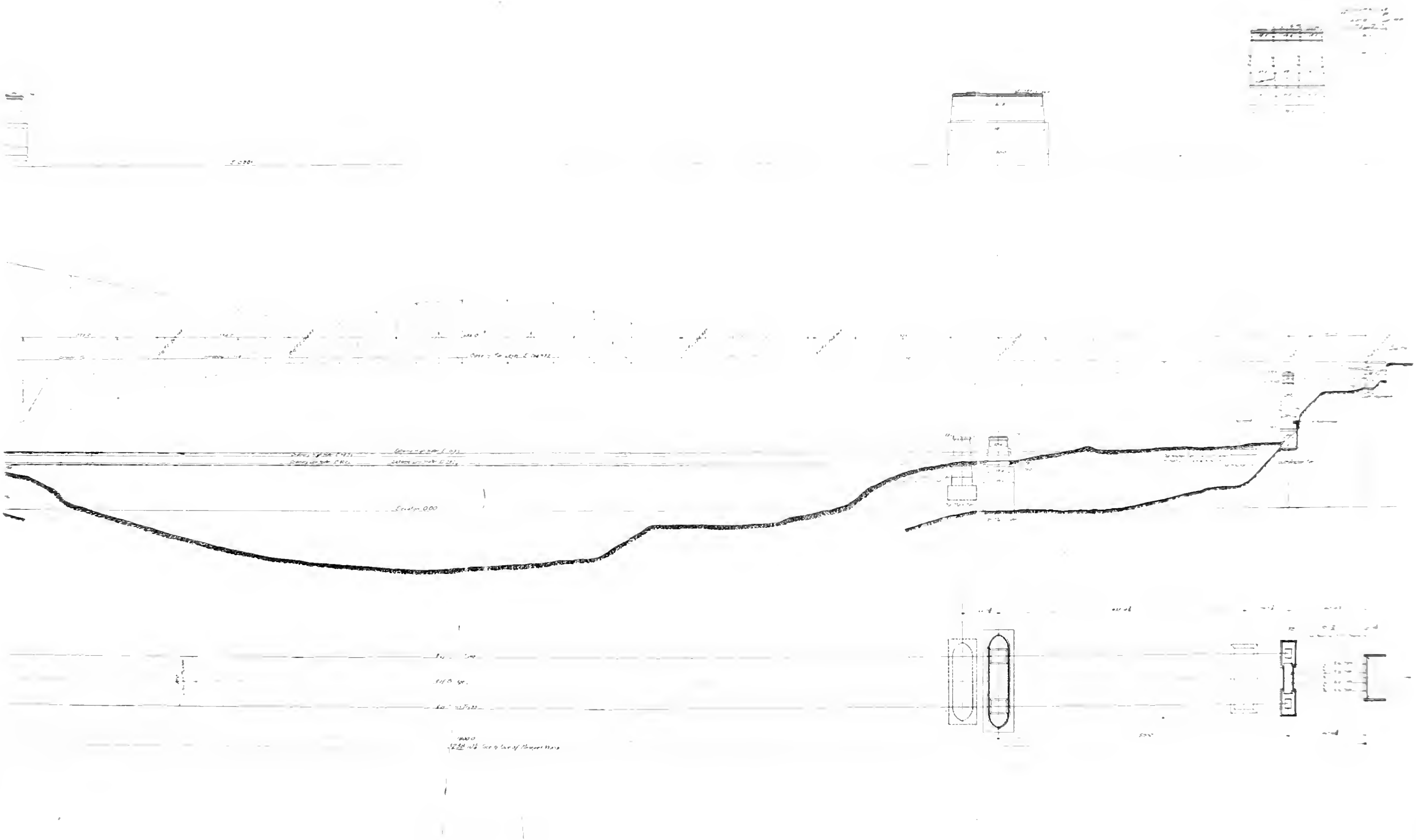


platform carrying the lifting jacks was built of two plate girders connected together by heavy diaphragms. The outer diaphragms supported the hydraulic jacks while the lifting chains passed between the next set of two which were made of sufficient strength to carry the load in the lifting chains and transfer it to the side girders. The girders were riveted to the outside of the suspension frame hung from the rocker girder on top of the cantilever. These hangers were well braced together so that the jacking platform, the suspender and the rocker girder above could swing laterally only as a unit.

The lifting girders were practically duplicates of the supporting girders with corresponding diaphragms for the hydraulic jacks to thrust against, but they were free to slide on and were guided by the suspenders carrying the jacking platform. All of the lifting was performed by the hydraulic jacks, but as a measure of safety and to permit the renewal of a packing or any portion of the hydraulic equipment which might fail in operation, four 12″ screw jacks shown on Plate XCIV were provided at each corner and during the operation of lifting, these jacks were always kept in contact with the lifting girder although exerting no pressure thereon. The jacks were supported in brackets built out opposite to the diaphragms carrying the suspension pins. They were geared together in pairs and operated by means of hand wheels from the level of the platform. The screws were counterweighted, shown on the drawing, and a counterweight was also attached to a drum on the gear shaft tending to keep the screws always in contact with the top bearing.

Each lifting chain was made of seven links of 4—28″ x 1½″ plates about 27 ft. long. The links were connected together by 12″ pins, 24′ centres, the end connecting pins being half way between the holes for the pins connecting to the jacking girders which were placed every 6 ft.

The system was arranged for a lift of 2 ft. in each jacking operation and three holes were placed in the diaphragms at 2 ft. centres so that a hole might always be brought to register with the holes of the links which were 6 ft. centres. The lower hole of the hoisting chain was elongated to 5 feet to facilitate the connection to the suspended span and the chains were made sufficiently long to connect to the span while afloat at the lowest stage of the tide. It was expected that the connection would be made on a falling tide and it was, of course, most important that no chain should take a load until all had been connected and, furthermore, that they should all be loaded simultaneously and equally.

As the pins connecting the links came clear of the upper girder, the links were taken off and laid on the floor of the bridge.

Plate XCV. — Shows the rigging of the tackles for taking off the links. It also shows the platforms from which the jacking pins were operated, a detail of the jacking pin in place and the method of supporting these pins from counterweights so that they might easily be shifted from hole to hole.

It will be noted that the suspension line was given a lead which would tend to release the pin when it was without load. A trigger was placed on the pin to prevent this action, also to automatically insure that the pin was in its proper position before transferring the load to it.

Plate XCVI. — Shows the arrangement of the hydraulic jacks and the piping. The rams of the jacks were 22 inches in diameter and the working pressure about 3,700 lbs. per sq. in., giving each jack a capacity of about 700 tons. They were tested, however, for a load 33% in excess of working capacity by pinning the lifting and jacking girders together and working the hydraulic pumps to a gauge pressure of 5,000 lbs. Rocker bearings were provided at the top and bottom of each jack to prevent any possibility of eccentric loading. The four jacks at each end were operated by two direct acting double plunger pumps operated by compressed air connected up as shown on the diagram. There was a double controller valve opposite each corner and opposite this valve a telltale showing the comparative level at each jack so that the operator stationed at this valve could regulate the flow of water to each jack and easily keep the girder practically level. A similar valve was placed at the centre of the span and opposite to it a telltale showing whether the two corners of the span were level. The operations of jacking were generally controlled by these valves, those at the corners being used merely as auxiliary to keep the jacking girders themselves level. A signal system was arranged across the span so that all corners were kept practically level at all times.

Plate XCVII. — Shows the arrangements for holding the span when afloat and for launching it from its berth to a position where the tugs could be attached to it.

Part of the floating programme was to choose a tide that would float the span as near as practicable to daylight in the morning so that there might be ample time for making the connections and lifting the span to a position where it could be safely left through the night, it being considered impracticable to carry on the work by artificial light. This necessitated closing the valves in the bottom of the scows somewhere about midnight at a time when the morning weather is difficult to foretell and it was felt that if conditions at daylight proved unfavorable for moving there should be guides and anchors to force the scows to settle back with the falling tide on the foundations provided for them in exactly the old position. A timber framework was built around the outside corner towers to act as end guides and timber shims were fitted between the scows and the framework with only sufficient clearance to allow the scows to rise and fall with the tide. A steel framework was placed at the shore end of the centre scow of each end set which prevented the span from moving shorewards and the scows were anchored to the framework by a tackle led to the hoisting engine of sufficient power to hold or to pull the span against the assumed wind force. All of this temporary framework was estimated for a wind force of 5 lbs. per sq. ft., it being considered that any greater pressure would be foretold by the Meteorological Office.

Most of the regular stringers which had been used as traveller tracks in erecting the span were taken off to form grillages on the scows, but those in the L_1L_2 Panels were left in place and a heavy timber platform was built between the two inside stringers to carry a steam hoisting engine, the engines being required for pulling the span out from its berth, for adjusting the tackles and for taking up the three-quarter inch wire ropes with which the span was first connected to the hanging frames.

Plate XCVIII. — Shows the method of guiding the span from its berth to a position in the River where there was sufficient depth of water to permit the large tugs to be attached to it. The operation was undertaken at strong flood tide, requiring an anchorage down stream to guard against the possibility of the span drifting against the Sillery Wharf, and a wire rope was run to a heavy crib anchor about 1240 feet to the eastward. An eddy often occurs at flood tide and it was thought prudent to have an additional anchor to the westward which was connected to 3″ bolts grouted into the rock. Both of these anchor cables were connected by rope lashings to the span which were cut when the span was in a position where the cables ceased to be of service. Springing tackles, led to the engines, were rigged to the cables to permit end adjustment while the span was moving out. The span was connected to the anchor bents on the shore side by 2″ rope tackles which were paid out as desired until the scows were clear of the supporting towers when the rope lashings connecting these tackles were also cut.

To keep perfect control of the span and to pull it out from its berth, tackles were rigged from the inside corner of the end scows and connected to the outside towers, the falls of these tackles being led to the hoisting engine. Marks were placed at every 5 feet along the edge of the scows and every 5 feet of movement at each end was reported by telephone to the Superintendent.

(k) *Floating and Hoisting the Suspended Span, September, 1916:*

The preparations for moving and hoisting the span were practically completed about the 1st of September and the high tides near that date might have been utilized for the work, but the success of the operation seemed so dependent upon every man being familiar with his duties and upon team play that it was decided to postpone it until September the 11th, and the intervening time was spent in practise drills, the operations being gone over as far as practicable with all men at their stations. The weather promising well on the night of the 10th the operation of closing the valves was completed by 12.45, about half an hour before the tide started to rise (see Fig. 16), and an hour and a half before it reached the bottom of the scows. At 3.30 a.m., the span was floating on the barges with a freeboard of about 3′ 4″; it being then quite dark and there being ample time to move to the site, work was postponed until 4.40 when there was sufficient light, and the hoisting engines started to pull the span out from its berth. This operation took about 7½ minutes.

It will be seen from Plate XCVIII there is a shoal at elevation 80 about 600 feet outside the span over which the large tugs could not pass, but there was sufficient draft for two small harbour tugs of about 300 H.P. each, and these tugs were attached to the span as it was moved out of its berth. They then pulled it out swinging it around the eastern anchor crib until it was past the shoal, when four tugs each of about 500 H.P. were connected on the east side of the scows two at each end. A large tug of 1,000 H.P. was also connected to the centre of the span and the two smaller tugs took up their position on the west end of the barges for the purpose of guiding the span endwise or of swinging it. These connections were completed about 5.13, when the down stream anchor line was cast loose.

Plate XCIX.—Is a chart of the course followed by the span showing the ranges established for following this course and regulating the speed of the span up the river. Two large balls were hung in the centre of the span from wires connecting the outside of the cantilever ends which gave the exact course into the opening, but the span could not make this range until it had reached a point about one mile from the bridge. Course ranges were established on shore at measured distances from which the rate of progress up the River could be ascertained and a table was made of the time at which the span should pass each range. The motion of the span was checked after range No. 14 to see that it was under full control, was allowed to reach a speed of four miles per hour between ranges 13 and 12 and checked again almost dead at range No. 8. After passing range No. 9, the tugs brought the span to a standstill in three minutes. When the span was within about 150 feet of the bridge it was held motionless by the tugs and the ¾″ lines wound on the drums of the hoisting engines were carried out by boats and connected to the frames. Then, with the assistance of the small tugs, the span was brought almost into position before the large 1¼″ lines were connected to their bollards. As these lines crossed in a rather confusing way the bollards were painted different colors and the loops on the lines were painted to correspond.

The operation of mooring the span and connecting the hoisting links took considerably less than an hour and was all completed at 7.30, by which time the current had practically ceased. At 8.15 the hoisting chains began to take a load from the falling tide and at 8.51 jacking was commenced; at 9.21 the scows drifted out during the third lift leaving the span suspended on the hoisting chains. Everything had worked as planned up to this point, it was thought all risk in the operation had been successfully overcome and that nothing remained but to carefully jack the span to its final position. The workmen had been on duty since midnight, some all night, and, after the span was raised four lifts at the north end and five lifts at the south end, they were allowed a recess of an hour for breakfast and rest, the span at this time hanging about 30 feet above the water. Work was resumed again about 10.36, one lift was completed and the upper lifting girder was being lowered for the next lift when the span slipped off its supports and fell into the river at 10.46.

The evidence that could be immediately gathered as to the manner in which the span failed was most conflicting, but, as far as could be seen from the deck of the bridge, the supporting girders on which the span had rested were uninjured and the hoisting chains remained intact. The Engineers of the Company at once proceeded to examine the supporting girders, being lowered down in a cage by the locomotive hoist to each girder in turn.

Figures 30, 31, 32 and 33. — Show the conditions of all four shoes. The photos will be better understood by a reference to Plate C showing the construction of the shce supporting the span on the girders.

On both of the north girders it was quite apparent, as will be seen from the photographs, that the castings had simply slid off the lower supporting pins, the tap bolts in the keeper plates being sheared off in a horizontal direction and all scores and abrasions showing the same movement. At the south east corner, the lower pin had disappeared altogether while the centre pin had been revolved through an angle of 90 degrees and fallen into the seat vacated by the lower pin. The western hanger chain was badly bent to the west, indicating that the span had fallen heavily against it in a westerly or transverse direction, but the manner of failure was not evident until the southwest corner was reached where the initial cause of failure became apparent. Here it could be plainly seen from the crushing of the keeper plates and the shearing lines on their bolts that the first fall had been vertical. The marks seemed to indicate that the span had struck first on the north side of the girder kicking it back from under the span thus allowing this corner to fall into the river.

A photograph, Fig. 34, taken at the moment of failure and published a few days later, confirmed the opinions formed at the time of examination. It is quite apparent from the photograph that the southwest corner fell first and that the end floor beam revolved into almost a vertical position twisting the southeast corner off the supporting girder and that the two north corners were pulled into the river after the south end of the bridge had gone down. The torsion naturally destroyed the lateral and sway bracing which caused the collapse of the top chords.

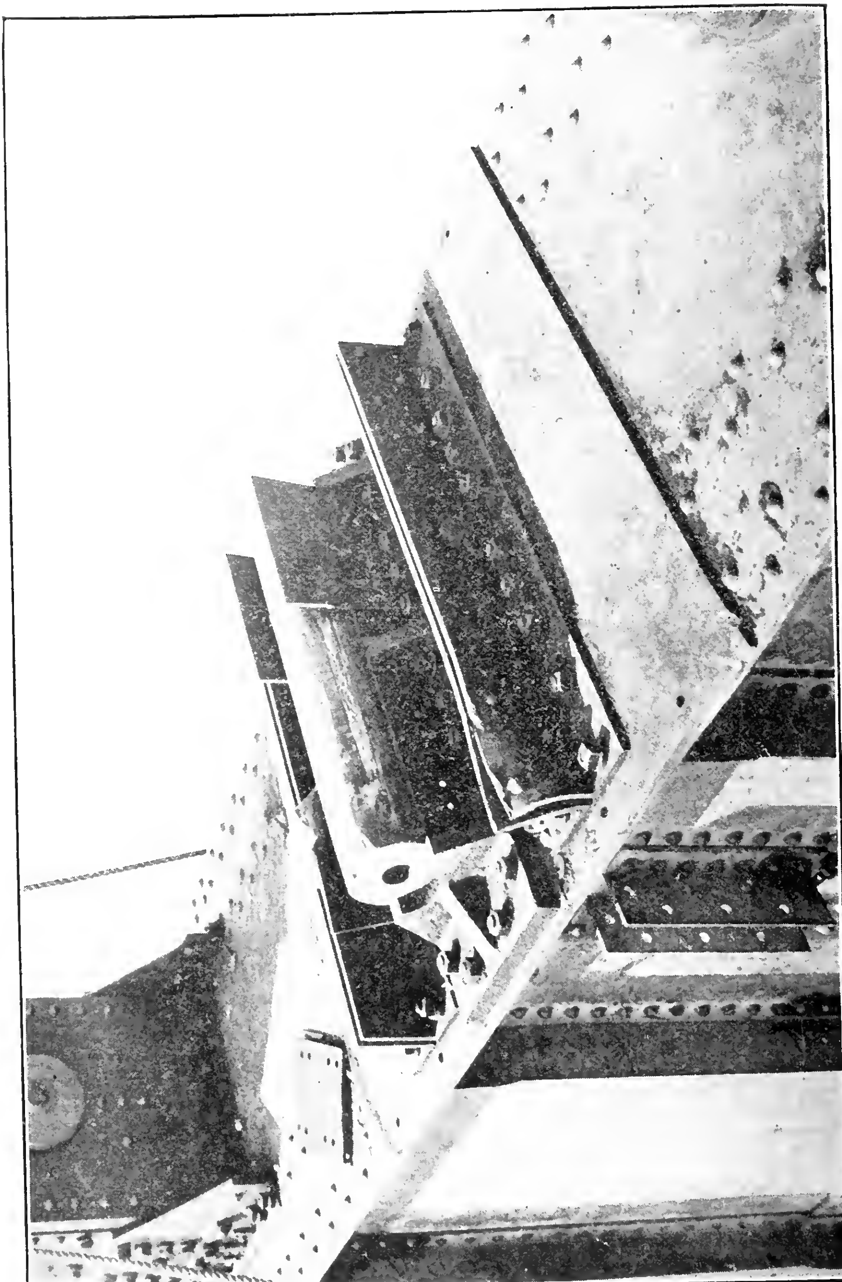


Figure 30 **Photograph showing the girder at the N.W. corner after the accident.**

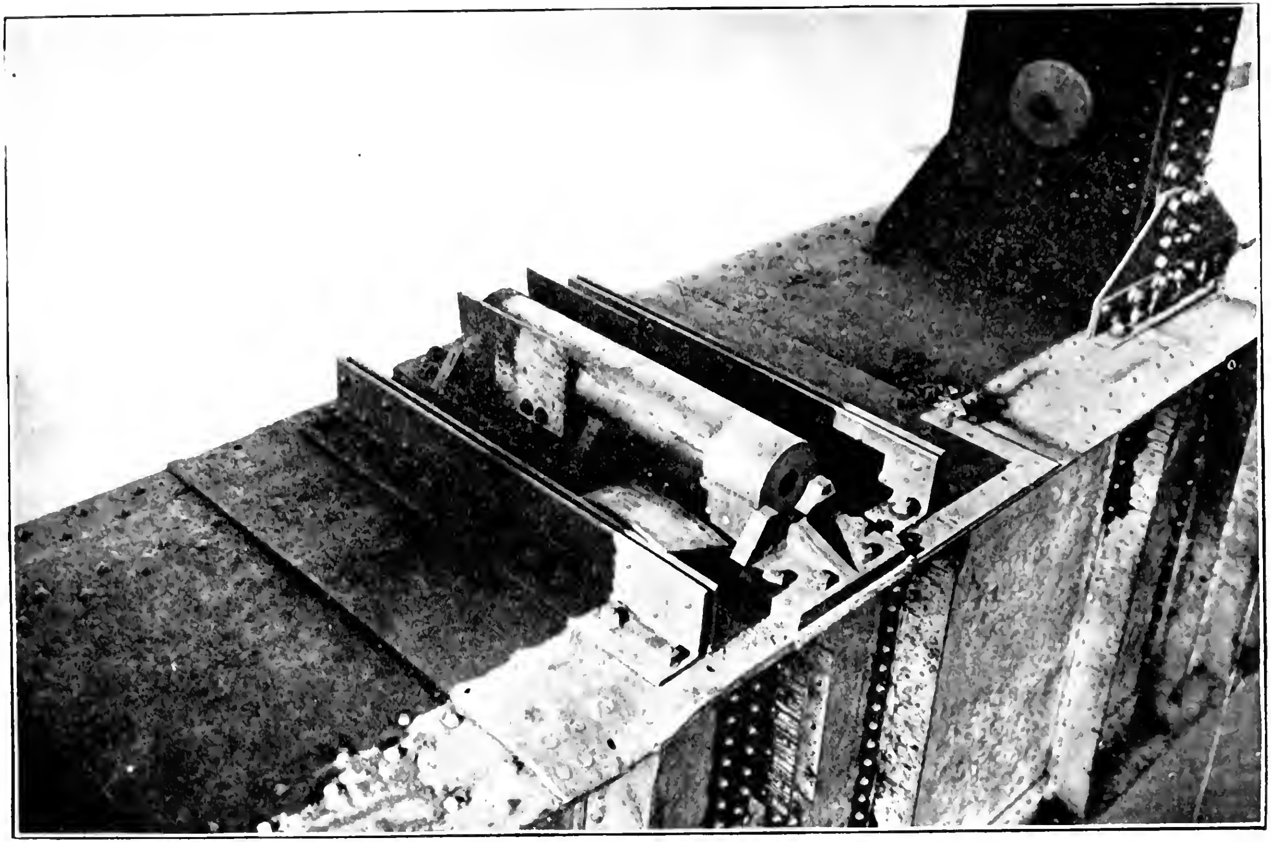


Figure 31 Photograph showing the girder at the N.E. corner after the accident.

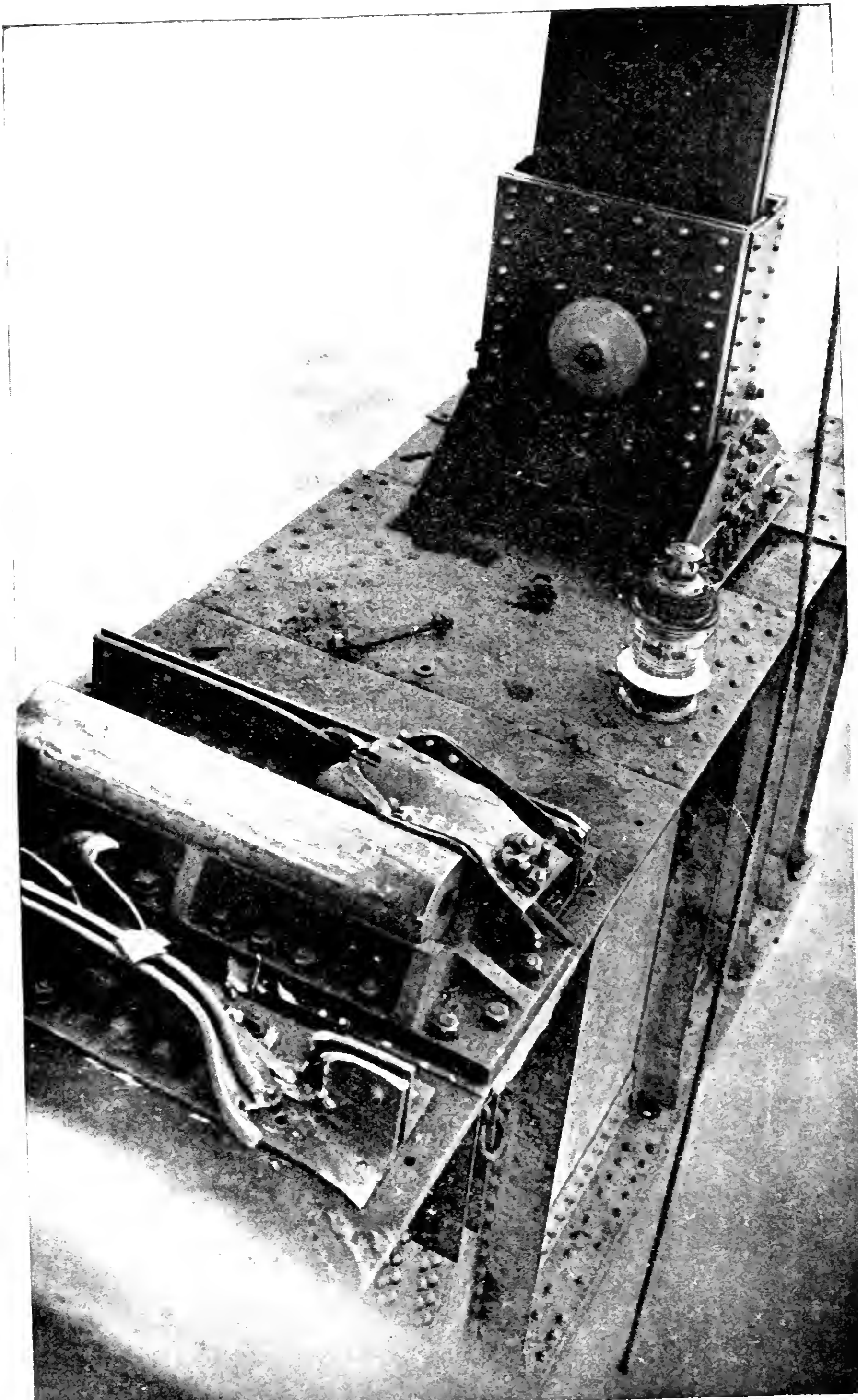


Figure 32 Photograph showing the girder at the S.W. corner after the accident.

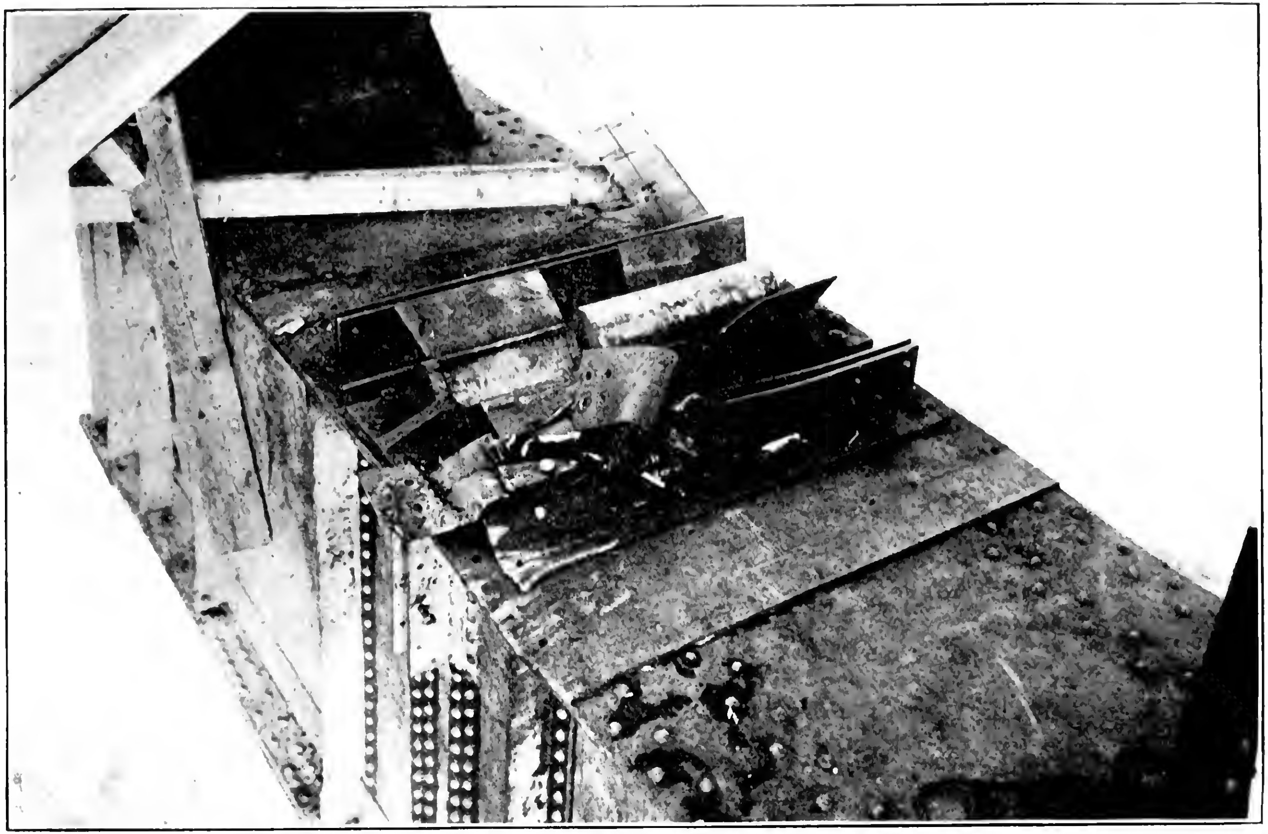


Figure 33 **Photograph showing the girder at the S.E. corner after the accident.**

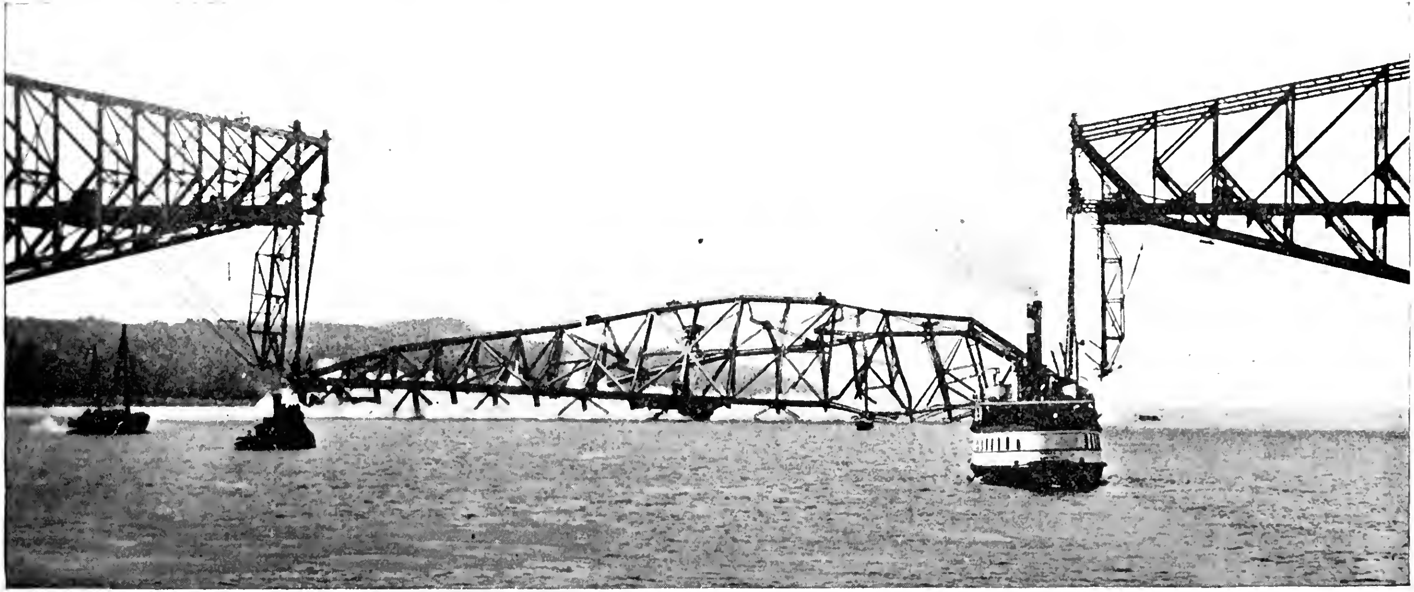


Figure 34 Photograph showing the suspended span falling.

The impact of the span falling at the south west corner, the partial transfer of the load at this and the diagonally opposite corner by the action of the lateral bracing to an axis from the south east to the northwest corners; the side fall at the southeast corner, the sudden release of the extra load first at the southeast and afterwards at the north corners and the parting of the $1\frac{1}{4}''$ anchor cables all combined to cause very heavy vibrations, both vertical and horizontal, in the ends of the cantilevers, these being so severe as to throw some of those on the cantilever ends off their feet. It was feared that the impacts and vibrations might have overstressed some of the material or the joints, and a very rigid inspection was at once undertaken. Every joint was inspected, rivets were carefully tested wherever excessive strains would be likely to occur and new lines and levels were run over the span. With the exception of the diaphragms under the supporting girders on the top of the cantilevers on the south east and northwest corners where a slight slip in the rivets was noticed, no indication of any injury could be found and the new levels and alignment corresponded exactly with those that had been recorded before the span was attached. All felt quite satisfied after this inspection that it was only necessary to build and place a new suspended span.

The decision to duplicate the span and to use the same methods of erecting it was made on the day of the failure. It was thought at the time that all of the hoisting rigging would be available with the exception of a few pieces known to be injured, such as the lower links of the chain and the upper suspension frame, but it was decided to take it down and to carefully measure and inspect each individual piece; also to test the links while taking them off by pinning the hoisting and jacking girders together and putting the full pressure of the pumps upon them, this being about 50% in excess of the stress under the actual load. When the links were measured in the yard it was found that a number had been stretched beyond the elastic limit and it was decided to test some of these links to destruction as well as some of those that had not received any apparent injury. While the tests did not indicate any danger in using the links again, even those that had been stretched beyond the elastic limit, there were difficulties in rematching them and there had been so much criticism of the Company for its decision to again adopt a method which, in the opinion of many, was the cause of the failure, that it was decided to order entirely new hoisting chains, to re-build all of the suspension gear and jacking girders and to make some experiments to determine the amount of yielding at the pin bearing in wide and comparatively thin plates.

Plate CI.—Shows the loads and stresses estimated. The experiments indicated somewhat greater strength for thicker and narrower material than had before been used and $28 \times 1\frac{1}{2}''$ links were substituted for the $30 \times 1\frac{1}{8}''$ previously used. Sway bracing was introduced in the top suspension frame and steel castings were omitted wherever practicable, partly as a matter of sentiment—a steel casting having been the cause of the first failure. The steel cast shoes and pins under the upper rocking girders were replaced by a steel rocker bar bearing on heavy rolled steel plates at both top and bottom. The greatest change was, however, in the method of supporting the span on the lower girder.

Plate LXXXVIII. — Shows the new bearing adopted.

It will be seen that during the process of transferring the load from the staging to the supporting girders at Sillery, it was necessary to provide for a horizontal movement of two inches at both ends and for an angular displacement of about 5/16 of an inch in the length of the bearing. In the first shoe the angular displacement had been provided for by the transverse pin and the longitudinal movement had been secured by allowing the lower casting to slide on the bottom rocker pin. The expansion and contraction of the span while resting on the end supports at Sillery was also provided for in the same way, but when the bridge was being hoisted it was necessary to have the transverse pin over the centre of the girder in the same vertical plane as the pins connecting the chains. As the span floated it picked up the girders on which it had been supported through the temporary links shown on the drawing marked " C." The pins of these links were carefully placed in the vertical plane of the transverse pin and as the girder was lifted it automatically swung into its correct position. Keeper plates marked " D," which had been previously fitted in the shop were then tap bolted in place to keep the girders properly centered as they were being suspended by the hoisting chains and the load of the span was again being transferred to them. Several theories were advanced for the cause of failure, but the most generally accepted was that the cruciform casting "A" had split under the upper pin on the shore side throwing all the weight on the river side and causing the girder to tip. Experiments were made later with duplicate castings poured in the same manner as the casting which failed, which when sectioned showed gas pockets and spongy material in the centre and the authors have no doubt that the initial failure was in this casting.

(1)—*Hoisting and Coupling in Place of the Suspended Span in 1917:*

Plate LXXXVIII. — Shows the method of supporting the span at Sillery and when being hoisted. The load of the span while being hoisted was carried by the nickel steel rocker bar shown in the sections which was planed to a top radius of 25 inches, but while the span rested on the end supports at Sillery, it was carried by shoes under the outer ribs of the end post. These shoes which were carefully planed and polished slid on bronze plates and allowed ample horizontal movement for temperature changes as well as for the operation of transferring the load from the staging to the corner supports. The supporting girders were suspended by links when the span floated and until it was carried by the hoisting chains at the bridge. As soon as the span floated and the load was relieved off the side shoes the bronze plates and shims were taken out. The keeper blocks holding these were reversed, having been previously fitted in the reverse position to take up the 3 inch clearance between the ends of the shoe and the ends of the shims. A key was also inserted at the end of the centre rocker bearing, as shown on section C-C. The supporting girder was thus firmly held centrally under the rocker bearing and perpendicular to the centre line of the truss.

The angular movement which took place when the load was transferred from the staging to the end supports was provided for by means of lead shims placed under the outside shoe bearing. Experiments made for the purpose showed that the lead would flow under a pressure of about 5,000 lbs. per sq. in. and the bearing surfaces were proportioned for this pressure. The bearing of the centre rocker, which was nickel steel, was also determined by experiment on a short section and was made 50,000 lbs. per lineal inch.

The new construction shown on Plate LXXXVIII required that the full weight of the centre span should be carried by the centre ribs when the span was being hoisted into position, while the outside ribs carried this weight when the span was resting at Sillery. It was therefore necessary to add additional diaphragms and tie plates in the shoe and end post to transfer these loads. It was also necessary to add a small amount of reinforcement to the centre rib at the bearing, but otherwise the new span was an exact duplicate of the span lost.

A change was made in the method of attaching the hoisting chains to the supporting girder to save lowering the links through the end strut as in the previous case, short links being introduced which brought the connecting pin well above all interference and made it only necessary to swing the ends of the hoisting chain into position between these links.

Erection of the New Span:

It was known that the cove at Sillery was subject to heavy ice shoves in the Spring and that during the Winter the rise and fall of tide in extremely cold weather would impose severe conditions on the falsework and connecting trestle remaining at the cove. All the bents of the trestle were cribbed in and filled with loose stone but, in spite of the precautions, the ice upset the heavy steel corner towers which had carried the span and very nearly wrecked the tower on which the traveller stood. Repairs were, however, completed in time to start erection on May 20th, and the erection proceeded as in the previous year, being completed on July 20th. Concurrently with the erection of the span the new hoisting gear was put in place and arrangements made to take advantage of the spring tides running from the 13th to the 20th September. The towing fleet was assembled and on the night of the 14th the crews were called out and actually started to close the valves about 11.30, but bad weather and unfavorable reports from the Meteorological Office caused a postponement until Sunday night, the 16th. The range and time of the tide on Sunday night happened to correspond very closely with those of 1916. The span was floated off its end bearings about 5.15, and was moved out from the berth at 5.47. At 6.15 the tugs were attached and the anchor cable cut, when the move up the River started. After the experience of 1916 the trip up the River was made with much more confidence, without any of the checking of the previous year, and the span was in position for connecting the anchor cables at 7.30. At 8.05 the lifting chains were let down, but a little difficulty was experienced in entering some of these and it was nearly nine o'clock before everything was connected, inspected and ready to begin the operation of jacking, which started at 9.10. The tide was falling and after three lifts, or 6 feet of rise, the scows came free at 10.28 and floated out. As soon as the span came clear the men were allowed half an hour for breakfast, and during this time the Engineers inspected the bearings at all four corners and found that all connections had come to place as intended.

To provide lateral anchorage in case of need during the hoisting of the span, two of the tackles from the top of the anchor lines were connected between diagonally opposite corners of the anchor frame and the scow. These tackles were shifted up the anchor frame as the span was raised.

Nine lifts were made on the first day in cycles varying from 13 to 19 minutes. Jacking operations stopped at 4.40, when the mooring lines were set tight and the span left for the night. On the morning of the 18th, the first operation was to take off and lower the upper set of links which had come through the top jacking girders. This required about three-quarters of an hour and the actual jacking only started at 8.16; it was continued steadily until 12.02, when the 26th lift had been taken. In the afternoon the first operation was to take off another set of links, in which a little difficulty was encountered, and it was not until 3.30 that jacking was resumed. Seven lifts were made up to 5.31 and work stopped, when the span had been raised thirty four lifts or 68 feet. On the 19th twenty-six lifts were taken, giving a total rise at the end of the day of 120 feet and bringing the span within 30 feet of its final position. The weather had been perfect for the operations up to this time, but just as work closed at the end of the third day a thunder squall broke from the north and the weather was very unsettled throughout the night. On Thursday morning it was blowing from 30 to 35 miles per hour from the northeast, with strong gusts which probably exceeded the recorded velocity. There was some nervousness amongst the men and hesitation about casting the moorings free to allow jacking to be resumed, but after carefully easing off the tackles it was found that the span swung off centre from 5 to 7 inches without appreciable side sway although there was longitudinal oscillation of about one-half inch within a period of 10 to 12 seconds. Jacking was started at 9.05. After the noon recess only eight feet of lift remained and soon the upstanding permanent hanging bars came through the jacking girders. After the first lift a short stop was made to inspect clearances and after the next, the 73rd lift, there was a delay of 30 minutes for taking down the free top links of the hoisting chains and some spacing angles which would interfere with driving the pins. The 74th lift was taken very slowly as some of the wooden working platforms had to be taken down, the clearances inspected and the lower eyebars guided into the spaces between the eyebars hanging from the cantilever. It was completed at 3.10 and was followed immediately by the 75th lift, during which the pin holes in the top and bottom bars were registered. At 3.25 the first pin was driven. The clearances were perfect, the heavy pins requiring only a few taps of a light rail to drive them home and the last pin was in place at four o'clock. After this the jacks were lowered to let the load come on the permanent hangers and to let the lifting girder bearings come free. The following day, Friday the 21st, the permanent connections were made between the lateral systems of the suspended span and the cantilevers and all temporary connections were released.

Plate CII. — Gives a record of the lifting operation.

(m)—*Final Operations to Complete:*

When the span was thus made safe, after the continuous work and strain of the past ten days, the men generally laid off for about a week, but the floor system across the span was laid and a train was crossed on Oct. 17th. Locomotive derrick cars and shunting engines had, of course, crossed before this time, but the regular train was not allowed over until the riveting of the floor system had been completed. Work proceeded steadily with the balance of the steel and the bridge was handed over to the Government, opened for traffic on December 3rd, 1917. At this time there still remained the concreting of the sidewalk over the suspended span, some concrete to be placed in the anchor pits and a considerable amount of painting, but none of this work could be carried on during the winter months and it was not until August 19th that all was completed and ready for the final inspection of the Government Engineers.

(n)—*Erection Details:*

It may be interesting to call attention to Plate LVIII showing some of the special erection appliances and connections designed for the work.

(o)—*Accuracy of Work:*

Reference should be made to the extreme accuracy of workmanship which contributed so largely to the success of the erection and which inspires the confidence that every member of the Bridge is performing the work for which it was calculated. All compression joints throughout the bridge came to a true bearing over their entire surface before they were riveted up; there were no loose tension members and as far as could be judged by the ordinary methods of inspection all tension members were equally stressed. As an instance of the precision of the shop work, of the field measurements and of the field operations, the starting points of the cantilevers were the large shoes on the piers, giving a base centre to centre of

88 feet. When the cantilevers were erected and extended out 580 feet the alignment of the cantilevers from both sides of the river was perfect, it being impossible to detect with a transit any variation whatever. The ends of the cantilevers on both sides of the river were exactly the same elevation and the deflection when the weight of the centre span was hung upon them was the same at all four corners. Another illustration of the accuracy of the shop work is to be found in the large vertical posts over the piers; each of these posts was composed of four separate columns riveted together in the field after erection. The total length of the post was 310 feet, made up of seven sections 36 ft. to 52 ft. long. At each one of the posts when erected the tops of the columns were absolutely level and the tops of all the four posts were at the same elevation so far as this could be checked.

APPENDIX "A"

SUMMARY OF TESTS — QUEBEC BRIDGE

One of the important conclusions arrived at by the Royal Commission appointed to investigate the cause of the collapse on Aug. 29th, 1907, of the Quebec Bridge in course of construction by the Phoenix Bridge Company, was that the knowledge of the behavior of the very large compression members required in a structure of this magnitude was not sufficient to enable their design to be made with that degree of confidence essential to such an undertaking.

There had been many theories advanced and supported by an imposing array of mathematical calculations, but these, did not in themselves give sufficient data on which to design such large members.

This fact was realized very soon after the failure of the Phoenix design and tests were first made on a model of the chord "A9L," (the failure of which was believed to have caused the collapse of the structure) by the Phoenix Bridge Company under the supervision of Prof. Burr. This test showed a low ultimate unit stress and indicated that the lacing material used was not sufficient to develop the full strength of the web material.

This test was followed by one made for the Royal Commission on a model built to represent two webs of the chord A9L, but having the strength of the lacing system increased about 50%. As had been anticipated from the manner of failure of the first model tested this member carried a much higher unit load and failed in the web instead of in the lacing system as was the case in the other test.

The Board of Engineers appointed to prepare a design for the new structure took full advantage of the knowledge gained from the collapse of the previous structure and of the subsequent tests when preparing their design, the chords of which had a cross sectional area nearly 2½ times greater than the previous design, due to the increased loading and to the lower unit stresses specified. After having tentatively decided on the details of the compression members they had several models of them made and tested to destruction by the Phoenix Bridge Company. These models developed a very high ultimate strength and proved that the proposed lacing system was adequate to develop the full strength of the section.

After the contract to build the new bridge had been awarded to the St. Lawrence Bridge Company and the detailing of the members well advanced, it was decided to make further tests on some of the more important compression members as they differed in many important features from the members previously tested. These tests all showed remarkably good results and gave a feeling of confidence in the ability of the members to carry the loads which would be imposed on them with a reasonable margin of safety.

There were also several tests made on tension members.

It was at first proposed to reduce the allowable unit stress in the horizontal and inclined tension members by an amount equal to the unit bending stress due to the weight of the member itself but in view of the results of several tests on built-up tension members and on eye-bars this was not done as the tests proved that the bending stresses did not appreciably effect the ultimate strength of the members.

Oblong pin holes were used in many of the eye-bars and built-up tension members and the ease with which the structure was erected and the pins driven is due very largely to their use. Before finally deciding to use these elongated holes some careful tests were made which showed that they had no important effect on the strength of the members.

Several other tests were made of minor importance but all of value in solving the problems which presented themselves as the details of the design and of the erection were being worked out.

A summary of the more important tests made, and of which a fuller description follows, is given below:—

LIST OF TESTS DESCRIBED.

1.—Test of Model of Chord A_9L.
2.—Test of Model of Two Webs of the Chord A_9L Made for the Royal Commission.
3.—Tests of 16 Nickel Steel Models of the Compression Members of the Board of Engineers' Design for the Quebec Bridge.
4.—Tests on 6 Eye-bars, 28 Tension Members and 12 Compression Members.
5.—Tests of 6 Carbon Steel Eye-bars with Oblong Pin holes.
6.—Pin Friction Tests.
7.—Tests of 8 Tension and 10 Compression Models of Members Used in the Quebec Bridge as Built.
8.—Tests on Lifting Hitch for Compression Members.
9.—Test of Plates with Pin holes without Reinforcement.
10.—Tests on 2—28" x 2" and 2—26" x 1½" Plates to Ascertain Their Ultimate Strength.
11.—Tests on Links Removed from the Hangers used in Raising the Suspended Span in 1916.
12.—Tests on Plates in Tension to Investigate the Effect of Inclining the Plate to the Axis of Load.
13.—Tests on Lead in Bearing.
14.—Test on Steel Rocker Bearing.

1.—TEST OF MODEL OF CHORD A9L

After the collapse of the bridge the Phoenix Bridge Co., on its own initiative, built and tested a model chord having, as far as possible, the same relative dimensions as the No. 9 chords of the Quebec Bridge. The test was made on November 21st and 22nd, 1907, and was under the general direction of Professor W. H. Burr, who reported on the result of this test, in part, as follows:—

A model chord section was built to a linear scale of one-third of the lower chord section 9 of the anchor arm truss of the Quebec Bridge and was tested to destruction at the shops of the Phoenix Bridge Company.

The chord section No. 9 was built of four ribs 54″ deep with 4″ x 3″ x 3/8″ double angle latticing. Its area of cross section was 780 sq. inches.

All the linear dimensions of the model were exactly one-third of those of the full sized chord section, making the area of the cross section (86.526 sq. inches) one-ninth of that of the full sized member. Each of the two interior ribs was composed of 1 - 18″ x 5/16″ plate, 1 - 18″ x 3/4″ plate, 2 - 15-5/16″ x 5/16″ side plates and 2 - 2-11/16″ x 1 1/4″ x 5/16″ angles; and the two exterior ribs were each composed of 1 - 18″ x 5/16″ plate, 2 - 18″ x 1/4″ plates, 1 - 12 5/8″ x 5/16″ side plate and 2 - 11/16″ x 2″ x 5/16″ angles. The latticing was a double oblique system of 1 - 11/32″ x 1 x 1/8″ angles, with 1 - 3/16″ x 1″ x 1/8″ crossing angles at the pane, points of the former at right angles to the axis of the member. All of these lattice angles had two - 7/24″ rivets at the ends of each with the single rivet at each crossing of the interior flange angles.

All the metal used for the main parts of the model column was medium steel, but soft steel was used for the rivets. Tensile tests were made on both plates and angles and the usual effect of rolling thin metal was apparent in the high elastic limit.

The column was accurately placed in the machine with four fine wires stretched throughout its length in the general plane of the upper flanges and with two similarly placed in relation to the lower flanges. Longitudinal timber scantlings on the two center lines of the exterior ribs, carrying steel scales at their ends, were used to measure the shortening of the column under loading for 16 ft. of its length to 1/128 of an inch.

Progressive loading was applied in stages of 3000 lbs. per sq. inch of cross section of column, beginning with an initial load of that value. At the end of every other stage of each loading, the column was relieved of stress in order to make observations in that condition. This program was adhered to up to a stress of 21,000 lbs. per sq. inch, when the next increment was made 1,500 lbs. per sq. inch after which the column was freed of load.

After the application of each 3,000 lbs., or finally 1,500 lbs., increment of loading and upon each removal of loading an accurate series of measurements for shortening of the column and for horizontal and vertical deflection were made.

Under a stress of 12,000 lbs. per sq. inch one rivet in one of the lattice angle was found loose, but toward the end of the test it appeared to become less so. Up to the final loading all other rivets appeared to remain in good condition.

After having attained a load of 25,000 lbs. per sq. inch, instructions were given to load the column to 25,500 lbs. per sq. inch, but inadvertence in signalling to attendants at the pump caused the load to reach 26,850 lbs. per sq. inch, at which stress the member suddenly failed. This failure was attended by a quick sharp report, and was so sudden that all failures of details appeared to be absolutely simultaneous.

Aside from the raising of scale on the pin plates immediately in front of the 12″ pins, the collapse of the column consisted in the failure by shearing of the majority of the lattice rivets at the central panel of latticing and of a considerable number of other rivets throughout the length of the column in both flanges. There were no permanent strains or distortions of any kind discovered or apparently discoverable up to the loading producing failure. All the circumstances of the test indicated that no main part of the column was stressed up to its elastic limit; in other words, that the entire loading was insufficient to develop more than a part of the elastic resistance of the column as a whole, and that if the latticing details had been stronger the column would have carried a greater load before collapsing.

It should be carefully observed that as the column lay in the testing machine the ratio of its length divided by the horizontal radius of gyration was 35, while the ratio of the same length over the vertical radius was 42. The column failed, therefore, in the plane of the greatest radius of gyration. Furthermore its failure was wholly in a horizontal plane, there being no sensible vertical deflection of the failed column.

2.—TEST OF MODEL OF TWO WEBS OF THE CHORD A_9L MADE FOR THE ROYAL COMMISSION

Extract from the report of the Commission.

In December the Commission ordered the construction of test chord No. 2 for the purpose of determining the strength of the web of the design used in the Quebec Bridge. This chord had a section half that of the test chord No. 1; the number of rivets in the lattice connection was doubled, the section of the lattice bars was increased 50% and the weak parts at their centres were strengthened by the use of a connecting plate. The webs were of the same section as the outer web of test chord No. 1. Material from the same heats was used in the manufacture of the two test chords.

This chord was tested at Phoenixville on Jan. 18th, 1908, and failed under a stress of 37,000 lbs. per sq. inch by buckling in the webs in the center bay, the latticing being sufficiently strong to fully develop the strength of the webs. The nominal strength of the column was slightly less than the elastic limit of the metal in the webs.

3.—TESTS OF 16 NICKEL STEEL MODELS OF THE COMPRESSION MEMBERS OF THE BOARD OF ENGINEERS' DESIGN FOR THE QUEBEC BRIDGE

A number of tests to determine the ultimate strength of models of some of the typical nickel steel compression members of the cantilever arm of the 1758 ft. span Quebec Bridge as designed by the Board of Engineers were made at the testing plant of the Phoenix Iron Co., at Phoenixville, Pa., during May, June and July, 1910.

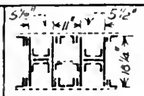
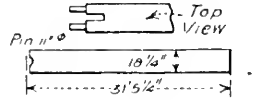
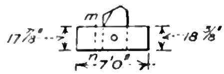
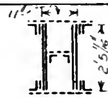
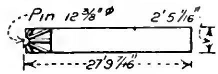
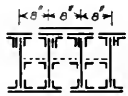
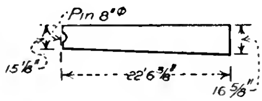
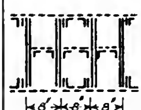
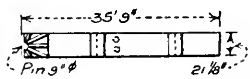
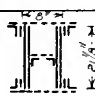
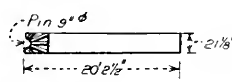
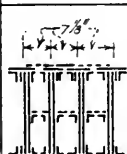
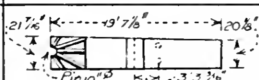
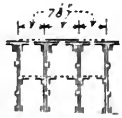
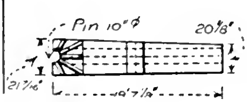
Eight pairs of models were made of the same material and of the exact shape and proportion of the bridge members as designed. The material was nickel steel containing as a minimum 3.5% of nickel, with the exception of the rivets which were of soft carbon steel.

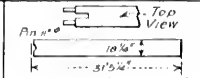
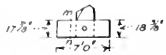
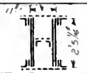
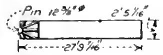
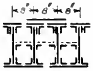
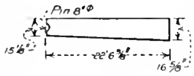
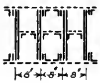
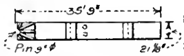
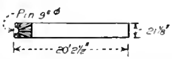
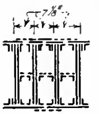
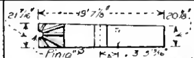
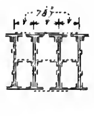
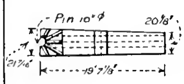
All the members were tested in a horizontal position. With the exception of T_6, all the columns had a flat bearing at one end and a pin bearing at the other. The pin bearing was so arranged as to practically eliminate bending in the pin. The flat end of the specimen abutted against the planed surface of a heavy casting through which the pressure of the ram was applied. No attempt was made to counterbalance the effect of the weight of the column. To measure deflections fine wires were stretched in the general plane of the top flange. To measure the longitudinal compressions four compressometers of the Johnson Type were used, one attached to each side of the test piece. The loads applied were measured by a Shaw mercury gauge.

The make-up of the members and the results of the tests are shown in Table 1 which is given here by courtesy of the Engineering News-Record, and is taken from an article which appeared in the Engineering Record of November 19th, 1910, describing this series of tests. The location in the bridge truss, of the members represented by the test model, is shown in Fig. No. 1.

The Board of Engineers were represented by Professor H. M. Mackay, of McGill University, who reported on the results of these tests, in part, as follows:—

T_1A—The full capacity of the testing machine, 50,134 lbs. per square inch, was applied for ten minutes without failure. Then 2 - $2\frac{5}{8}''$ holes were bored through the web of each of the four ribs, reducing the section to 46,335 sq. inches. It failed, with very little general distortion, by buckling of the webs at the section where the holes were bored at a load of 59,894 lbs. per sq. inch of net section or 49,730 lbs. per sq. inch of gross section.

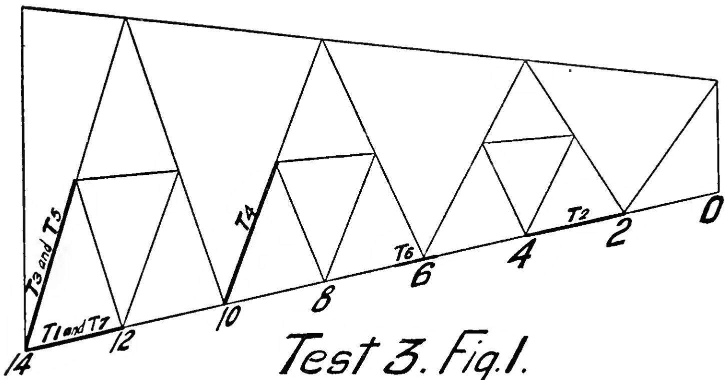
Test Piece	Cross Section		Length	Actual Area Sq. In.	Elastic Limit Lbs per Sq In.	U
T 4 A	4 Webs 18" x 13/32"; 8 Ls 2" x 2" x 1/4" - 3.25#; 12 Flats 2" x 1/4"; 10 Ls 2" x 1 1/2" x 1/8"; 2 Horz Webs 4 1/4" x 1/8"			45.763	45,482	
T 4 B				45.763	42,309	
T 6 A	4 Webs 18" x 7/16"; 1 Cover 28" x 3/16"; 16 Ls 2" x 2" x 1/4"; 1 Cover 7 1/2" x 3/16"; 4 Flats 2" x 3/16"; 4 Flats 2" x 1/4" } Section Constant for a length of 2'1/2" at m-n; Section changes from 56.66 Sq In to abt 114.7 Sq In	 Section m-n		66 66 at m-n	Could not be determined as section changes continuously	
T 6 B				56.66 at m-n	Ditto	
T5B Long	4 Webs 28 7/8" x 11/32"; 4 Ls 2 3/4" x 2 3/4" x 11/32"; 4 Flats 2 3/4" x 11/32"; 2 Ls 2 3/4" x 2 1/16" x 3/16"			52.305	37,017	
T5A Long				52.305	37,017	
T 2 A	4 Webs 15 3/4" x 9/32"; 4 Ls 2" x 2" x 3/16"; 4 Ls 2" x 1 1/2" x 5/32"; 4 Flats 2" x 3/16"; 4 Covers 4" x 5/32"; 1 Cover 7 1/2" x 3/16"; 8 Ls 2" x 2" - 3/16"; 2 Plates 12" x 5/16"; 6 Ls 2" x 1 1/2" x 1/8"			40.21 At Breaking Point	40.126	
T 2 B				40.21 At Breaking Point	40,126	
T 3 A	8 Webs 21" x 1/4"; 8 Flats 2" x 1/4"; 8 Ls 2" x 2" - 3.25#; 6 Ls 2" x 1 1/2" - 1/8"		 To break T3 A-2 holes 2 1/2" φ were drilled in each Web. To break T3 B-2 holes 3" φ were drilled in each Web	56.358; 46.792 Through 2 1/2" φ Holes	38,526	
T 3 B	Same as T3 A.			57.10; 44.871 Through 3" φ holes	41,820	
T5A Short	4 Webs 21" x 1/4"; 4 Ls 2" x 2" - 1/4" - 3.25#; 4 Flats 2" x 1/4"; 2 Ls 2" x 1 1/2" - 1/8"			27.59	41,953	
T5B Short				27 59	45,615	
T 1 A	8 Webs 20 21/32" x 7/32"; 4 Top Ls 2" x 2" x 3/16"; 4 Top Ls 2" x 1 7/8" x 5/32"; 4 Top Flats 2" x 1/8"; 4 Top Covers 4" x 1/8"; 1 Top Cover 6 5/8" x 1/8"; 8 Bottom Ls 2" x 2" x 3/16"; 2 Bottom Cov 11 1/8" x 1/8"; 6 Center Ls 2" x 1 3/8" x 5/32"		 To break T1 A-2 holes 2 5/8" φ were drilled in each Web	56 806; 46 335 through 2 5/8" φ holes	43,947	
T 1 B			To break T1 B: First 2 holes 2 13/16" were drilled in each Web. Then the 2 holes in each Web were enlarged to 3 3/4" φ	T1 B 57 497; 47 069 through 2 13/16" holes; 43 722 through holes 3 3/4" φ	42,654	
T 7 A	8 Webs 5 9/64" x 5/16"; 4 Webs 10 1/4" x 5/16"; 8 Webs 10 19/64" x 3/16"; 4 Top Ls 2" x 2" x 5/16"; 4 Top Ls 2" x 1 3/8" x 5/32"; 4 Flats 2" x 1/8"; 4 Covers 4" x 1/8"			55.249 At Center	39,716	

Test Piece	Cross Section		Length	Actual Area Sq. In.	Elastic Limit Lbs per Sq In	Ultimate Strength Lbs. per Sq. In.	Proportion of Linear dimensions to full size member	L/R around Hor. Axis	L/R around Vert. Axis	Representing Truss Member
T 4 A	4 Webs 18"x 13/32" 8 ∟s 2"x2"x 6"-3.25# 2 Flats 2"x 1/4" 10 ∟s 2"x 1 1/2"x 1/8" 2 Horiz Webs 4 1/4"x 1/8"			45.763	45,482	50,840	1/4	63.8	43.8	C L 10 - M 9
T 4 B				45.763	42,309	51,264				
T 6 A	4 Webs 8"x 7/16" 1 Cover 28"x 3/8" 16 ∟s 2"x2"x 1/4" 1 Cover 7 1/2"x 5/16" 4 Flats 2"x 3/16" 4 Flats 2"x 1/4" Section Constant for a length of 2'1/2" at m-n Section changes from 56.66 Sq In to abt 44.7 Sq In	 Section m-n		56.66 at m n	Could not be determined as section changes continuously	Highest load on 48,980# per □" Result: Buckling of Cover and Bulging of Ribs	1/4	As Section changes often, a section of one Rib will be considered only, taken symmetrical about neutral Axis 12.45	37.3	Panel Point C L 6
T 6 B				56.66 at m-n	Ditto	Ditto				
T5 B Long	4 Webs 28 7/8"x 5/32" 4 ∟s 2 3/4"x 2 3/4"x 5/32" 4 Flats 2 3/4"x 5/32" 2 ∟s 2 3/4"x 2 1/16"x 3/16"			52.305	37,017	51,022	5/32" of full size member or, 1 3/8 times linear sizes of T5 Short	34.9	61.6	C L 14 - M 13
T5 A Long				52.305	37,017	51,084				
T 2 A	4 Webs 15 3/4"x 3/32" 4 ∟s 2"x 2"x 3/16" 4 ∟s 2"x 1 1/2"x 5/32" 4 Flats 2"x 3/16" 4 Covers 4"x 5/32" 1 Cover 7 1/2"x 3/16" 8 ∟s 2"x 2"-3/16" 2 Plates 12"x 3/16" 6 ∟s 2"x 1 1/2"x 1/8"			40.21 At Breaking Point	40.126	57,782	1/4	43.6	30.5	C L 4 - L 2
T 2 B				40.21 At Breaking Point	40,126	58,264				
T 3 A	8 Webs 21"x 1/4" 8 Flats 2"x 1/4" 8 ∟s 2"x 2"-3.25# 6 ∟s 2"x 1 1/2"-1/8"		 To break T3 A-2 holes 2 1/2"φ were drilled in each Web	56.358 46.792 Through 2 1/2"φ Holes	38,526	Applied load of 49,120 lbs. per sq in for 24 hrs. then added a load of 7000 lbs. at center 55,515 Ult No load at Center	1/4	62.5	48.4	C L 14 - M 13
T 3 B	Same as T 3 A.		To break T3 B-2 holes 3"φ were drilled in each Web	57.10 44.871 Through 3"φ holes	41,820	Applied load of 49,000 lbs. per sq. in. besides a load of 17,000 lbs at center 58,252 Ult. No load at Center	1/4	62.3	48.4	C L 14 - M 13
T5 A Short	4 Webs 21"x 1/4" 4 ∟s 2"x 2"-1/4"-3.25# 4 Flats 2"x 1/4" 2 ∟s 2"x 1 1/2"-1/8"			27.59	41,953	54,972 Ult.	1/4	34.9	61.6	C L 14 - M 13
T5 B Short				27.59	45,615	56,954 Ult.				
T1 A	8 Webs 20 21/32"x 7/32" 4 Top ∟s 2"x 2"x 3/16" 4 Top ∟s 2"x 1 7/8"x 5/32" 4 Top Flats 2"x 1/8" 4 Top Covers 4"x 1/8" 1 Top Cover 6 5/8"x 1/8" 5 Bottom ∟s 2"x 2"x 3/16" 2 Bottom Cov 11 1/8"x 1/8" 6 Center ∟s 2"x 1 3/8"x 5/32"		 To break T1 A-2 holes 2 5/8"φ were drilled in each Web	56.806 46.335 through 2 5/8" holes	43,947	Highest load 50,134 Not broken 59,894 Ult.	1/45	32.2	30.2	C L 12 - L 14
T1 B			To break T1 B: First 2 holes 2 13/16" were drilled in each Web. Then the 2 holes in each Web were enlarged to 3 3/4"φ	T1 B 57.497 47.069 through 2 13/16" holes 43.782 through holes 3 3/4"φ	42,654	Highest load on 48,660 Not broken Highest load on 59,989 Not broken 63,990 Ult	1/45	32.2	30.2	C L 12 - L 14
T 7 A	8 Webs 5 5/64"x 3/16" 4 Webs 10 1/4"x 3/16" 8 Webs 10 13/64"x 3/16" 4 Top ∟s 2"x 2"x 3/16" 4 Top ∟s 2"x 1 3/8"x 5/32" 4 Flats 2"x 1/8" 4 Covers 4"x 1/8" 1 Cover 6 5/8"x 1/8" 8 Splices 2"x 3/16" 8 Bottom ∟s 2"x 2"x 3/16" 2 Covers 11 1/8"x 1/8" 6 ∟s 2"x 1 3/8"x 5/32"			55.249 At Center	39,716	50,056 Ult	1/45	32.1	30.2	C L 12 - L 14
T 7 B				55.249 At Center	38,560	48,770 Ult				

Results of Tests of Nickel-Steel Models Made by the Phoenix Bridge Company for the Board of Engineers, Quebec Bridge.

TEST 3 — Table No. 1

T_1B—The capacity of the testing machine, 48,660 lbs. per sq. inch, was applied for 5 minutes without failure. 2 - 2-13/16″ holes were then bored in each web and a load of 59,989 lbs. per sq. inch was applied for 20 minutes without failure. The holes in the web were then re-bored to 3 - ¾″. The member failed by buckling of the webs at the reduced section under a load of 63,990 lbs. per sq. inch of net section, or 48,660 lbs. per sq. inch of gross section.



Test 3. Fig. 1.

T_2A—The member failed by buckling of the ribs at the point of attachment of the transverse diaphragm near the pin end.

T_2B—One rib buckled at the point of attachment of the nearest transverse diaphragm to the pin end, the other three at the second set of diaphragms from that end.

T_3A—The full load of the testing machine, 49,120 lbs. per sq. inch, was applied for two and a half hours without failure. Then a vertical load of 7000 lbs. was applied to the center of the column so as to be supported by the edges of the webs only, and the full load of the testing machine was applied without failure. Finally 2 - 2 ½″ holes were bored in each web. The webs buckled at the reduced section.

T_3B—A load of 49,000 lbs. per sq. inch was applied without failure, and then a vertical load of 17,000 lbs., resting on the webs, was imposed at the center and the full axial load applied without failure. Afterwards 2 - 3″ holes were bored in each web. The member failed at the reduced section under a load of 58,252 lbs. per sq. inch of net section.

T_4A—At failure the piece bent downwards and found a support on some blocking inadvertently left in place, thereafter carrying a load of 52,650 lbs. per sq. inch. All four ribs were latticed on top and bottom flanges, and the piece failed near the center by buckling of the ribs near the center. The forked end showed slight signs of scaling near the pins at a load of 40,000 lbs. per sq. inch which gave a bearing stress on the pin of about 45,000 lbs. per sq. inch.

T_4B—The forked end showed slight signs of scaling at a load of 36,800 lbs. per sq. inch, corresponding to a bearing stress upon the pin of about 43,300 lbs. per sq. inch. The mode of failure was almost identical with that of T_4A, the inner ribs of both buckling outwards at the top flange. The bottom latticing at the point of failure showed very little distortion.

T_5A (Long)—The member failed by buckling of the ribs close to one of the transverse diaphragms.

T_5B (Long)—The failure was similar to T_5A (Long), and also took place at the point of attachment of a transverse diaphragm.

T_5A (Short)—Failure was by buckling of the ribs at the point of attachment of a transverse diaphragm.

T_5B (Short)—Failed by buckling of ribs at point of attachment of a transverse diaphragm nearest pin end. One lattice bar in the central plane failed in tension without shearing its connecting rivets, and one angle of the lower transverse diaphragm was torn from the rib throughout its depth.

T_6A—Two square ends. The sectional area changed almost continuously from point to point, being 56-66 sq. inches at the weakest point. The highest load applied, 48,980 lbs. per sq. inch, did not cause complete failure, but a section of the 3/16" top cover plate bulged up at a point where the rivet pitch in the line of stress was 2", that is to say, 10-2/3 times the thickness of the plate, and 9-1/7 times the diameter of the rivet. The upper flanges of the outer rib also bulged outward so as to take a permanent set.

T_6B—The results were almost identical with those of T_6A.

T_7A—Was similar to T_1, except that the webs were spliced longitudinally so as to avoid the use of wide plates. The distortion was greater from the beginning of the test than in the corresponding member with solid web. One rivet had failed at a load of 23,363 lbs. per sq. inch. Failure took place by bucklnig of the ribs at the transverse diaphragm nearest the central splice and a large number of loose rivets were found in the vicinity of the point of failure and in the reinforcement at the pin end.

T_7B—Failure was similar to T_7A but took place at the diaphragm nearest the pin end.

As might be expected from the local character of the failures, there is but little relation between the ultimate strength and the ratio of length to the least radius of gyration.

T_1A, T_1B, T_3A and T_3B would certainly have failed at lower unit stresses had sufficient power been available to test them to destruction without reduction of area.

A noteworthy feature of the test is that in all cases where the points of failure were not predetermined by boring holes in the webs, with the exception of T_4A and T_4B, failure took place at the points of attachment of the transverse diaphragm. This might be expected in as much as, when the member as a whole is shortened, the action of the lattice bars tends to spread the ribs apart. The restraining action of the diaphragm, therefore, causes bending stresses of considerable magnitude in the rib.

The design and construction of the members were such as to develop an unusually high and satisfactory percentage of the yield point value of the material employed, except in the case of T_7A and T_7B, which members were obviously weakened by the longitudinal splicing of the webs.

4.—TESTS ON 6 EYE-BARS, 28 TENSION MEMBERS AND 12 COMPRESSION MEMBERS

Made at the Phoenix Iron Company's Testing Plant under the supervision of Professor H. M. Mackay of McGill University and Mr. James E. Howard, Engineer-Physicist of the Bureau of Standards, Washington, D.C., during January, February and March, 1912, for the St. Lawrence Bridge Co.

Brief descriptions of these tests, taken generally from Professor Mackay's reports, follow:—

Test on 6 Eye-bars:

The six bars tested were each 10″ x 1½″ x 40′ 0″ c. to c. of pin holes. The end pins were 10″ diameter and the pin holes approximately 10-1/32″ diameter. Bars Nos. 1 and 2 were tested under their own weight. Bar No. 3 was counterpoised with three upward loads of 800 lbs. each applied at the centre and quarter points. Bar No. 4 was counterpoised with loads of 600 lbs. similarly applied; and bars Nos. 5 and 6 had loads of 50 lbs. hung upon them by wire loops at intervals of 13-1/3 inches throughout their length. Extensometer readings were taken, (1) by Mr. J. E. Howard's micrometer gauges over 20″ lengths at 9 points, (2) by Martens mirror extensometers over 4″ gauge lengths at 4 points, and (3) by the Phoenix Bridge Co's Olsen extensometers over 200″ gauge lengths on the top and bottom of the bar.

A general inspection of the curves, plotted from the extensometer readings, shows that the elastic limit (limit of proportionality) and yield point occur under very different loads at different parts of the bars; and while the elastic limit can be pretty closely determined, some arbitrary rule is necessary to determine the yield point. Relatively large deviations from the stress-strain curve, and considerable "creeping" of the extensometers under a fixed load, occur at points considerably lower than would be indicated by the "drop of the beam" or "drop of the mercury column" in a rapid continuous test.

Perhaps the load at which the stress-strain curve becomes sensibly a horizontal line may be as good a point as any to choose for the yield point. The following table is compiled on that basis. The figures given are the ***axial*** loads per square inch.

TEST 4 — Table No. 1 — EYE-BARS

Bar	Elastic Limit		Yield Point		Ultimate Strength	Remarks
	Ends	General	Ends	General		
1	14,000	21,000	26,000	28,000	60,200	Under its own weight
2	12,000	20,000	24,000	27,000	59,470	do do
Mean	13,000	20,500	25,000	27,500	59,835	
3	18,000	23,000	28,000	28,000	59,970	Counterpoised
4	16,000	26,000	28,000	30,000	59,620	do
Mean	17,000	24,500	28,000	29,000	59,795	
5	16,000	24,000	28,000	28,000	60,100	Cross loaded.
6	16,000	26,000	30,000	30,000+	62,190	do do
Mean	16,000	25,000	29,00Q	29,000+	61,145	

While the above values are tentative and might be selected differently according to the definitions adopted, they probably give as good a basis as any for comparing the bars with each other.

From the above comparison and a study of the curves, it may be concluded,

(1) That the elastic limit and yield point occur not only at much lower axial loads, but also at much lower measured fibre stresses at the ends than at the centres. The most reasonable explanation seems to be that the metal near the ends of the bars was affecting by the forging, so that the elastic limit and yield point there are, in fact, lower than in the central portion of the bar.

(2) That the counterpoised and cross loaded bars, although representing the extreme conditions tested, show very little difference as to their yield point or elastic limit. In the central portions of the bar the distribution of stress tends to equalize itself as the axial load increases.

(3) While the elastic limit and yield point for bars No. 1 and No. 2 are smaller than for the others, an inspection of the measured stresses at which these points occur shows that the low values are in all probability due to the quality of the material.

(4) There is no evidence that the weights of the bars or cross loading had any influence in lowering the ultimate strength.

(5) The bars all broke in the body between 5 ft. and 13 ft. from an end. No particular relation is traceable between the point of fracture and exceptionally low load values of elastic limit and yield point. As is shown in the "measurements after testing," several bars exhibited marked elongations and reductions of area at points other than that at which fracture actually took place.

Tests on 22 plate members.

12 of these test pieces were made of single plates 36" x ⅝", with pin plates of varying lengths while the remaining 10 were made of 3-36" x 5/16" plates stitch-riveted together and having pin plates all the same length.

The make-up and general results of the tests are summarized in Table 2. Column 8 gives the ultimate strength per square inch of gross nominal sectional area. The test pieces ran from 2.36% light to 5% heavy of the nominal calculated weights. The ultimate strengths corrected for the percentage by which the whole piece is light or heavy are given in Column 9. For the ultimate strength per square inch of net section given in Column 11, three rivet holes one inch diameter are deducted. The values of the yield point and the elastic limit are not included in this table under these names as it is uncertain as to just what they refer to in a built up member, however, there is given in Column 12 the stress per square inch of gross section at which the stress-strain diagram over 200 inches of length on the scale plotted ceases to be a straight line. Column 13 gives the modulus of elasticity for the central 200 inches of length including the splice in case of spliced members. It will be noticed that the modulus for the spliced members runs a little lower on the whole than that of the unspliced ones notwithstanding the extra section.

Several of the earlier test pieces failed by dishing back of the pin hole. Steps were taken to avoid this by driving strips of white pine between the pin plates and pulling shackles just back of the pin. The restraint thus introduced, although slight, was sufficient to prevent dishing. In nearly every instance where dishing occurred it was evident from the distortion of the web at the ends of the pin plates that failure was very near, and it is probable that the ultimate strengths obtained were but little reduced by the dishing.

Extensometer readings were taken on 200 inch gauge lengths on two sides and two edges of each piece by means of Olsen Extensometers, and the elastic limit was determined by means of these readings. Numerous readings were also taken with a Howard gauge with a view to obtaining the distribution of the stress across the section, particularly at the ends of pin plates and splice plates. Numerous readings were also taken from the ends of pin or splice plates to the web in order to obtain the slip between the web plate and the pin (or splice) plates at the first line of rivets.

TEST 4—TABLE 2—

1	2	3	4	5	6	7	8
Member	Date of Test 1912	Webs	Pin Plates	Splice Plates	Material of Web Ultimate Strength	Member Light or Heavy %	Ultimate Gross Nominal Section
$A_1$1	Jan. 15	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-1″	....	68700	3.7H	44660
$A_1$2	Jan. 15	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-1″	....	68700	1.5H	52000
$A_2$1	Jan. 16	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-1″	....	68700	5.1H	50980
$A_2$2	Jan. 17	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-1″	....	68700	4.0H	53330
B_2X1	Feb. 16	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-7″	....	68700	1.5H	52270
B_2X2	Feb. 16	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-7″	....	68700	1.6H	52000
B_2Y1	Feb. 20	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 5′-1″	....	62900	0.2L	46930
B_2Y2	Feb. 21	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 5′-1″	....	63440	....	48670
C_2X1	Feb. 19	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-7″	36″x5/16″ x 3′-8″	68700	2.6L	51330
C_2X2	Feb. 20	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-7″	36″x5/16″ x 3′-8″	68700	2.4L	53330
C_2Y1	Feb. 17	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-7″	36″ x 3/8″ x 3′-2″	68700	1.0L	50670
C_2Y2	Feb. 19	1 Pl. 36″ x 5/8″	36″ x 1/2″ x 4′-7″	36″ x 3/8″ x 3′-2″	68700	0.8L	52270
$D_1$1	Feb. 21	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	....	65620	1.4L	49510
$D_1$2	Feb. 22	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	....	65620	1.2L	47110
$D_2$1	Feb. 22	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	....	65620	1.7H	51730
$D_2$2	Feb. 23	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	....	65620	1.9H	47550
$D_3$1	Feb. 23	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	....	65620	0.8L	51380
$D_3$2	Feb. 24	3 Pls. 36″x5/16″	36 x 3/4″ x 4′-7″	....	65620	0.7L	48710
E_2X1	Feb. 14	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	36″x5/16″ x 5′-6″	62910	1.7H	53330
E_2X2	Feb. 15	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	36″x5/16″ x 5′-6″	62910	1.9H	52000
E_2Y1	Feb. 15	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	36″x5/16″ x 4′-6″	62910	2.2H	52440
E_2Y2	Feb. 15	3 Pls. 36″x5/16″	36″ x 3/4″ x 4′-7″	36″x5/16″ x 4′-6″	62910	2.1H	54930

9	10	11	12	13	14	15
Ultimate Gross Corrected Section	Mean	Ultimate Net Section	Limit of Proportionality, 200″	E over 200″ Length (10^6)	Mode of Failure	Remarks
43060	....	46970	28000	28.5	Rivet Line end of Pin Plate.	
51220	47140	55880	24000	28.5	Dished Back of Pin.	
48440	....	52840	29300	28.5	Rivet Line end of Pin Plate.	
51260	49850	55920	29500	28.5	Rivet Line end of Pin Plate.	
51490	....	56170	26200	28.5	Dished Back of Pin.	
51330	51310	56050	26700	28.5	Dished Back of Pin.	
47030	....	51300	20000	28.5	Rivet Line end of Pin Plate..................	Pin Ends Blocked, 4″ x 1 3/8″ White Pine.
48670	47850	53090	25000	26.6	Rivet Line end of Pin Plate..................	Pin Ends Blocked as above.
52710	....	57500	30700	28.9	Rivet Line end of Pin Plate..................	Pin Ends Blocked as above.
54620	53665	59580	26700	25.0	Rivet Line end of Pin Plate..................	Pin Ends Blocked as above. Stretched to 30,000 lbs./sq. in. before Testing.
51170	....	55820	24000	28.5	Dished Back of Pin.	
52700	51935	57490	20000	26.6	Rivet Line end of Splice Pl.....................	Pin Ends Blocked.
50230	....	54800	26700	25.1	Rivet Line in Body.	
47690	48960	52020	26700	26.0	Dished Back of Pin.	
50860	....	55480	26700	28.5	Rivet Line end of Pin Plate..................	Pin Ends Blocked.
46600	48730	50840	25300	27.0	Rivet Line in Body......	Pin Ends Blocked.
51880	....	56600	25500	28.5	Rivet Line end of Pin Plate..................	Pin Ends Blocked.
49000	50490	53450	25500	27.0	Rivet Line end of Pin Plate..................	Pin Ends Blocked.
52420	....	57190	28000	27.2	Rivet Line end of Splice Plate..................	Load reduced to 0 after reaching 24,000 lbs./sq.in. Ensuing E = 30.1 x 10^6.
51020	51720	55660	30700	26.9	Rivet Line end of Splice Plate..................	Load reduced to 0 after reaching 24,000 lbs./sq.in. Ensuing E = 29.2 x 10^6.
51340	....	56000	28000	27.0	Rivet Line end of Splice Plate.	
53790	52565	58680	31000	27.0	Rivet Line end of Splice Plate.	

Large slips at the point might result from either comparatively loose rivets or high stresses on the rivets, and possibly on this account there seems to be no particular relation between the amount of slip and the strength of the test piece.

The slip was usually found to be very small or zero up to a load of from 4000 to 8000 lbs. per square inch on the test piece, but the relation between this slip and the load was usually not linear; it was often nearly so in the middle range of loading say from 8000 to 24,000 lbs. per square inch. It may be concluded that when the rivets are well driven friction between the plates prevents slip for the lower loads, and that the point where the slip begins marks the failure of friction to maintain its load. When the slip becomes proportional to the load the friction has probably become negligible and the slip likely denotes the elastic deformation of the rivet or the metal surrounding the rivet hole, which continues until the local elastic limit is reached.

The stress-strain curves for the web plates indicate a very uniform distribution of stress. It may be said of the web plates (1) that within the elastic limit the highest strain observed at any section seldom exceeds the mean for that section by more than 20%. (2) That the variation of the strain across the face of the plates from top to bottom, or of the mean strain of the two faces from top to bottom, are still less, the highest strain seldom exceeding by more than 6% or 7% the mean for the whole section. (3) That no relation is traceable between variation of stress across the section and the ultimate strength of the test piece.

The results of the tests show that the stresses in the web and pin plates are probably equalized within less than two feet from the ends of the pin plates, and that the length of the pin plate apparently has no effect on the distribution.

From a careful study of the data on the 22 tests it would appear that the variation in the arrangement of rivets, lengths of pin plates, and lengths of splice plates are of secondary importance as regards strength and rigidity.

Tests on 6 two-web tension members.

The make-up and general results of the tests of these members are given in Table No. 3. Numerous extensometer readings were taken in an endeavor to throw light on the action of built-up members.

All the slip curves are quite similar in character to those of the plate members.

Nearly all the curves show at the foot the effect of the weight of the member but the general impression given is that of a remarkably uniform distribution of stress at all points measured. No clear evidence was obtained to show that the stresses due to the weights of the members had any influence on the breaking strength.

TEST 4—TABLE 3—2-WEB TENSION MEMBERS

Member	Main Section	Web Splices	Material Ultimate Strength	Member Light or Heavy %	Ultimate Gross Section	Ultimate Net Section	Mean	Mode of Failure
T101	2 Webs 24″ x ½″ 4-L's 4″x3″ x 5/16″	2-24″x5/16″Pls3′-0″lg. 2-16″x5/16″ " " " 4-6½″ x 3/8″ " " "	Pls 63360 Ls 64360	0.03H	47100	57090	......	Webs Rivet Line end of splice Ls one space from splice.
T102				0.12L	45830	55580	56335	Broke in Body.
T111	2 Webs 24″ x 3/8″ 8 L's 4″x3″ x 5/16″	4-16″x5/16″ Pls3′-0″ lg. 8-6½″ x 3/8″ " " "	Pls. 63360 Ls 64360	0.28L	45810	57420	......	Webs Rivet Line end of splice. Outer Ls Rivet Line end of splice Inner Ls one space from splice
T112				0.53L	46210	57950	57685	Same as T111
T121	1 Web 36″ x 5/16″ 2 do 28″ x 5/16″ 4 Ls 4″ x 3″ x 5/16″	2-35″ x 3/8″ Pls 4′-3″ lg.	Pls 63470 Ls 63920	2.03L	53130	58880	......	Broke in Body.
T122				1.47L	50060	55440	57160	Broke in Body

A very noticeable feature was the reduction of section of the angles and their opening up from the webs at practically all points of lattice connection. Extensometer readings did not show, within the elastic limit, any stress of importance produced by the action of the lattice bars as the member elongates, and it does not seem that such stresses can amount to much at points remote from diaphragms or tie plates.

Curves plotted from extensometer readings taken on the top and bottom splice plates between the two webs show a comparatively small stress in these plates, that the stresses get smaller away from the edges and that these plates do not do their share of the work.

In T-11 No. 2 the extensometer readings were limited to the top flanges, stay plates and lattice bars, the curves show considerable transverse compression at the ends of the end and center stay plates doubtless due to the action of the lattice bars in pulling the ribs together while measurements taken at the center stay (or splice) plate show very little distortion. Readings taken on the lattice bars gave very little information of value.

On the whole it will be seen that the six members developed in their net sections about 90% of the ultimate strength of the material of which they are composed which seems a very satisfactory result.

Twelve compression tests of built-up members.

This series of tests included 6 short columns 10 ft c. to c. of pins marked TC_1, TC_2 and TC_3; and 6 columns 49 ft. c. to c. of pins marked TC_4, TC_5 and TC_6. Two pieces of each mark were tested. The make-up and general results of the tests are summarized in Table No. 4.

The short columns differed only in the length of the pin plates. A prominent feature of these members is the great weight of the lattice bars. As might be expected, these members all failed by local buckling. The members with the longer pin plates sustained heavier loads, but the gain in strength is not so great as the increase in weight, so that the longer pin plates do not give higher economic efficiency.

The mode of failure was nearly the same in all six members. In five members both ribs buckled outwards beginning at the end of the longest pin plate. In the sixth one rib buckled at that point, the other about a lattice panel nearer the centre. At failure scaling lines were developed all along the inside of the webs, some were also observable on the outside, particularly near the ends. Generally speaking no scaling lines developed in the angles.

The long columns had values of l/r from 73.7 to 80.8. As these members were tested horizontally without counterpoise the weights and resulting deflections had a very marked influence in inducing failure. They all failed by bending until they touched the cross beams at the bottom of the testing machine without any perceptible local buckling. No loose rivets or deformed lattice bars were noticed.

TEST 4 — TABLE 4 — COMPRESSION MEMBERS

Item	Mark	Material of Member	Nominal Area Sq. ins.	Length c. to c. Bearings	Ratio L/R	Elastic Limit lbs. per sq. in.	Yield Point lbs. per sq. in.	Ultimate Strength lbs. per sq. in.	Remarks
1	TC1-1	2-22 x 5/8 pl. 4-4 x 4 x 5/8 Ls.	46.46	9′–5″	16.2	Not Determined	30844	37293	Failed by buckling of webs
2	TC1-2	do	do	do	do	do	34941	39271	do
3	TC2-1	do	do	do	do	do	33905	38637	do
4	TC2-2	do	do	do	do	do	32599	38410	do
5	TC3-1	do	do	do	do	do	36000	41958	do
6	TC3-2	do	do	do	do	do	35373	40537	do
7	TC4-1	2-22 x 3/4 pls. 4-4 x 4 x 5/8 Ls.	51.96	48′–5″	80.8	18886	20138	26306	Failed by vertical deflection downward
8	TC4-2	do	do	do	do	18977	20242	26062	do
9	TC5-1	2-24 x 3/8 pls. 8-4 x 4 x 3/8 Ls. 1-16 x 3/8 pl. 4-4 x 3 x 3/8 Ls.	56.56	do	73.7	20278	27037	30698	do
10	TC5-2			do	do	20299	25938	29941	do
11	TC6-1	2-22 x 3/4 pls. 4-4 x 4 x 5/8 Ls.	51.96	do	80.8	18948	24000	26464	do
12	TC6-2	do	do	do	do	18930	22716	25240	do

Numerous readings were taken with Olsen, Howard and Marten's extensometers. The Howard gauge readings in the first member of each kind were devoted principally to investigate the distribution of stress in the ribs near the pin plates. In the second member particular attention was devoted to the relative motion of the two ribs. The behavior of each pair of members was remarkably uniform.

Points of high local stress are obviously, in general, the points which limit the strength of the columns. Considering the 12 tests included above the highest stresses noted were just in front of the pin. These stresses are not of so much importance, as the high values occur only for a very short length, and in all the members of the present series transverse diaphragms made the webs secure against local buckling at that point. In the long columns the deflection due to weight is clearly the determining factor. Apart from these considerations the part of the column seeming to need the closest attention is that just at the end of the pin plates. High local stresses have been noted there. Among the causes are, (1) Inequality of stress between the two ribs due to lateral deflection or imperfect workmanship. (2) Concentrations of stress in the webs from rivets of pin plates which do not extend the full width of the member. These loads are usually applied eccentrically as regards the web and must tend somewhat to cause dishing. (3) Pin pressure applied eccentrically as regards the centre of gravity of the rib. (4) The combined action of the tie plates and lattice bars in bending the ribs at the ends of the former.

Slip between the pin plates and the webs was observed of the same character as the corresponding slip in the tension tests, indicating a large concentration of stress on the rivets near the end of the pin plates.

Vertical readings over a 10″ length at various points in the webs were taken, all showing the transverse expansion in the plates accompanying longitudinal compression.

The transverse readings taken on the second member of each pair to determine the lateral deflection of the ribs due to the action of the lattice bars, are of considerable interest. The spread of the ribs in the three members measured was very nearly the same.

Under the most favorable circumstances the curvature indicated must be accompanied by rather heavy bending stresses.

The spreading of the ribs increases rapidly at high loads.

Transverse measurements at the end of the tie plates denote a transverse tension stress as high as 7,500 lbs. per square inch.

5.—TESTS OF 6 CARBON STEEL EYE-BARS WITH OBLONG PIN HOLES

Extracts from report by Professor H. M. Mackay

This report includes the tests of six eye-bars, nominally 14″ x 1½″ x 12′ 0″ centre to centre of pins, made at Phoenixville, September 10th to 13th, 1912. The pin holes at one end, called "East," were circular; those at the other end, called "West," were oblong so that the longitudinal diameter was ½ inch greater than the transverse. Details of the actual measurements of the bars are given in Table 1. The main object of the tests was to discover any differences in the strength or the distortion of the heads which might arise from the elongating of the pin holes.

Method of Testing.

The bars were measured and points were established on the axis of the bar, on both sides and at each end, at a distance of 3′ 0″ from the back of the pin holes. The bars were then placed in the testing machine and a load of 20,000 lbs. per sq. inch applied. Under this load the elongation from the *front of the pin* to the points established as above was measured by means of a scale graduated to one-hundredth of an inch. The bars were then removed from the machine and the permanent set in the three feet from the back of the pin holes was measured by means of a Howard gauge. The bars were then loaded to 28,000 lbs. per sq. inch and corresponding measurements made; after which the bars were again placed in the machine and pulled to destruction.

The measurements from the front of the pin to the "three ft. point," under the applied load, are assumed to give elongation in the three ft. length from the back of the pin holes. The only error in this assumption would arise from the flattening of the pin. It would be difficult to devise any scheme of measurement which would entirely eliminate this error, and it must be so small compared with the distortions observed as to be negligible, at all events for comparative purposes.

Results of Tests.

Table 1 gives the yield points and general results of the tests to destruction. The yield points were determined by the drop or halting of the mercury gauge. Bars 3 and 4 gave unmistakable yield points at 24,350 and 25,650 lbs. per sq. inch respectively; but when these bars were replaced in the testing machine after being loaded to 28,000 lbs. per sq. inch, new yield points of 29,220 and 29,120 lbs. per sq. inch respectively were obtained. All bars failed in the body, but the East head of bar No. 2 (which was actually tested first) dished back of the pin, the dishing being almost simultaneous with the fracture in the body. To prevent recurrence of the

TEST 5—TABLE 1—EYE-BARS

Bar	Actual Section	Thickness of Head	Yield Point	Ultimate Strength	Elong.n in 7'-1" %	Section at Fracture	Reduction of Area %	Elongation of Pin Holes		Elongation in 3'-0" from Back of Pin Hole		Character of Fracture and Remarks
								East	West	East	West	
1	14.10" x 1.487" =20.97□"	East 1.443" West 1.441"	30040	61230	24.4	11.00" x 1.13" =12.43□"	40.7	3.71"	2.75"	5.00"	3.62"	Silky Angular 40% Fine Granular Heads Blocked
2	14.16" x 1.525 =21.59□"	East 1.460" West 1.471"	31960	60860	23.7	11.03" x 1.05" =11.86□"	45.0	3.69"	2.72"	4.81"	3.56"	Silky Half Cup 20% Fine Granular East Head dished behind Pin
3	14.16" x 1.523" =21.56□"	East 1.464" West 1.480"	24350 (29220) *	59830	21.5	10.96" x 1.40" =15.34□"	28.9	2.42"	3.42"	3.19"	4.69"	Silky Half Cup Heads Blocked *After loading to 28,000 and releasing
4	14.08" x 1.495" =21.05□"	East 1.480" West 1.460"	25650 (29210) *	60990	19.5	12.90" x 1.12" =14.45□"	31.3	2.36"	2.52"	3.50"	3.81"	40% Silky Cup 60% Fine Granular Heads Blocked
5	14.08" x 1.510" =21.26□"	East 1.490" West 1.460"	29910	60390	26.6	10.92" x 1.10" =12,01□"	43.5	2.63"	3.58"	3.44"	4.81"	60% Silky Angular 40% Fine Granular Heads Blocked
6	14.10" x 1.515" =21.36□"	East 1.485" West 1.450"	29210	59550	27.2	11.08" x 1.10" =12.19□"	42.9	2.74"	3.56"	3.50"	4.62"	Silky Cup Heads Blocked
Mean.	....	....	....	....	....	....	....	2.93"	3.26"	3.91"	4.18"	

dishing, the heads of all the other bars were rather lightly blocked with strips of white pine inserted between the heads and the shackles, just back of the pins.

The elongations of the pin holes after testing are in two cases greater in the East (round) holes, and in four cases greater in the West (oblong) holes. The mean elongation of the round holes is 2.93 inches, and that of the oblong holes 3.26 inches. Similar relations exist in the elongations of the three foot length measured axially from the backs of the pin holes, the mean elongation for the East end being 3.91 inches and for the West end 4.18 inches.

Table 2 gives the permanent sets in the three foot length referred to, after loads of 20,000 and 28,000 lbs. per sq. inch. After the 20,000 lb. load these values are greater for the West end in bars 1 and 6; for the East end in bars 3 and 5; and the same for both ends in bars 2 and 4. The greatest permanent set .026 inch is at the West end, while the mean for the east end is .0107, and for the West end .0130, a difference of .0023 inches.

After a load of 28,000 lbs. per sq. inch, the permanent sets in the same distance are greater for the East end in bars 1 and 2; and greater for the West end in all the others. The mean for the East end is .0795 inches and for the West end .0906 inch, a difference of .0111 inch.

Table 3 gives the elongations *under* loads of 20,000 lbs., and 28,000 lbs. per sq. inch in the same three foot length from the back of the pin hole, the actual measurements having been made as explained above from the front of the pin. The elongations under the 20,000 lb. load are more concordant than the permanent sets. The mean elongation for the East end is .048 inch and for the West end .052, a difference of .004 inch. For four bars the elongation is greater in the West end, and for two bars the same in both ends. Under the 28,000 lb. load, the elongation is greater in the East end for two bars, greater in the West for three, and the same in both ends for one. The mean elongation for the East end is .134 inch and for the West .150 inch.

From the above data it may be concluded that (1) There is no evidence that the ultimate strength of the bars is affected by elongating the pin holes, since all bars broke in the body. (2) At a load of 20,000 lbs. per sq. inch the elongation and permanent set in the oblong heads are on the average greater than in the others by about .004 inch and .002 inch respectively, but these differences are probably too small to be of much significance or of great importance. (3) At higher loads the distortions of the oblong heads continue to be greater on the average, but the differences are rather small compared with variations in the individual bars. In some instances indeed the omission of a single bar would make the mean permanent set or distortion, as the case may be, less for the oblong end than for that with the circular pin hole. These variations are not astonishing in view of the fact that the elastic limit is undoubtedly reached locally in all heads at a lower load than 20,000 lbs. per sq. inch.

TEST 5—Table 2

PERMANENT SET IN 3′-0″ LENGTH. MEASURED FROM BACK OF PIN HOLE AFTER LOAD OF 20,000 LBS. PER SQ. IN.

Bar	East = Round Pin Hole.			West = Slotted Pin Hole.			Remarks
	North	South	Mean	North	South	Mean	
1	.007	.008	.0075	.027	.025	.0260	
2	.016	.000	.0080	.016	.000	.0080	
3	.022	.011	.0165	.011	.005	.0080	
4	.007	.008	.0075	.011	.004	.0075	
5	.011	.011	.0110	.008	.010	.0090	
6	.013	.014	.0135	.019	.020	.0195	Scaling behind Pin, West End, below 20,000 lbs. / sq. in.
	Mean.		.0107	Mean.		.0130	

AFTER LOAD OF 28,000 LBS., PER SQ. IN.

Bar	North	South	Mean	North	South	Mean	Remarks
1	.127	.120	.1235	.072	.035	.0535	
2	.090	.078	.0840	.031	.016	.0235	East Head scaling between 20,000 and 28,000 lbs.
3	.081	.068	.0745	.149	.137	.1430	West Head heavily scaled at 28,000 lbs. behind Pin.
4	.033	.028	.0305	.045	.031	.0380	Scaling general at 26,000 lbs.
5	.053	.055	.0540	.063	.062	.0625	
6	.111	.110	.1105	.219	.227	.2230	Heavy Scaling Both Heads at 26,000 lbs.
	Mean.		.0795	Mean.		.0906	

TEST 5—Table 3

Axial Elongation in 3'-0" Length. Measured from Back of Pin Hole, Load 20,000 Lbs. per Sq. In.

Bar	East = Round Pin Hole.			West = Slotted Pin Hole		
	North	South	Mean	North	South	Mean
1	.04	.04	.040	.05	.04	.045
2	.05	.04	.045	.05	.04	.045
3	.05	.05	.050	.06	.04	.050
4	.05	.04	.045	.06	.04	.050
5	.05	.06	.055	.07	.05	.060
6	.06	.05	.055	.07	.05	.060
	Mean.		.048	Mean.		.052

Load 28,000 Lbs. per Sq. In.

1	.15	.14	.145	.10	.09	.095
2	.14	.15	.145	.09	.08	.085
3	.13	.14	.135	.21	.19	.200
4	.09	.10	.095	.11	.08	.095
5	.11	.11	.110	.15	.13	.140
6	.17	.18	.175	.31	.26	.285
	Mean.		.134	Mean.		.150

6.—PIN FRICTION TESTS

A test to investigate the pin friction in eye-bar heads and the efficiency of certain lubricants was made at McGill University during the early months of 1912.

Extracts from the report by Professors Brown and Mackay follow:—

An apparatus was designed whereby an eye-bar under tension could be subjected to a bending moment, the moment being applied to the pin through which the bar was loaded and being transmitted to the eye-bar by the friction between the surfaces of the pin and the head of the eye-bar. The principle of the apparatus is shown on Figure I.

Levers L_1 L_2 are keyed to the pins in the holes in the head of the eye-bar. Links L_3, L_4 and L_5, the latter of which terminates in a platform P clear of the eye-bar, are connected as shown by pin joints at A, B, C, D, and a load W_1 may be hung from P so as to be in the line of pull applied to the eye-bar. This gives tensions T_1 and T_2 in the vertical links L_3 and L_4. and thus moments T_1l_1 and T_2l_2 of equal magnitude are applied to the pins at the ends of the eye-bar. These moments bend the bar so that its axis moves into a position shown by the dotted line. If with a given pull on the eye-bar, a series of tests be made to determine the load W_1 which must be applied to the system of levers in order that the pins may slip in the holes, comparative values of the friction coefficient, using different pin clearances and different lubricants between the surfaces, can be determined.

Further if r = radius of pin in inches

T = tangential friction force between pin and eye-bar

Tr = moment of pin friction

$= T_1l_1$ or T_2l_2

Then $T = \dfrac{T_1l_1}{r}$

In the following report $\dfrac{T}{\text{Pull}}$ has been called the friction coefficient for the pin = u.

The lengths of the levers and radius of pin (2-½″) are such that

$$u = \frac{104.2 \quad W_1}{\text{Pull on the bar}}$$

Pins and Eye-bars supplied for Test.

Three eye-bars 5″ x ¾″ x 5′ 0″ c. to c. were used for the test. Two sets of pins of approximately the same diameter 5″ were used, one set having a highly polished surface while the other set had the ordinary machine turned surface. One eye-bar was bored so as to have a snug fit on the end and the other two had a clearance of about 1/32" in one case and slightly more than this in the other.

Method of test:—A given pull, say 60,000 lbs. was applied to the eye-bar and the lever L_5 was then adjusted, by means of a device at D, to a definite position and the load W_1 was then added in equal increments until slip occurred. After each increment the lever L_5 was adjusted to its initial position from which it had descended when W_1 was added. Another increment was then added and the process repeated until the slip load was reached. The apparatus itself and the weights used in balancing the lever system before adding Load W_1 gave a pull of 1,110 lbs. on the eye-bar. The addition of the weights W_1 also affected the pull of the bar and these two items are included in the pull used to determine the friction coefficient (amount given in the table is exclusive of these two items).

The first eye-bar tested had a snug fit on the pin. It was tested without lubricants and the average value of u for several loads was .40. Tests with lubricants was not made on this bar.

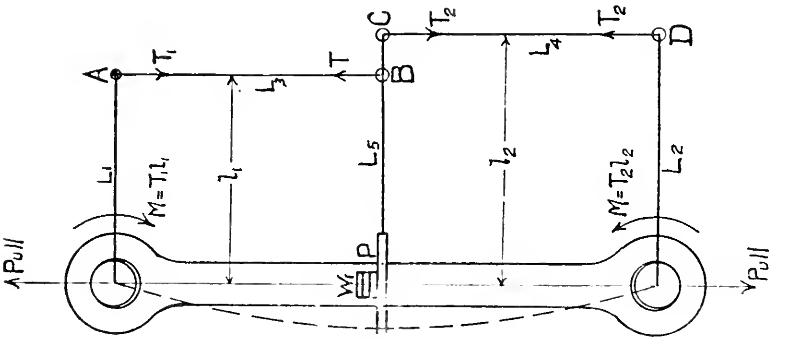
The results of the tests using the other two bars are given in Table 1.

TEST 6—Table 1—FRICTION COEFFICIENTS

Bar	Pin	Lubricant	"u" under pull 40,000	60,000	80,000	u Mean
2	Polished	Dry.............	{25,000 0.25}	0.29	0.27	0.27
		Heavy cylinder oil..	0.11	0.14	0.14	0.13
		Paraffin wax......	....	0.014	0.018	0.016
2	Turned	Dry (Residuum of Paraffin)........	0.076	0.066	....	0.071
		Dry (Cleaned by melting)........	0. 15	0. 17	....	0. 16
		Heavy cylinder oil.	0.114	0.112	0.134	0.120
		Tallow and white lead...........	....	0.042	0.038	0. 04
3	Turned	Dry.............	0.240	0.236	0.254	0.243
2		Tallow and white lead...........	....	....	....	0.013
		Graphite and Linseed oil.........	....	....	....	0.007
2		Necker's "XB" Bridge lubricant.	....	....	....	0.025
		Necker's "XXXB" Bridge lubricant.	....	....	....	0.023

The paraffin was applied by heating the pins with a gas blow torch until the wax melted and ran freely in a thin layer over the surface. The clearance 1/32″ was sufficient to allow the pins being slipped into position, but without damaging the wax layer. For the second series the same bar was used. The pin holes were thoroughly wiped out with dry cotton waste. The condition was assumed to be dry metal on dry metal. Unexpectedly low values of "u" were obtained (an average of .07). The previous tests showed an average value of u=.27. The difference was very striking and as it seemed possible that some of the paraffin wax had got thoroughly pressed in the uneven surface of the eye-bar heads and had not been removed by wiping with waste the bar was removed from the machine and heated with a gas blow torch so as to melt the residual wax. They were wiped off while hot and the test was then repeated. The value of u=.16 obtained was more than double that obtained before the hole was heated to remove the paraffin, thus demonstrating the marked effect of a small residuum of paraffin. The value is however much less than obtained in the first series of tests with the same eye-bar and the polished set of pins (u.=.27). This result may have been due partly to slight traces of paraffin which had not been removed by heating, to the fact that the eye-bar heads had been distorted in the previous tests, or it may be that the machined finished surface of the pin gave a bearing at a number of points more resembling a line of contact than an area of contact in the case of the smoother pins.

The tallow and white lead lubricant was made of three parts of white lead to one part of tallow by weight. The white lead was well stirred into the melted tallow while it was hot and the mixture was then allowed to cool before being smeared over the pin.



TEST 6 — Figure 1

7.—TESTS OF 8 TENSION AND 10 COMPRESSION MODELS OF MEMBERS USED IN THE QUEBEC BRIDGE AS BUILT

This series of tests was made at the Phoenix Iron Company's Testing Plant during February and March, 1913, for the St. Lawrence Bridge Co. and under the direction and personal supervision of Mr. James E. Howard, engineer-physicist, Bureau of Standards, Washington, D.C.

A description of these tests was published in the "Engineering Record" of March 21st, July 25th and November 14th, 1914, which is here reprinted by courtesy of the "Engineering News-Record."

Various details, diagrams etc., which acccompanied these articles are grouped in proper order for reference from the text as noted. A summary of the results is given in Table 1.

ENGINEERING RECORD, MARCH 21ST, 1914

ULTIMATE STRENGTH OF CARBON AND NICKEL STEEL MODELS OF QUEBEC BRIDGE MEMBERS

Destruction Tests of Precision on Large Riveted Compression Members in the 640-Foot Suspended Center Span.

A series of eighteen destruction tests of large-size models of tension and compression members for the trusses of the 1800-ft. cantilever span of the new Quebec Bridge has recently been made at the Phoenix Bridge Company's shop. The specimens were larger and stronger than similar members in many important railroad spans, and the results were carefully and accurately observed and recorded in great detail in order to furnish data for the analysis of the type of design, the character of detailing, fabrication and construction, the ultimate efficiency of the members, and especially for the comparative working value of duplicate compression members made of carbon or of nickel steel. A significant result is the demonstration of an elastic limit of a carbon-steel riveted compression member varying from 50 to 63 per cent of the elastic limit of a duplicate member made of nickel steel. The tests are also illuminating in their bearing on late developments of design for important tension and compression members.

Scope of Tests

The tests involved maximum loads of 2,800,000 lb. and included four tension tests on riveted plates, two tension tests on riveted members with rectangular cross-sections calculated to receive either tension or compression, and twelve compression tests on members made with two or more web plates and latticed flanges, models of compression members in the webs and chords of the cantilever trusses. The tests of two nickel steel and of two duplicate carbon-steel compression members showed the highest ultimate strength of the nickel steel members to be about 54 per cent greater than that of the weakest carbon-steel member and its elastic limit 101 per cent greater than that of the weakest carbon-steel member.

The compression tests in this series were made on models of the vertical and inclined web members and of the inclined bottom chords of the cantilever and anchor arms and suspended truss. The models have maximum cross-sectional areas of 70.65 sq. in. and maximum lengths of 45 ft. 10 in. The nickel steel models have a cross-sectional area of 35.2 sq. in. and a length of 34 ft. Some of the models represent members having shop weights up to 140 tons and lengths up to 90 ft.

The design of the old bridge was fully described and illustrated in many articles published in the Engineering Record and in the report of the Royal Commission, which was published in full in the Engineering Record of March 14, 1908, page 309. In connection with the official design for the new bridge prepared by the board of engineers, a number of models of its principal truss members were made in carbon-steel to a scale of about one-fourth the natural size and were tested to destruction by the Phoenix Bridge Company in the same machine which was used for the tests described in this article, giving results interesting for comparison of strength and efficiency of material and design. The largest of the sixteen models had a cross-sectional area of 57.1 sq. in. and a length of 35 ft. 9 in., and the highest elastic limit obtained was 36,618 lb. per square inch. The details of these tests were published in the Engineering Record of Nov. 19, 1910.

All of the models in both series were tested at Phoenixville, Pa., in the Phoenix Iron Company's machine which was built by them from their own design and has recently been overhauled, calibrated and standardized and the frictional correction determined, which has been applied wherever necessary to the results recorded for these tests. The machine is of a horizontal hydraulic type, giving direct stress up to a nominal capacity of 2,720,000 lb. tension or compression, and can receive members up to 50 ft. long for compression and about 40 ft. long for tension, dependent upon elongation of metal. It has a plunger area of 3000 sq. in. and stroke of about 6 ft. The cylinder head is permanently attached to the plate girder bed frame and the movable head can be pin-connected to the frame at any point within the limit of length. Pressure in the hydraulic cylinder has been developed by a direct-acting steam pump, but an electrically operated hydraulic pump is now used. To provide for testing compression members a steel casting with a half hole bearing for a horizontal pin 10 in. in diameter was bolted to the plunger head and split steel bushings were provided for it and for the pin bearing on the moveable head to correspond with the sizes of pins in the compression members. (See Figure 3).

Counterbalancing

Fulcrums consisting of steel pins in wooden half-hole bearings with clearance between them were set on the bed of the machine at three equidistant points intermediate between the fixed and movable heads, and supported horizontal I-beams transverse to the axis of the machine at their center points. One end of each I-beam projected across the compression specimen test and was attached to it by a bridle or sling inclosing the piece. From the other end of the beam was suspended a weight corresponding to the fraction of the weight of the truss member which it was intended to counterbalance, thus relieving the test specimen of secondary stresses due to its own weight.

The elongations and compressions on short distances were measured by J. E. Howard for numerous locations near the pin bearings by the use of standard Brown & Sharpe strain gages. Each gage had two conical steel points inserted in special holes of about 1-16 in. in diameter and about 1-16 in. deep, drilled and reamed with precision in the required positions, thus providing for very exact and unvarying setting of the instruments.

Measurements of the elongations and compressions between points a long distance apart, at the center of the model and in its full length were made by C. Scheidl, Jr., of the Phoenix Bridge Company. He used Olsen standard compressometers. Each had a graduated circular dial and an indicator needle having a horizontal axis with friction bearing intermediate between its journals. The friction bearing consisted of a concentric cylinder with a circumference of ½ in., which engaged a long horizontal steel rod fixed at one end to the test specimen at one of the points between which the elongation or compression was to be measured. The rod was supported intermediately on rollers and the other end rested on the friction cylinder and its weight developed sufficient pressure to produce friction revolving the cylinder and the axis of the needle to correspond with any longitudinal movement of the rod.

The bearings of the needle being firmly attached to the specimen at an accurately measured distance from the attachment of the rod, any longitudinal distortion of the model produced a relative movement of the rod on the axis and caused the latter to revolve. The ratio of the diameter of the friction bearing and the diameter of the dial of the machine being 25 and the dial being graduated to five hundredths, gave a direct reading of 1-1000 in., which could be reduced to 1-10,000 in. by the vernier scale of 1.10.

Application of Instruments

In order to correct eccentricity and irregularity and secure true values of the total distortion of the model a compressometer was applied to each of the four sides of the cross-section and the mean of the results was taken. Each machine was bolted securely to a steel bracket, and the latter was either bolted or riveted to the model. (See Figure 4). For readings taken on the vertical sides the brackets were riveted to the web plates, but for readings taken on the horizontal latticed faces the brackets could not be attached directly to the lattice bars, because the latter, not being directly affected by the compression in a member, are materially displaced by the compression in the web plates, and since they retain at first their original length while the longitudinal distances between their rivets are diminished, the web plates tend to move farther apart. To avoid errors due to this displacement, transverse expansion clips with compensating loops bent in them were riveted to the webs and the brackets attached to them, thus providing against displacement of the instrument by transverse movement of the webs. (See Figure 5).

Brackets were also riveted to opposite extremities of the model, and to them were attached small sheaves over which a fine steel wire was stretched by a weight, thus providing a straight line from which lateral deflections were measured by scale. To secure greater accuracy and uniformity of observation and avoid danger from possible fracture or movement at the time of destruction of the model, the compressometers were read through small telescopes mounted on standards and located at fixed points from 2 to 6 ft. distant from the specimen.

Method of Testing

The models, which were fabricated by the Pennsylvania Steel Company, were carefully inspected and measured and the cross-sectional areas very carefully determined. Brackets were riveted to them to receive the instruments and the alignment wires, and the necessary holes were drilled to locate and receive the strain gages. The fixed head of the machine was locked in the required position, the bearings, castings and pins were adjusted and the model was delivered to the machine by an overhead crane and was centered and adjusted in exact alignment with the axis of the machine. In some cases the weight of the model was counterbalanced by the cantilever beams already described, and in other cases its weight was not supported between the ends. (See Figure 2.)

Extreme care was exercised in making the pin bearings as accurate as possible; this was accomplished by a slight adjustment in the head of the machine. Usually one day was required for preparing the machine and making everything ready for the test, after which from one to three hours were necessary for putting the model in the machine, attaching the instruments to it, setting up the telescopes and making ready for the test. The test itself occupied from one to three hours and required four observers, one gageman, two recorders and about six laborers. Before making the test, sheets were prepared giving all the preliminary data, together with the different gage loads which were to be applied with the corresponding unit loads and the amounts of distortion which it was expected would result.

Deformation Diagrams

The diagrams of deformation shown by the compressometers were plotted, giving straight lines from the origins to the elastic-limit points, beyond which they are, of course, broken lines, with the yield points noted at an arbitrary distance from the straight line, thus giving a graphic representation of the distortion of the specimen under stress. In every case the pressure at 26,000 lb. per sq. inch net for nickel steel 20,000 lb. gross for nickel steel, and 14,000 lb. gross for carbon-steel was released, and as a permanent set had then taken place, the deformation line for succeeding pressures commenced at a point offset from the origin and intersected the first line at the point of elastic limit, thus accounting for the two lines shown in the diagrams.

Duplicate nickel and carbon-steel models, TX_{16}, 34 ft. long, made of two built channels latticed, were marked N and C respectively and corresponded to the upper sections of vertical posts in the center suspended span of the new Quebec Bridge. Each of them was spliced at the center, with cover plates on all sides, and had half-hole pin bearings at both ends. The weight was 7970 lb. and the area computed from it was 34.63 sq. in., being 1.61 per cent light of the nominal area of 35.2 sq. in. Each model was counterbalanced in the testing machine by a 1230-lb. weight applied at the center and at each quarter point to eliminate fiber stress due to the weight of the model. The models had a ratio $L/R=52$. The specimen test results were exactly the same for both nickel and for both carbon-steel pieces.

Strain gages were applied simultaneously to the latticed faces on both sides in nineteen 20-in. spaces on one model, and on both webs, and in eighteen spaces simultaneously on the other model. The compressometer was applied to lengths of 11 ft. 7 in., measured on the latticed faces at the center of the models, and to lengths of 34 ft. measured from center to center of end pins on the web-plate faces. The elastic limit of the body of the member was determined by observations on a short length near the center point, and that of the member as a whole, including its connection details at the ends, was determined by observations on the full length. In each case the initial gage reading was 30. (See Figures 1, 6, 7, 8, 9, 10, 11, 12, 29 and 30).

Engineering Record, July 25th, 1914

ULTIMATE STRENGTH OF CARBON-STEEL MODELS OF QUEBEC BRIDGE MEMBERS

Records and Diagrams of Destructive Tests; Anchor Arm Lower Chord Models Buckled under Unit Load of 50,886 Pounds.

Among the eighteen tension and compression tests of models to a scale, about one-quarter the natural size of the members of the trusses in the 1800-ft. span of the new Quebec Bridge which have recently been made there are four sets on carbon-steel sections of the compression lower chords L_{10-12} in the anchor arm, located as shown by the diagram published in the Engineering Record of March 21, page 333. These members have the same position in the truss as those which failed in the Quebec Bridge collapse several years ago. In the new bridge structure they are proportioned for heavier total loads and for smaller unit stresses, and their design differs materially from that of the corresponding members in the former structure, which had four deep vertical webs connected only by end diaphragms and by top and bottom chord latticing.

The models of the new bridge members have been made and tested in the interest of the accepted design which has been prepared after investigation of similar members in the other largest recent spans and may be considered as the latest development in compression members to resist excessively heavy stresses in long-span trusses.

Models

The models, about 30 in. wide, 18¾ in. deep and 18 ft. 9 in. long, weighed from 7610 to 7650 lb. each and were marked TX_{13}-A, B, 1 and 2. They had four deep vertical webs connected by full-length horizontal longitudinal latticing and tie plates on the center line and on the top and bottom flanges. They had ratio of $L/R = 38$ and a nominal cross-sectional area of 70.65 sq. in., which, however, owing to the inaccuracy of rolling the plates and angles, was reduced to an actual area of about 69 sq. in., making the sections a little more than 2 per cent light. Specimen tests made from the plates and angles showed an elastic limit varying from 39,780 to 42,700 lb. They had half-hole bearings in both ends of the webs, which were reinforced to give 52 sq. in. of bearing for the 6½-in. pins.

The accompanying illustrations are of special interest in that they include the first published details of the bottom chords of this great span. All of the models were symmetrical about their center transverse axes and those marked A were made with both ends exactly like the left-hand end of the illustration, with the flanges of the center webs connected by lattice bars. The models marked B were exact duplicates, except that they were made with both ends like the right-hand end of the illustration, with the flanges of the middle webs connected by tie plates. The models were supported by end pins alone, without intermediate counterweights to carry part of the weight. (See Figure 13).

General Results

The elastic limit of the models, taken as a whole, varied from 16,866 lb. to 22,441 lb., and in all except the last case was much lower than the elastic limit determined on a long length measured in the center of the member. One model failed with a maximum unit load of 39,359 lb. The other three models endured without failure several repetitions of the maximum load of the 2,800,000-lb. capacity machine. Afterward they were weakened by holes drilled through the webs near the center of the piece and failed under loads of from 43,060 lb. to 50,886 lb. per square inch computed on the reduced sections.

Former Similar Tests

A very interesting comparison can be drawn between the results of these tests and those published in the Engineering Record of Nov. 19, 1910, page 564, on nickel steel models of the official design of the same bridge, which has since been superseded by the somewhat modified contract design now being fabricated. The nickel steel models were about twice as long as these and had a considerably smaller cross-sectional area. They developed a maximum elastic limit of 43,947 lb. in the full-length measurements and had an ultimate strength beyond the capacity of the testing machine, so that it was necessary to weaken them also by boring holes, which reduced the cross-sectional area so that failure was finally produced by a maximum ultimate load of 63,990 lb. computed on the reduced section.

Manner of Failure

Model TX13A-1 failed by lateral deflection in the planes of the webs and by buckling of the webs near the center of the piece. Diagram of locations of the strain gages and of the compressometers is here given. The locations of the compressometers were the same for all of the tests in this set, except that for the models marked B the distance measured on the latticed faces was only 127.5 in.

Model TX13A-2 was compressed to the full capacity of the machine, which produced a stress of 40,657 lb. without failure. Afterward two 1-in. holes were bored through each of the four webs near the center of the piece, reducing the section from 68.87 to 64.87 sq. in., and the same maximum total load was applied three times at high speed, producing a unit stress of 43,164 lb. per sq. in., and on the third application caused the failure of the model by a downward deflection and the buckling of the webs near the center. The strain gage locations were all made on one latticed face of the model, none of them being made on the webs.

Model $TX_{13}B$-1 endured without failure the full load of the machine, producing a compression of 40,565 lb., after which two 1-in. holes were bored through all four of the webs near the center of the model, reducing the section area from 69.026 to 65.026 sq. in., and the same maximum load was applied again seven times at high speed, producing a stress of 43,060 lb. and finally causing the failure of the piece by bending at the center in the plane of the webs and by the buckling of the webs near the center. The strain gages were applied at ten points on the webs on both sides of the model and at eight points at the upper latticed face.

Model $TX_{13}B$-2 was loaded to the full capacity of the machine, gage 867.7, producing an ultimate compression of 40,565 lb. on the full section of the piece, which it endured without failure. The pressure was released and two 1-in. holes were bored through all four webs, reducing the section area 4 in., and the same loading was repeated ten times. As this did not produce failure, two more holes were bored and the same loading repeated several times. Finally the section was reduced 14 in. by boring six holes through the two south webs and eight holes through the two north webs and the same load again slowly applied and maintained for 10 min., producing a compression of 50,886 lb. per square inch on its reduced cross-section of 55.026 sq. in. After 10 min. application this load caused the failure of the model by bending transversely at the center in the plane of the webs, which were buckled somewhat at the location of the holes. (See Figures 1, 14, 15, 16, 17, 33 and 34).

ENGINEERING RECORD, NOVEMBER 14TH, 1914

ULTIMATE STRENGTH OF CARBON AND NICKEL STEEL MODELS OF QUEBEC BRIDGE MEMBERS

Tension Tests of Reinforced Steel Plates and Alternating-Stress Members, and Compression Tests of Four Struts.

Tests recently made on eighteen large-scale models of members of long-span trusses of the new Quebec Bridge included comparative tests of nickel and carbon-steel models, described in the issue of March 21, page 333; compression tests of carbon-steel models of the lower-chord pieces with four webs, published in the issue of July 25, page 110, and tests of three other sets of models herein described. These latter were all made on models which, like those previously discussed, were tested to destruction with an accurate and elaborate system of deformation measurements recorded for the study and analysis of the design.

Tension tests were made on a set of four large nickel steel plates with riveted reinforcements for pin bearings and on two nickel steel riveted members of a type adapted to resist alternating stresses. A third group of tests were made of H-shaped carbon-steel struts in compression. The distortion of the metal was measured on the members considered as a whole, and locally on the webs and latticed faces and around the pin bearings, to investigate the construction and detailing.

The tension plates developed a maximum elastic limit of 44,452 lb. and a maximum ultimate strength of 79,999 lb. in one of the two pieces broken. The other two pieces endured strains nearly as great without fracture. The alternating stress members under tension stress developed a maximum elastic limit of 41,526 lb., and an ultimate strength of 73,294 lb., breaking in the body of the pieces. The H-shaped compression pieces developed a maximum elastic limit of 21,979 lb., which, as in the case of the previous compression tests, was considerably higher in most cases than the elastic limit determined by measurements on the full lengths of the pieces. The maximum ultimate strength was 35,146 lb.

Tests of Alternating Stress Members

Two nickel steel models adapted to resist either tension or compression were tested to destruction under tension, developing, like most of the compression models, a higher elastic limit for a measured length in the body of the piece than for the full length. The models were marked $T_{10}N$-1, and $T_{10}N$-2, and were duplicates, each being made of two built channels about 24 in. deep and 40 ft. long with their flanges turned in and latticed and their webs bored for 8-in. pins about 37 ft. 9 in. apart. The models had actual gross and net sectional areas, figured from the weight, of 32.51 sq. in. and 26.83 sq. in respectively. The weight was 7110 lb. Test specimens of the angles and plates showed elastic limits of 58,500 and 62,880 lb. respectively, and ultimate strengths of 93,000 and 93,360 lb. The strain gages were applied in 10-in. lengths at twenty-five locations on the flanges and webs and the compressometers were applied to measured lengths of 228 in. in the middles of the latticed faces, and to pin holes 37 ft. 9 in., center to center.

Both models failed by breaking transversely through the body.

(See Figures 1, 18, 19, 20 and 31).

Tests of Plates

Models A_2N-1, 2, 3 and 4 were nickel steel members made of single plates nominally 36 in. wide, ⅝ in. thick, and 30 ft. long, with reinforcement plates riveted to them for the bearings of two 12-in. pins 26 ft. 8 in. apart on centers. The specimen tests gave elastic limits of 62,880 lb. and 66,520 lb., and ultimate strengths of 93,300 lb. and 89,360 lb. Three of the pieces weighed 3310 lb. each, and one of them weighed 3315 lb. The actual gross area computed from the weight was 22.38 sq. in. Strain gages were applied to one face of the model in twenty-two different locations at the end and in the middle of the piece, to measure the elongation in 10-in. lengths, and four compressometers were applied on both edges and on the center line to measure the extensions in a length of 200 in.

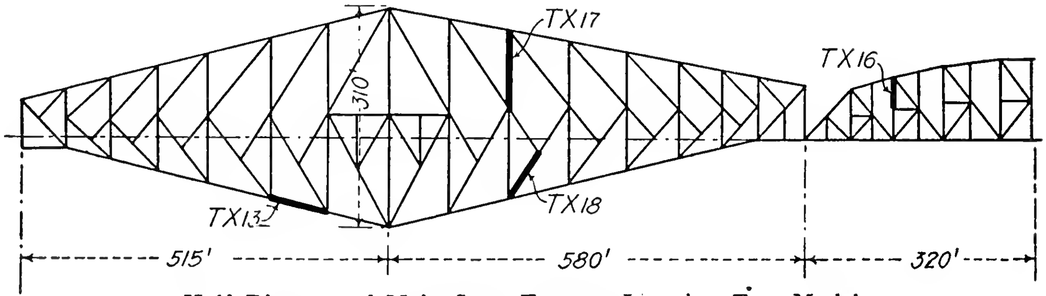
Models A_2N-3 and A_2N-1 failed by fracture through transverse rows of rivets in end reinforcement plates. Models A_2N-2 and A_2N-4 failed by dishing around the pin holes. (See Figures 1, 21, 22, 23 and 32).

Compression Tests of H-Shaped Models

The four H-shaped carbon-steel models marked TX_{17}-1 and 2 and TX_{18}-1 and 2, represented respectively vertical and inclined posts in the cantilever arms of the trusses. They were located as indicated in the general diagram published in the issue of March 21. Each was made of two built I-beams with latticed flanges and solid webs connected by an I-shaped horizontal latticed diaphragm riveted to them on the center longitudinal lines. At the center point they were spliced by web and flange cover plates, and at the ends the vertical webs were reinforced and had half-hole pin bearings. All of them were about 21 in. wide and 21 in. deep over all. The TX_{17} models had a nominal cross-sectional area of 42.75 sq. in., a weight of 8950 lb., a length of 46 ft. 6 in. center to center of pin holes, and a ratio L/R of 78. The TX_{18} models had a nominal area of 56.65 sq. in., weighed 9550 lb. each, were 34 ft. 6 in. long center to center of pin holes and had a ratio L/R of 58

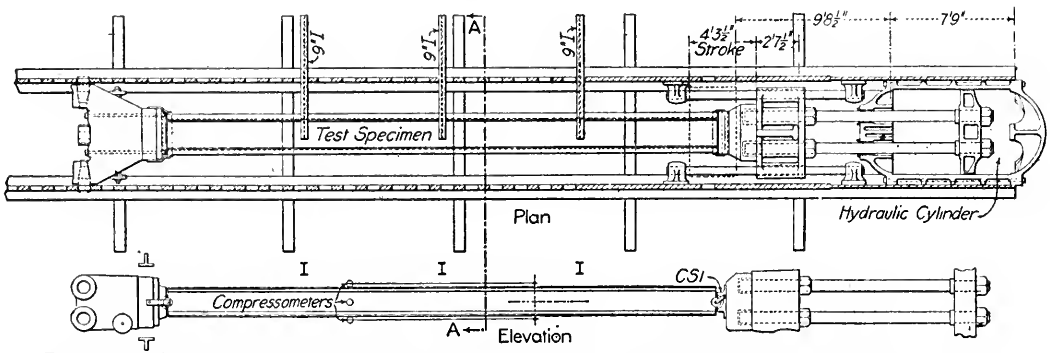
The TX_{17} models were counterbalanced by 2000-lb. weights attached to them at the quarter and center points, and the TX_{18} models were similarly counterbalanced by 870-lb. weights. The distortions produced by the test loads were measured by strain gages applied to the lattice bars, flanges and webs of the models on short measured lengths, and by the compressometers applied to measured lengths of about 19 ft. on the latticed faces and to the full length between pin centers on the webs of the models.

All of the pieces failed by bending at the centers, accompanied in two cases by comparatively small buckling of the webs near the center. In all cases but one the elastic limit was higher for the measured length in the center of the model than for the full length between pin centers. The maximum elastic limit developed was 21,979 lb. and the maximum ultimate strength, 35,146 lb. (See Figures 1, 24, 25, 26, 27 and 28).

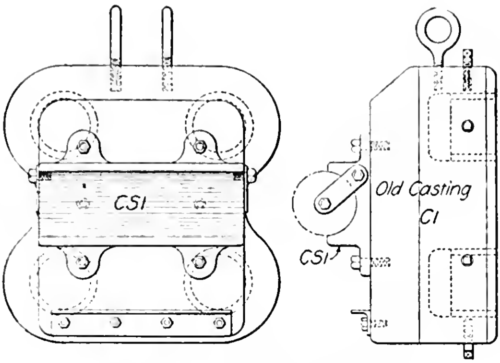


Half Diagram of Main Span Trusses, Locating Test Models

TEST 7.—Figure 1.

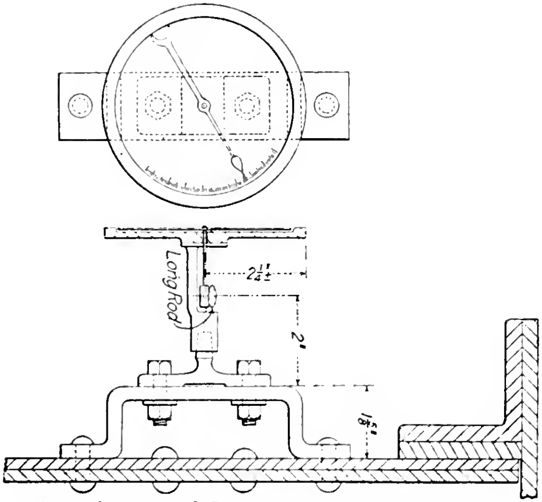


Position of Model and Counterbalance Levers in Testing Machine. TEST 7.—Figure 2.



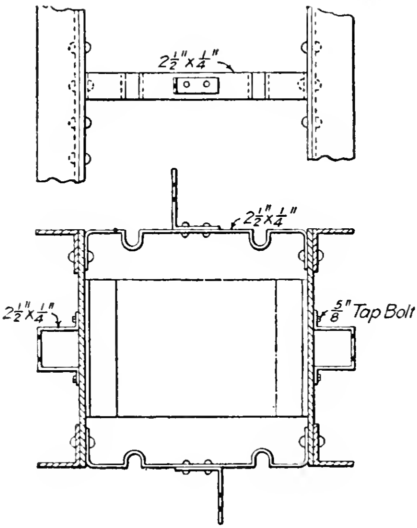
Casting to Receive Models

TEST 7.—Figure 3.

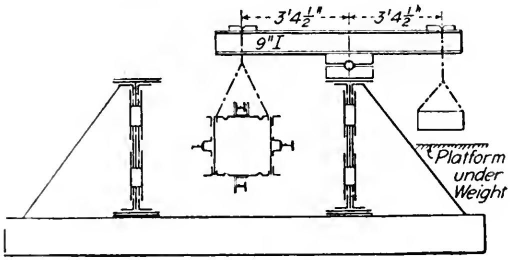


Attachment of Compressometer to Model.

TEST 7.—Figure 4.



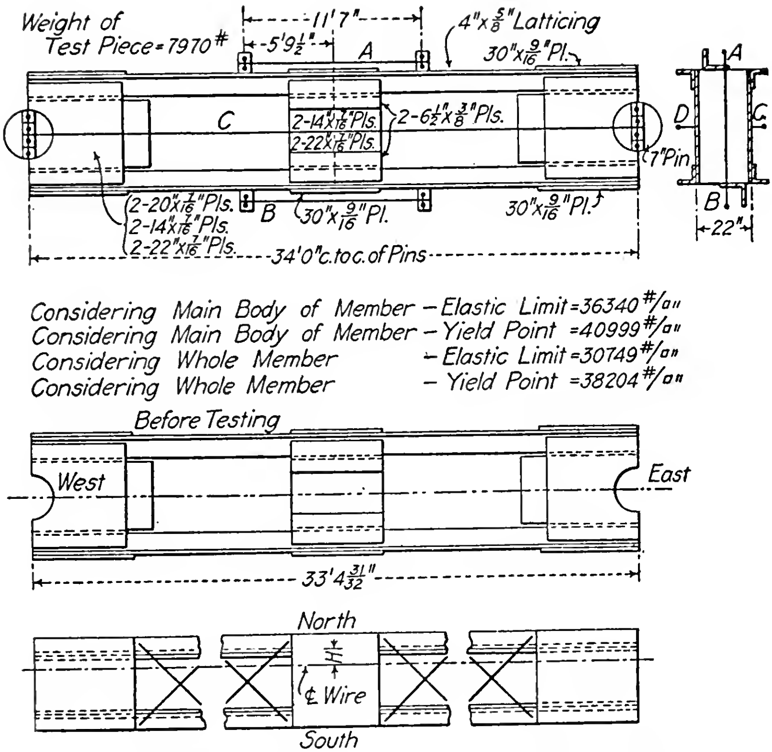
Attachment of Brackets for Compressometers



Section A-A

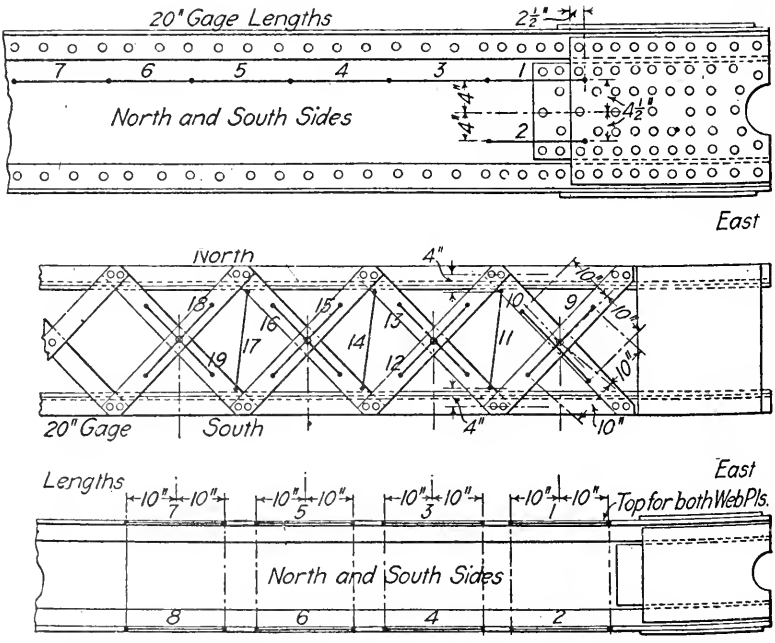
Arrangement of Compressometer Brackets and Counterbalance Levers.

TEST 7.—Figure 5.



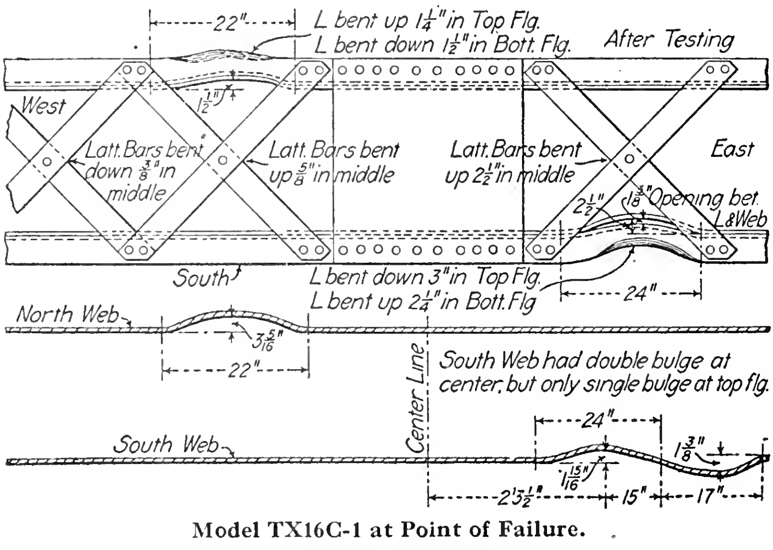
Models TX16 and Positions of Compressometers

TEST 7.—Figure 6.



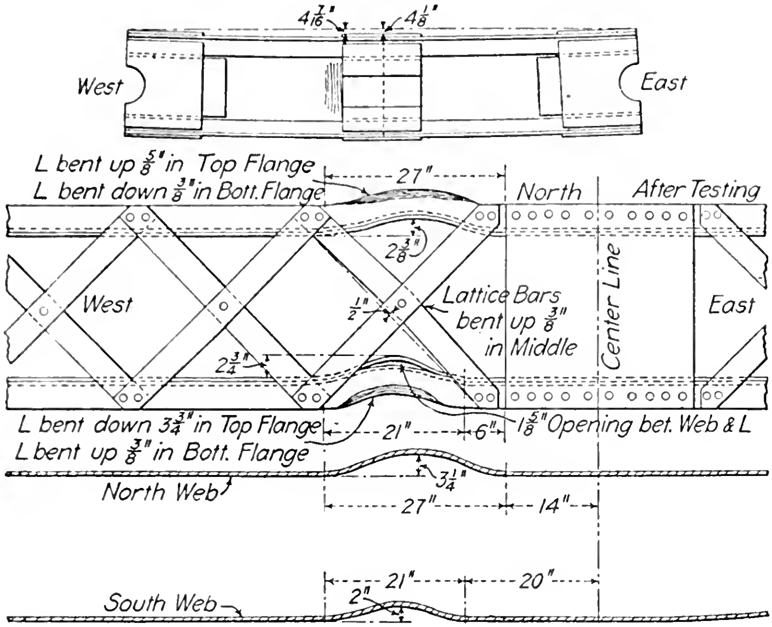
Location of Strain Gage Measurements on Models TX16N—1 and TX16N—2

TEST 7.—Figure 7.



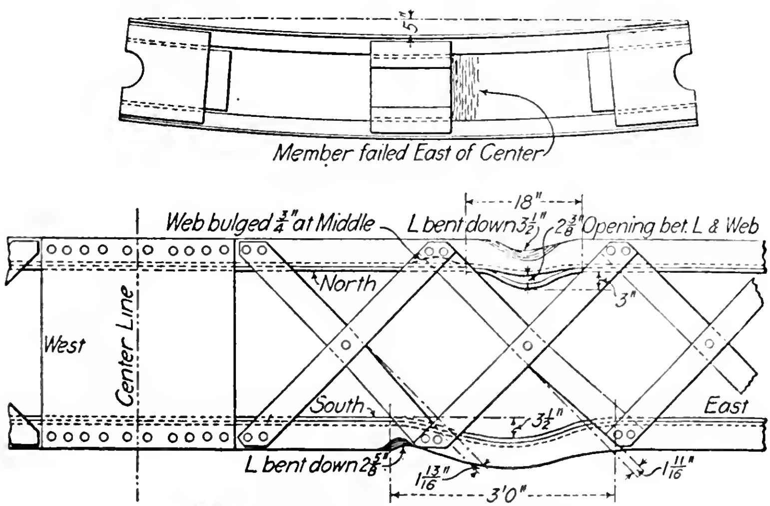
Model TX16C-1 at Point of Failure.

TEST 7.—Figure 8.



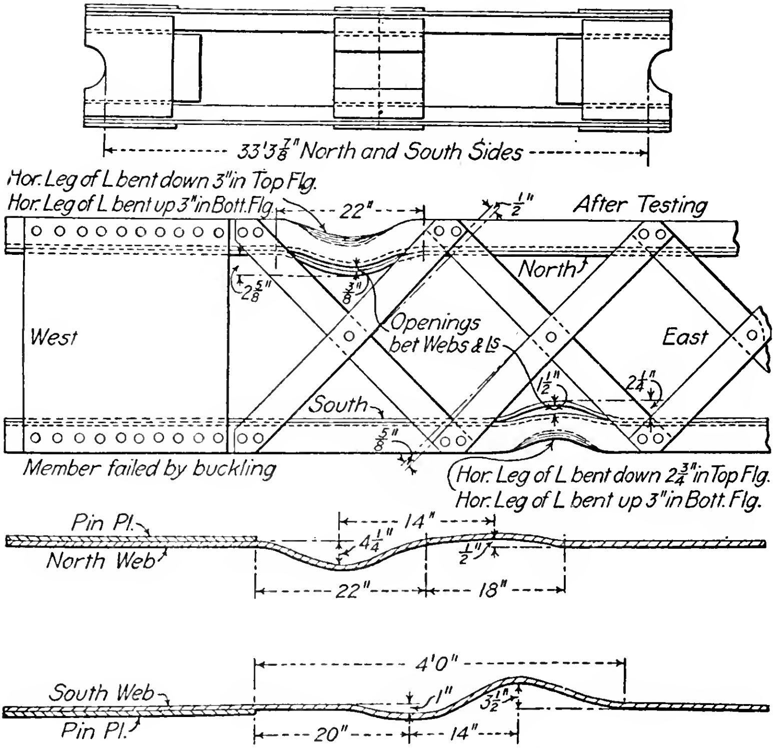
Model TX16C– 2 at Point of Failure, After Test

TEST 7.—Figure 9.

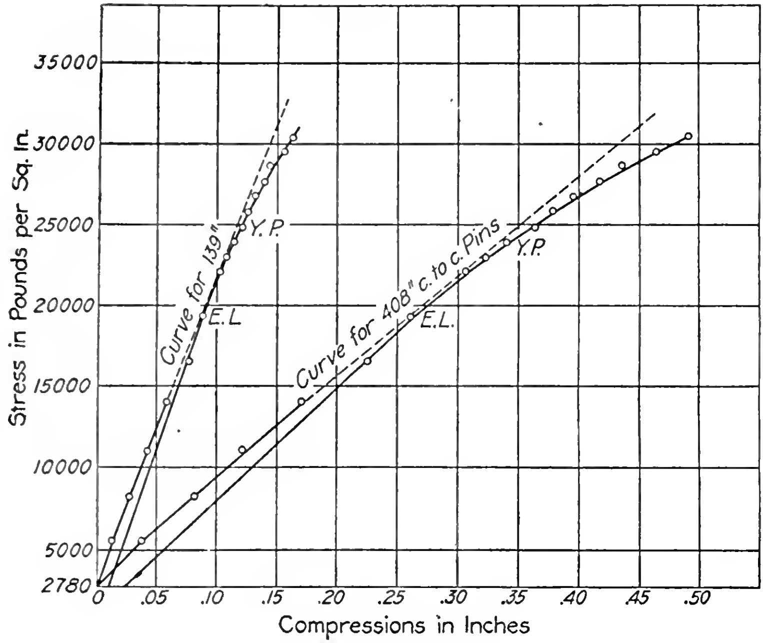


Model TX16N-1 at Point of Failure.

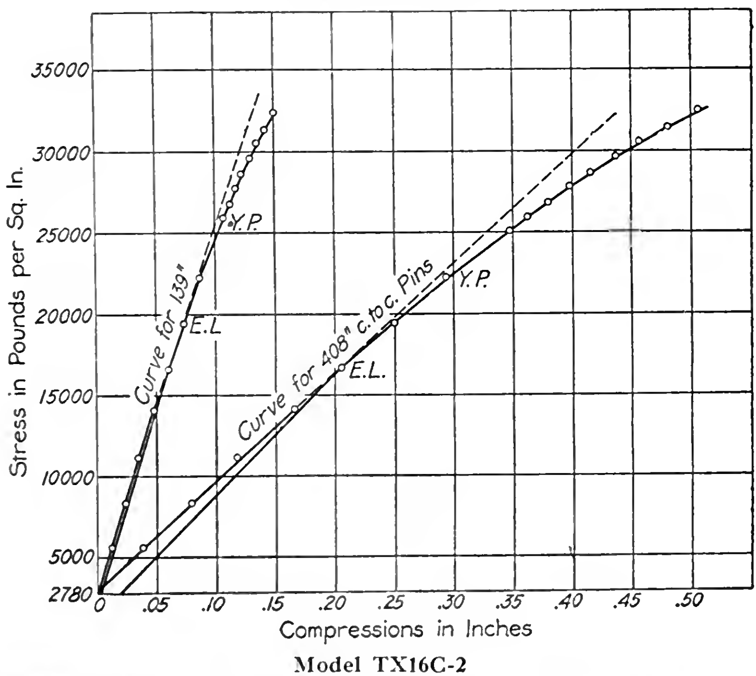
TEST 7.—Figure 10.



Model TX16N—2 at Point of Failure, After Test
TEST 7.—Figure 11.



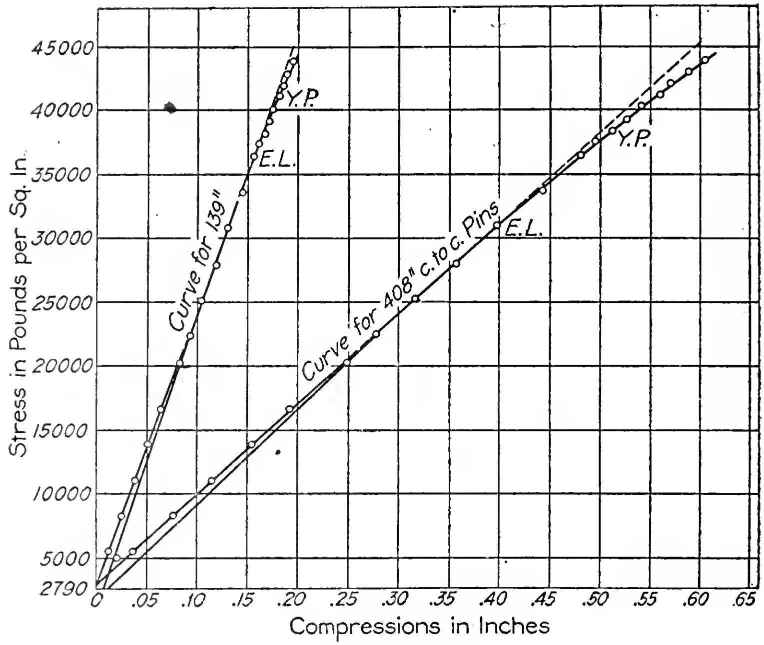
Model TX16C-1



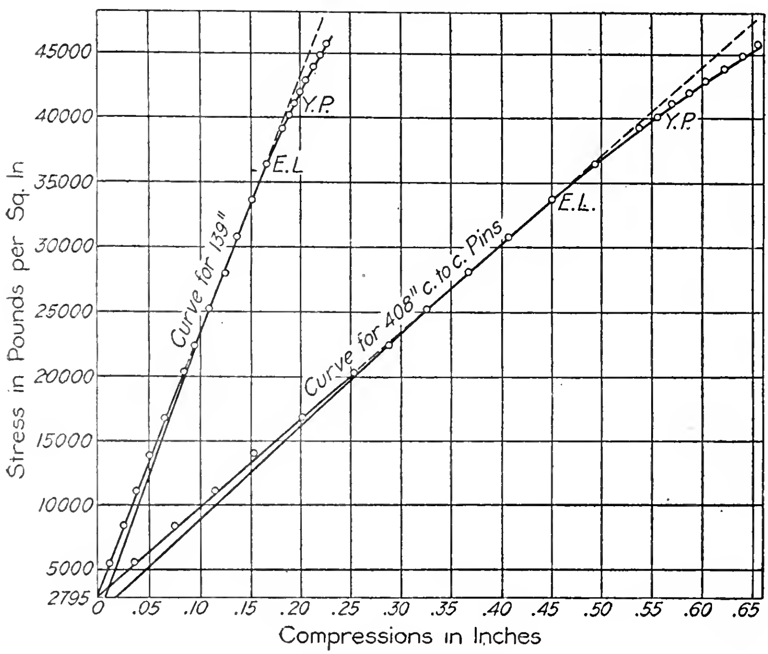
Model TX16C-2

Stress-Deformation Diagrams of Phoenix Bridge Company's Readings.

TEST 7.—Figure 12.



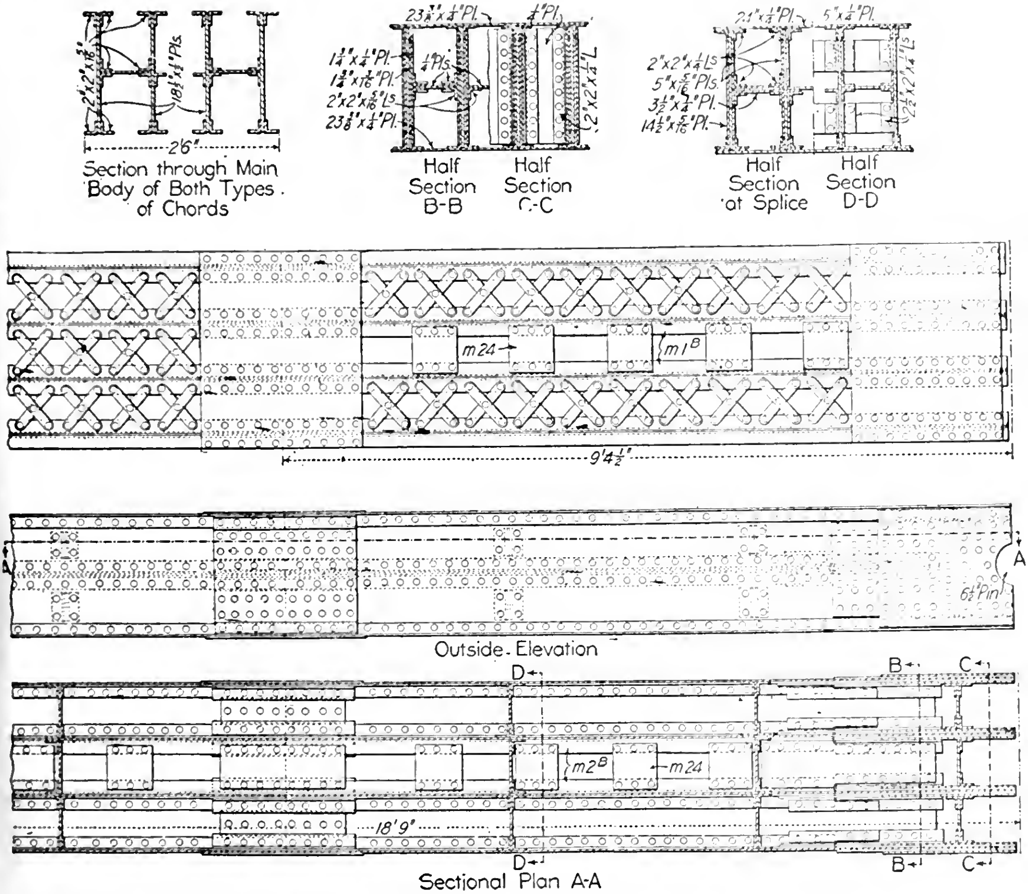
Model TX16N-1



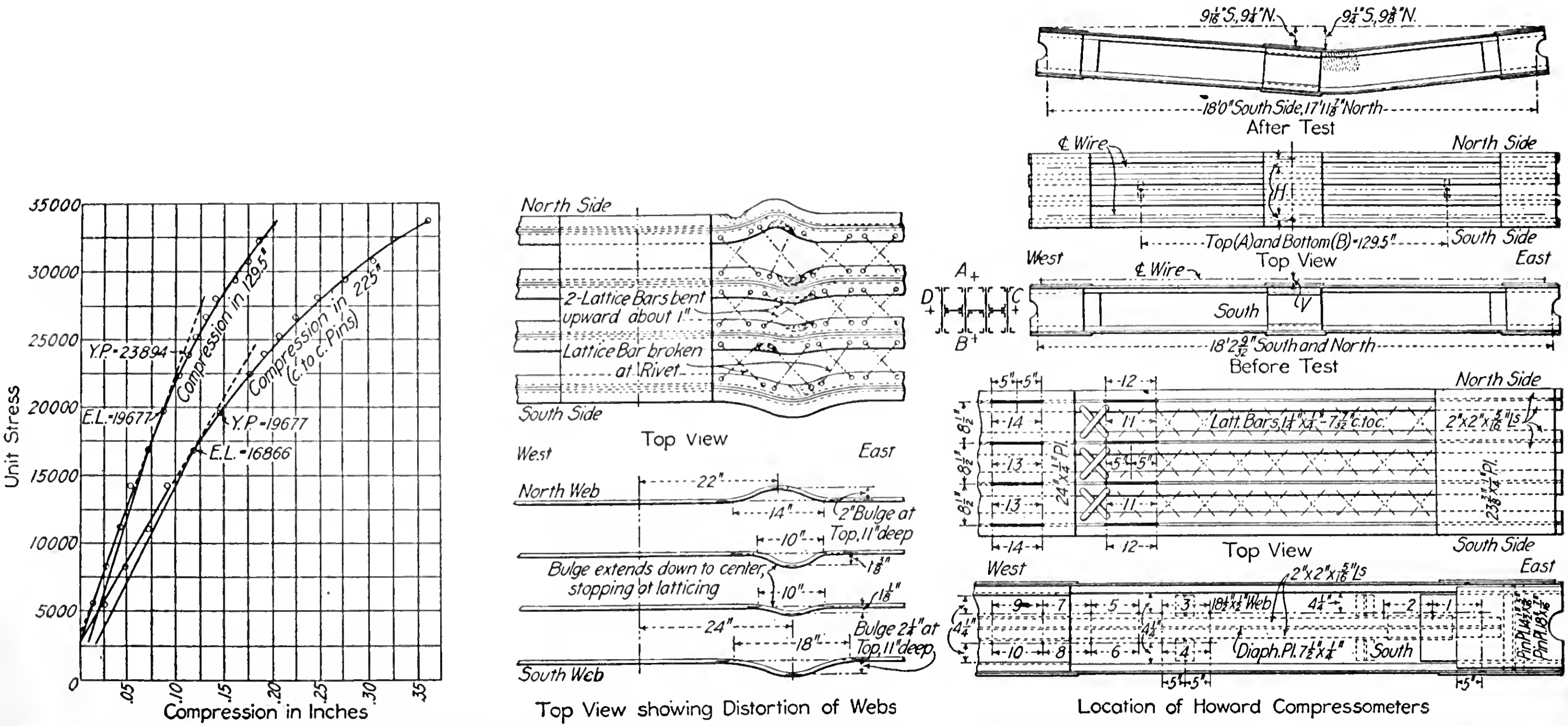
Model TX16N-2

Stress-Deformation Diagrams of Phoenix Bridge Company's Readings.

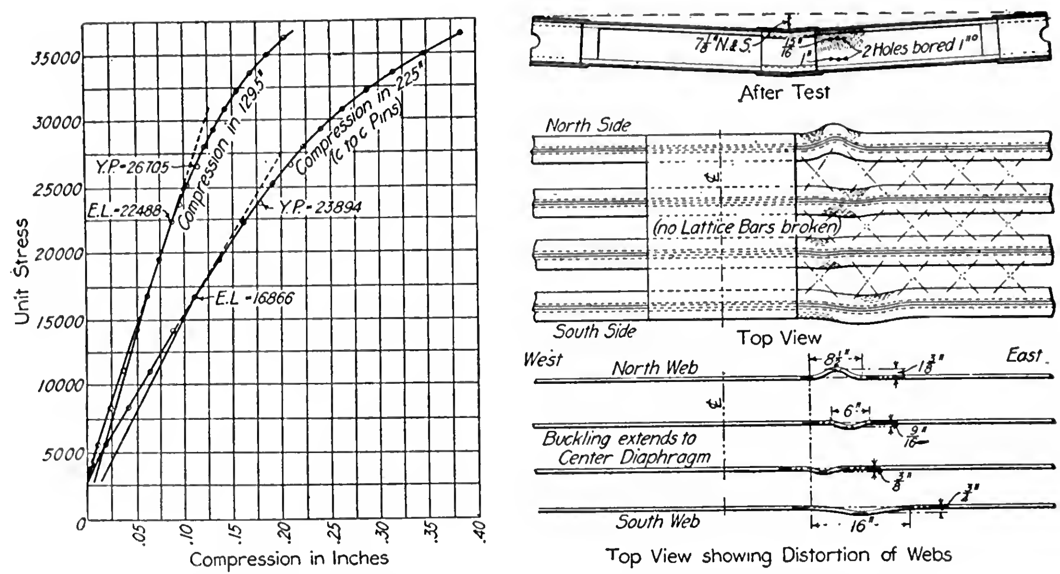
TEST 7.—Figure 12a.



Detail of Carbon-Steel Model of Anchor Arm Lower-Chord Section. TEST 7—Figure 13.

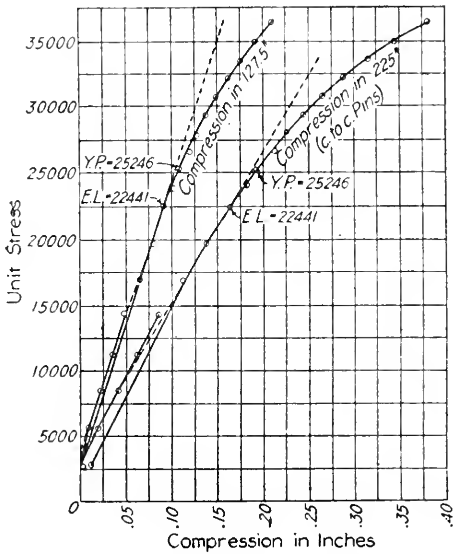


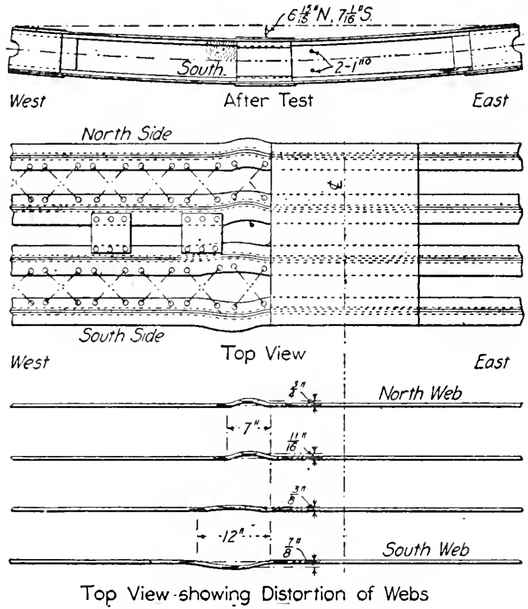
Compression Diagram and Manner of Failure of Model TX13A—1 and Location of Compressometers. TEST 7.—Figure 14.



COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13A-2

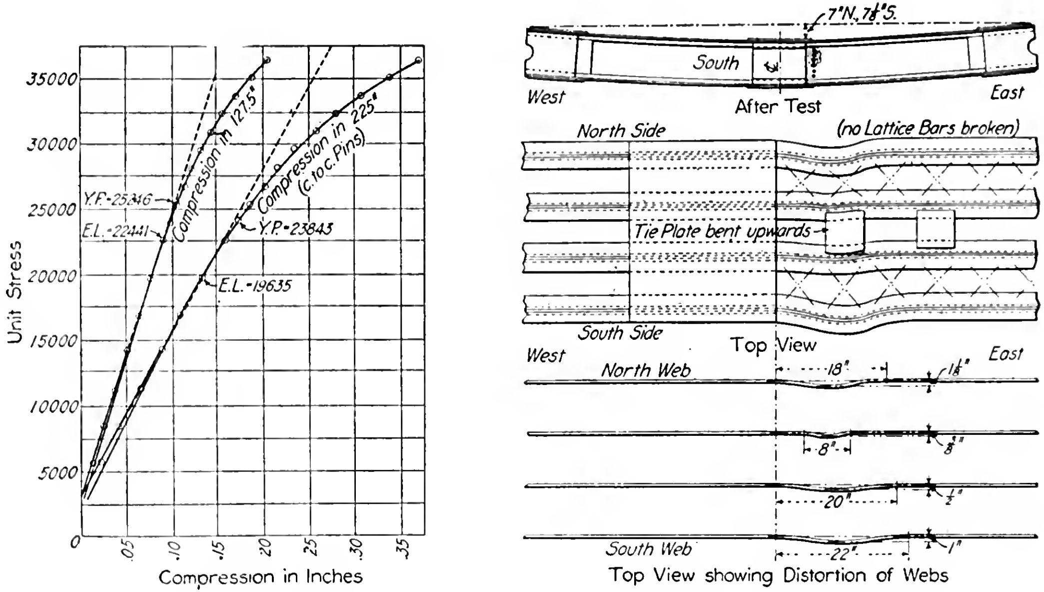
TEST 7.—Figure 15.





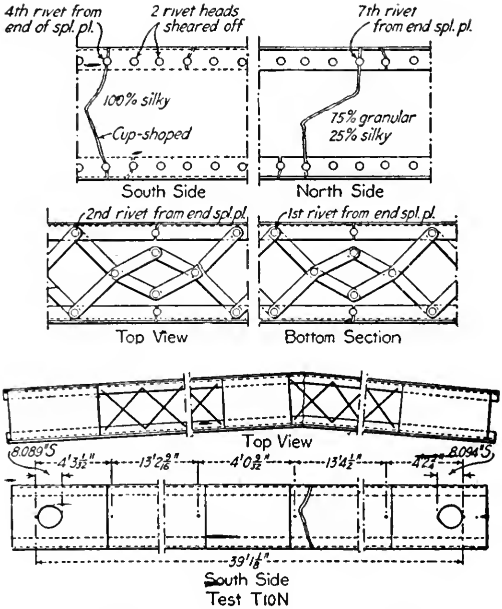
COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13B-1

TEST 7.—Figure 16.



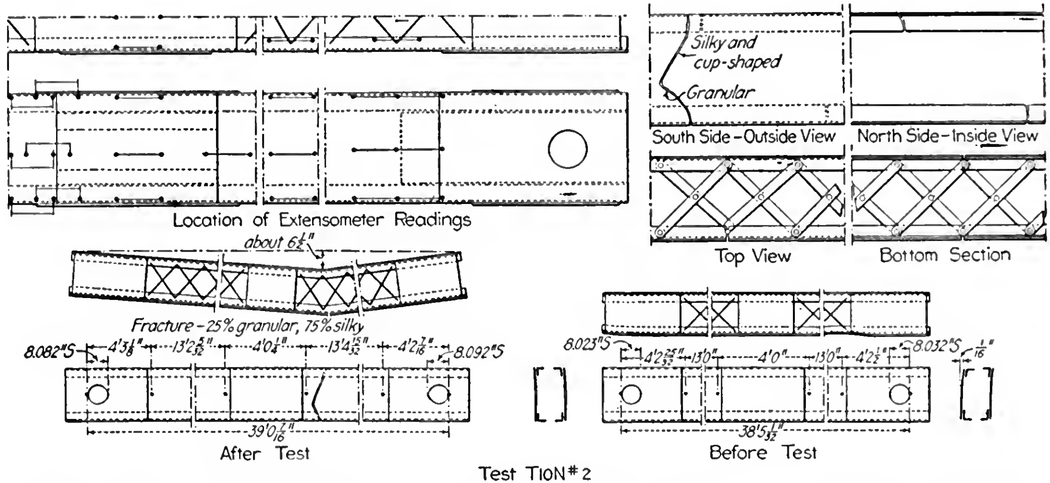
COMPRESSION DIAGRAM AND MANNER OF FAILURE OF MODEL TX13B-2

TEST 7.—Figure 17.

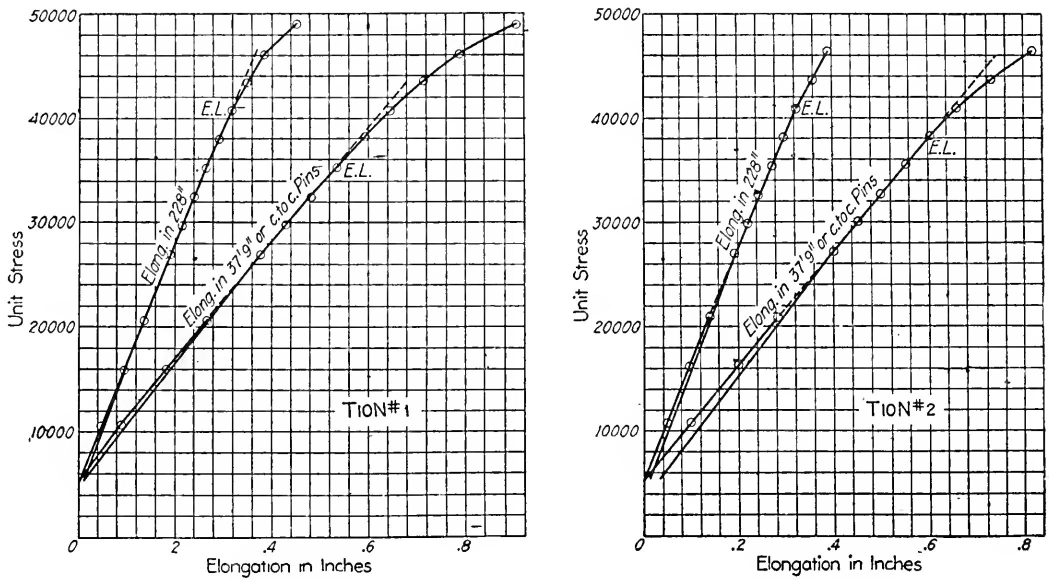


DEFORMATION OF MODEL T10N-1

TEST 7.—Figure 18

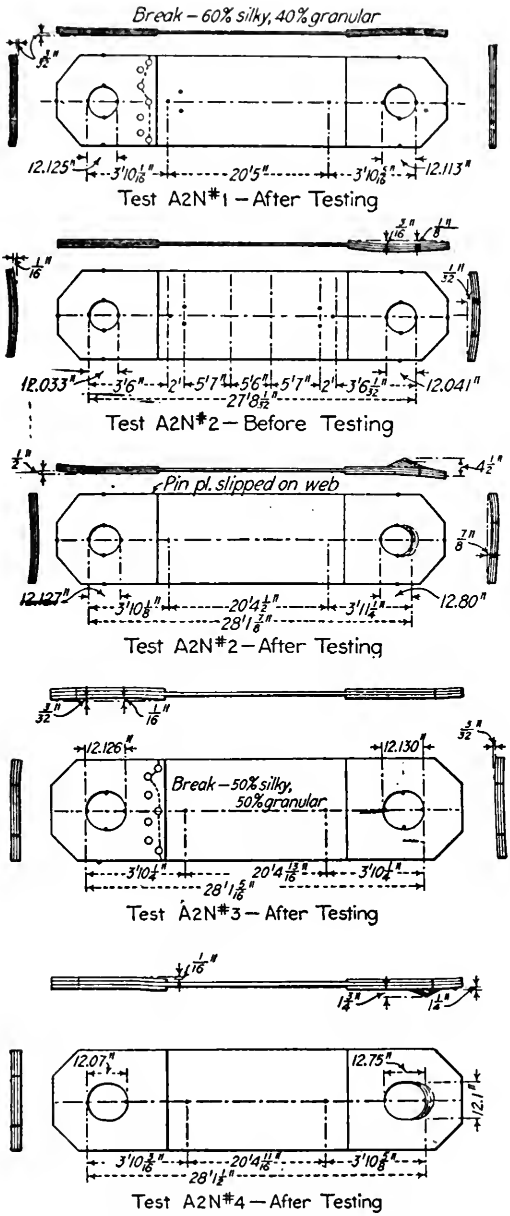


Deformation of T10N-2 and Location of Extensometer Reading. TEST 7.—Figure 19



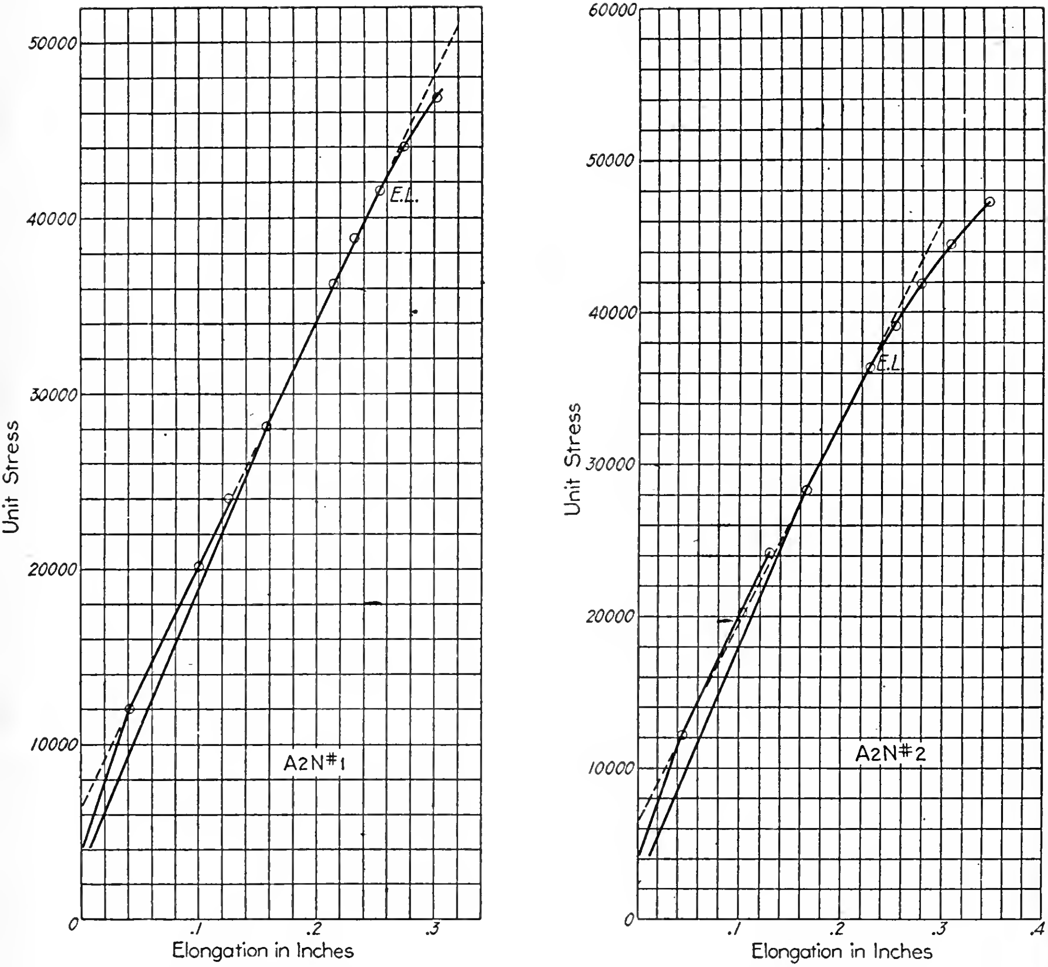
STRESS-DEFORMATION DIAGRAMS OF MODELS T10N-1 AND T10N-2

TEST 7.—Figure 20



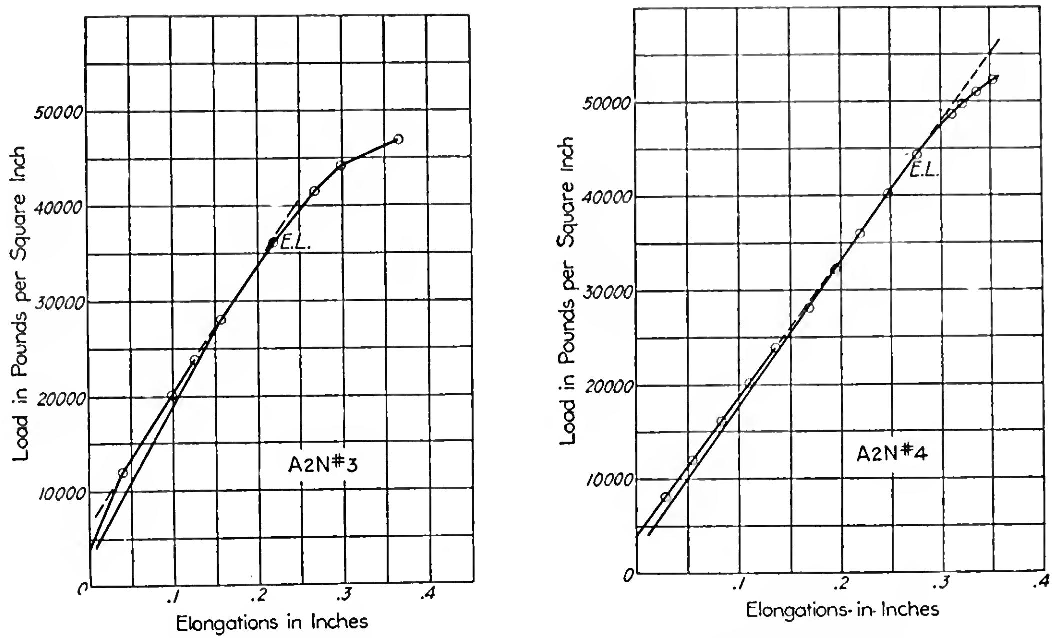
Details of Failures of Plates.

TEST 7.—Figure 21.



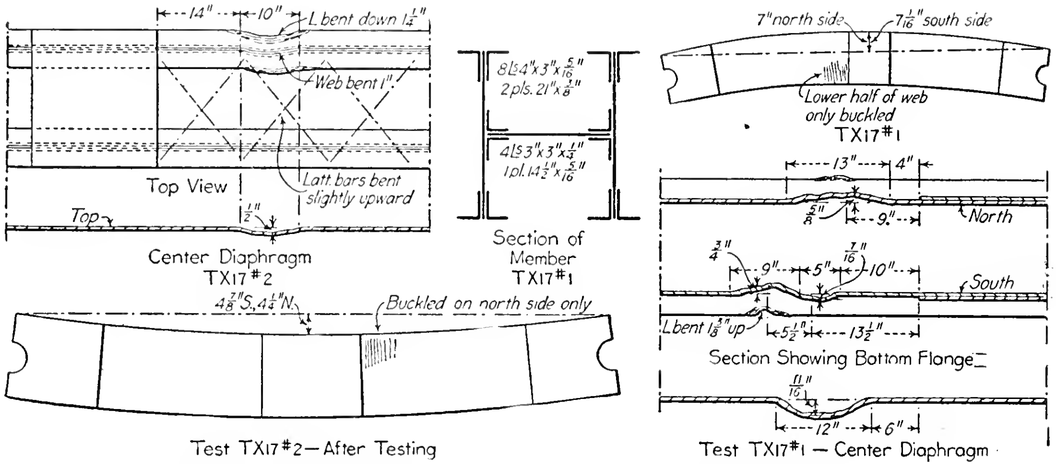
STRESS-DEFORMATION DIAGRAMS OF MODELS A2N-1 AND A2N-2

TEST 7.—Figure 22



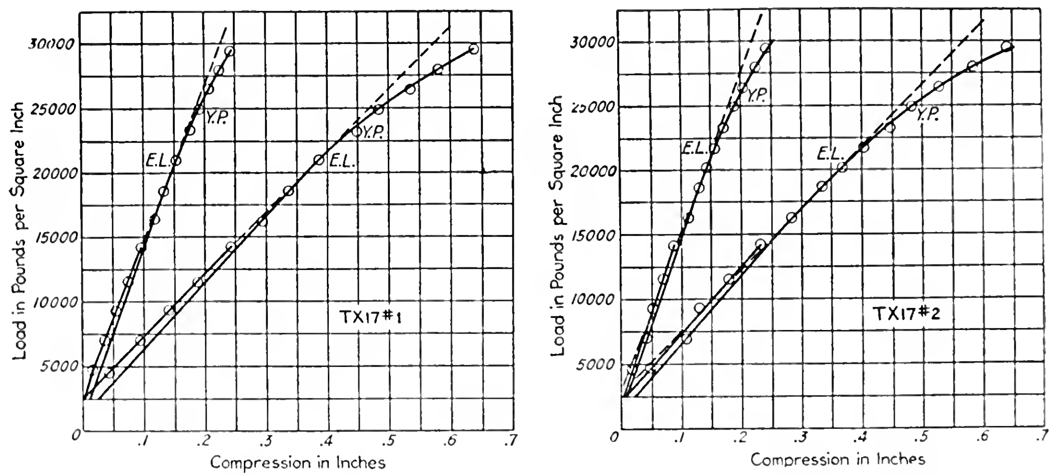
STRESS-DEFORMATION DIAGRAMS OF MODELS A2N-3 AND A2N-4

TEST 7.—Figure 23



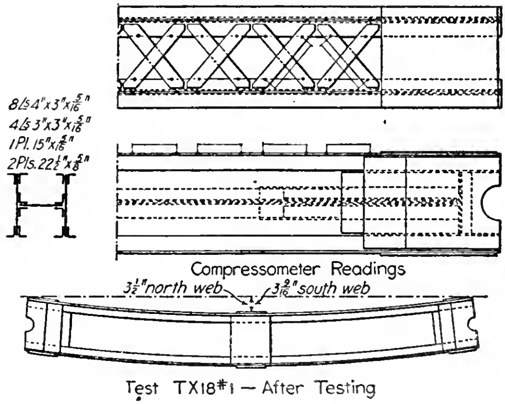
Sketches showing Deformation of TX17-2 and TX17-1 during test.

TEST 7.—Figure 24



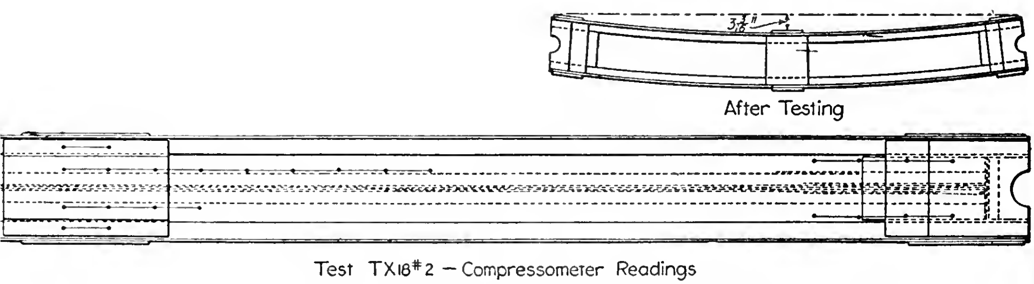
STRESS-DEFORMATION DIAGRAMS FOR MODELS TX17-1 AND TX17-2

TEST 7.—Figure 25



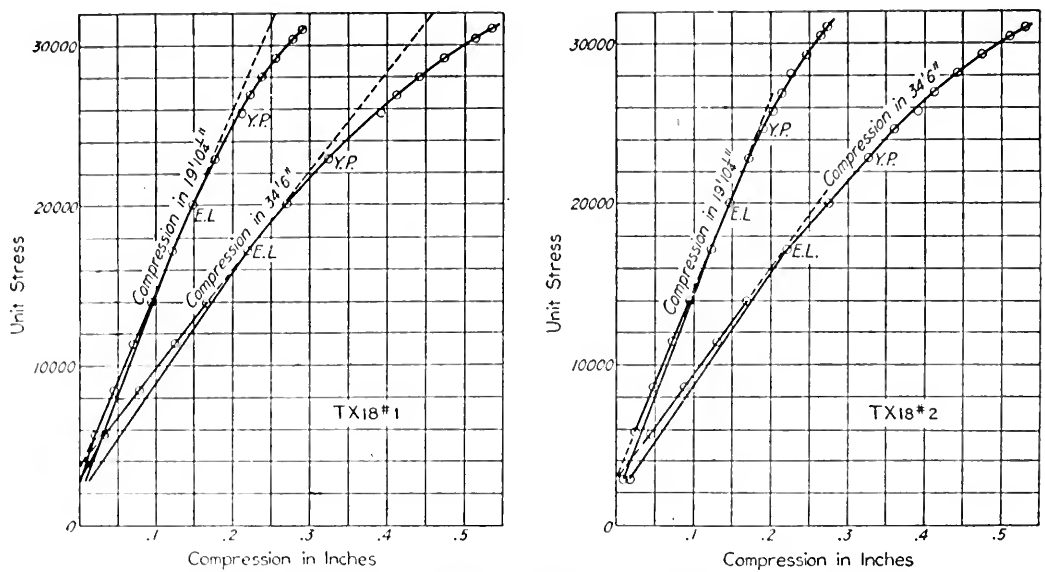
STRUCTURE, LOCATION OF COMPRESSOMETER READINGS AND DEFORMATION OF MODEL TX18-1.

TEST 7.—Figure 26

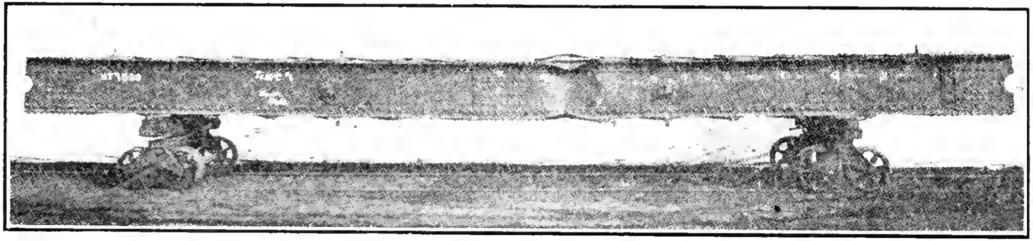


DEFORMATION OF TX18-2 AND LOCATION OF COMPRESSOMETER READINGS

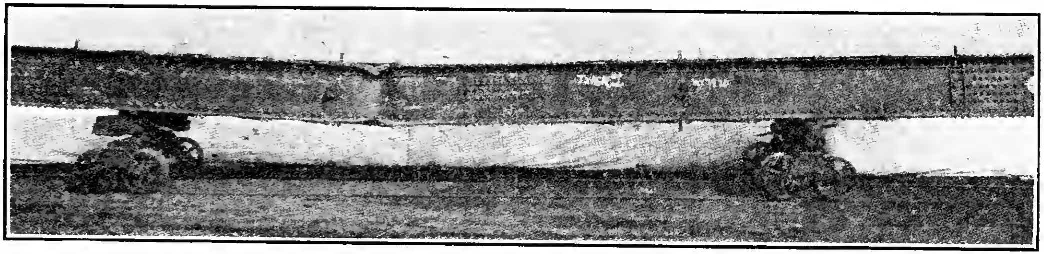
TEST 7.—Figure 27



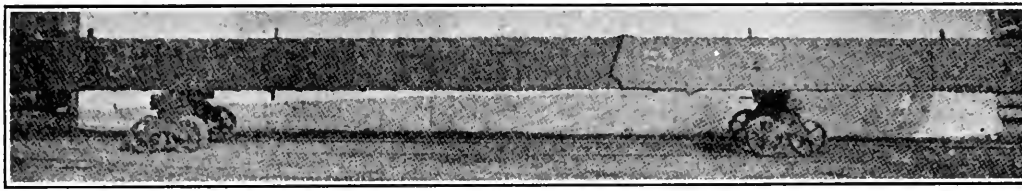
Stress-Deformation Diagrams for Models TX18-1 and TX18-2 TEST 7.—Figure 28



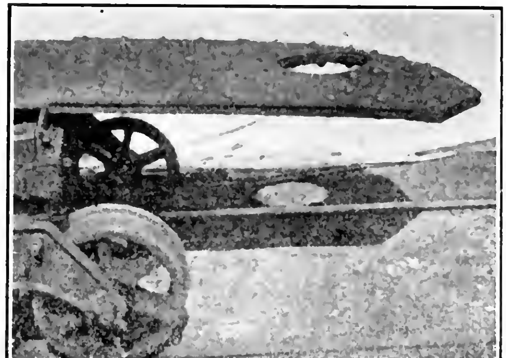
Model TX16C-1 after Failure. TEST 7.—Figure 29.



Model TX16N-1 after Failure. TEST 7.—Figure 30

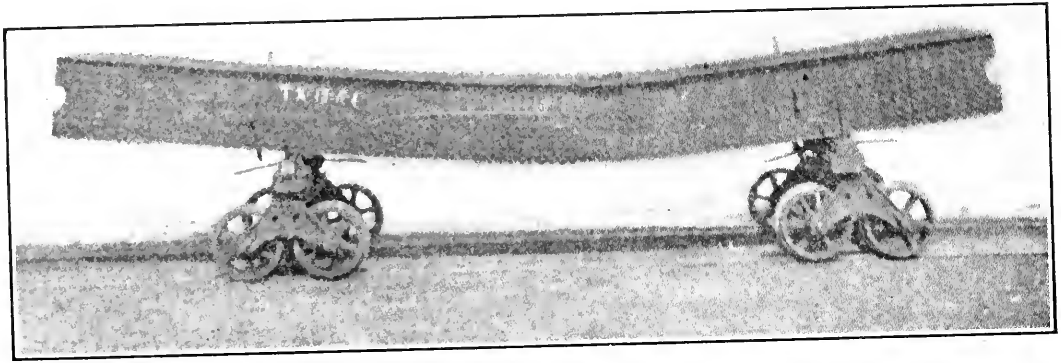


Model T10N-2 after Failure. TEST 7.—Figure 31



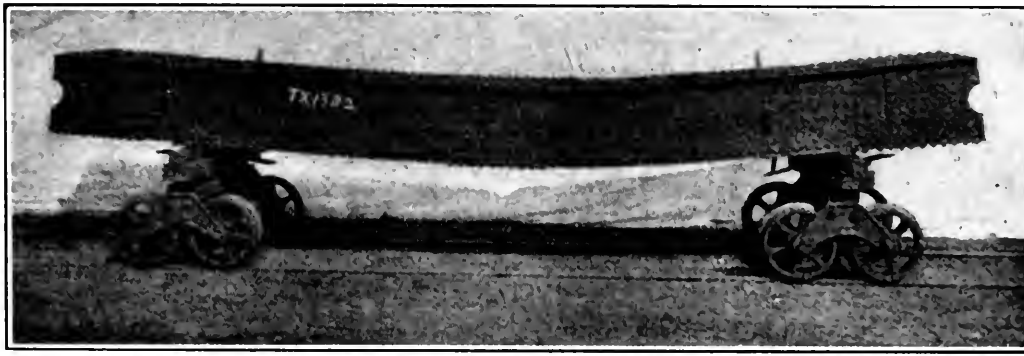
End of Plate A2N-4 after Failure.

TEST 7.—Figure 32.



TEST 7.—Figure 33

Model TX13A-1 after Failure.



Model TX13B-2 after Failure. TEST 7.—Figure 34

Tests of Vertical-

Material	Mark	Elastic Limit		Ultimate load, lb. per sq. in.
		At center, lb. per sq. in.	In full length, lb. per sq. in.	
Nickel steel	TX16N-1	36,340	33,545	47,522
Nickel steel	TX16N-2	36,340	30,749	51,624
Carbon steel	TX16C-1	19,457	16,667	36,597
Carbon steel	TX16C-2	19,457	19,457	33,354

Tension and Compression Tests of Carbon

Mark	Kind of steel	Per cent elongation in 20 ft.	Elastic Limit	
			In portion at center	In full length
T10N-2	Nickel	....	41,526	38,757
T10N-1	Nickel	....	41,526	35,989
A2N-1	Nickel	2.1	....	41,820
A2N-2	Nickel	1.875	....	36,424
A2N-3	Nickel	2.0	....	36,422
A2N-4	Nickel	1.95	....	44,452
TX18-1	Carbon	....	20,133	17,257
TX18-2	Carbon	....	20,183	17,257
TX17-1	Carbon	....	21,194	21,194
TX17-2	Carbon	....	21,979	20,409

Compression Tests of Carbon Steel

Mark	Elastic Limit		Ultimate strength	Percentage model is light of nominal weight
	In portion of member at center	In full length		
	Pounds per square inch			
TX13A1	19,677	16,866	39,359	2.51
TX13A2	22,488	16,866	43,164	2.51
TX13B1	22,441	22,441	43,060	2.29
TX13B2	22,441	19,635	50,886	2.3

Table 1.
Post Models

Manner of failure	Elevation of center point at commencement of test	Date of tests, 1913	Time of tests readings,	
			First	Last
Web buckled near center	1/64 in. high	Mar. 3	9.43 a.m.	1.15 p.m.
Web buckled at one end	3/32 in. high	Mar. 10	1.25 p.m.	5.33 p.m.
Web buckled on opposite sides of center splice	1/16 in. high	Mar. 11	1.13 p.m.	4.46 p.m.
Both webs buckled adjacent to center splice	1/16 in. high	Mar. 12	1.09 p.m.	4.49 p.m.

and Nickel Steel Models

Ultimate strength	Percentage under normal weight	Maximum vertical deflection at center		Time of test readings	
		Before tests	After tests	First	Last
70,983	....	....	....	3.20	6.41
73,294	....	....	....	2.13	3.34
79,257	....	....	....	8.10	10.04
79,556	....	....	....	1.45	3.36
79,257	....	....	....	10.00	11.45
77,956	....	....	....	4.05	6.13
33,766	0.96	13/64	4 5/64	1.22	4.25
33,146	0.96	—1/16	+3 7/16	1.23	4.14
32,576	3.83	—13/32	—7	1.13	4.25
32,968	3.83	3/16	6 15/32	1.23	4.59

Lower-Chord Models

Max. Vert. Deflection at Center, Inches		Date of tests, 1913	Time of test readings	
Before test	After tests		First	Last
3/64	7 4/32	February 28	1.55 P.M.	5.26 P.M.
1/32	7 ⅛	March 4	9.27 A.M.	1.14 P.M.
21/64	....	March 3	9.05 A.M.	1.05 P.M.
1/32	7 ⅛	March 5	1.10 P.M.	4.45 P.M.

8.—TESTS ON LIFTING HITCH FOR COMPRESSION MEMBERS

Tests made at McGill University under the supervision of Professor H. M. Mackay.

The tests were made upon a specimen designed to represent the action of a lifting hitch, which it was proposed to use for handling heavy compression members during erection. (See plate LVII—Typical Hitches, Type A.) The specimen consisted essentially of a 40″ x ⅝″ plate, 6′ 2″ long to which were riveted two 6 x 6 x ½″ angles 6′ 2″ long, the combination representing one rib of the member to be lifted. To the flange angles were bolted two 12″ channels 2′ 11″ long, constituting with the link and connecting pin a full sized model of the proposed hitch.

When placing a member in the bridge, two of these hitches would always be required for each main hoist. (See plate LVIII.) Each hitch would, therefore, be required to carry, as a maximum, a load of 60,000 lbs.

Under a load of 100,000 lbs. the maximum deflection observed at the outer margin of the flange angle was 0.047 inches and on removing the load a permanent set of .006 inches. The maximum deflection under a load of 140,000 lbs. was .079 inches and the permanent set .016 inches. At the end of the test it was observed that almost all the ⅞ inch bolts connecting the hitch channels to the flange angles were loose, so that in most cases the nuts could be readily turned with the fingers. It was evident, therefore, that they had been strained beyond the elastic limit and had acquired a slight permanent set.

After these bolts had been tightened, a second test was made. Both deflections and permanent set were somewhat less. At the end of this test most of the bolts were again found to be loose.

A test was then made with the bolts initially loose. The flange angles deflected rapidly at the earlier stages of loading but later the increase was about the same as when the bolts were initially tight. The permanent sets, however, were practically the same as for the case where the bolts were initially tight.

Except as regards the loosening of the bolts, no visible evidences of weakness were observable throughout the tests, and at the conclusion no permanent distortion was noticeable to the eye.

9.—TESTS OF PLATES WITH PIN HOLES WITHOUT REINFORCEMENT

Report by Professors H. M. MacKay and Ernest Brown.

The following Report covers tests made for the St. Lawrence Bridge Co., during 1913-14, in the Testing Laboratory of McGill University upon two series of plates.

FIRST SERIES

The first series consisted of four pairs of plates as follows:—

2	Plates	P1, 12½" x ½" x 6"–4½"	Net	Section	3.625	sq.	ins.
2	"	P2, 12½" x ⅝" x 6'–4½"	"	"	4.53	"	"
2	"	P3, 15" x ⅝" x 6'–4½"	"	"	6.09	"	"
2	"	P4, 15" x ⅝" x 6'–4½"	"	"	6.09	"	"

Plates P_1, P_2 and P_3 each had four pin holes bored to 5¼" diameter, the end holes being bored to two centres ¼" apart so as to make them 5½" long in the direction of the axis of the plate. Plate P_4 had three of these holes replaced by a slot with semi-circular ends. The slots were 5¼" wide and about 2'-11" long. The location of these pin holes and slots are shown in the accompanying drawings. Slight variations were found in the dimensions of the plates and pin holes, but nothing to affect the general results of the tests. It was thought at first that the pin holes and slots might affect the strength of the plates so prejudicially that the maximum load of the Emery Testing Machine in the McGill Testing Laboratory (150,000 lbs.) might produce incipient failure. Such, however, was not the case. At very moderate loads, indeed, the elastic limit of the metal was exceeded locally; and at loads well within the capacity of the machine the yield point was also exceeded locally, as indicated by the scaling of the metal, as well as by extensometer readings. But no damage was done affecting the strength of the plates under a static load. The tests on the first series of plates, therefore, consisted largely in ascertaining the distribution of stress at various sections at loads within the elastic limit of the most highly strained fibres of the metal; the elastic limit and yield point of the most highly strained fibres; and the permanent deformations after subjecting the plates to the working load. The latter was specified as 21,600 lbs. per sq. in. of net section.

The distribution of stress in Plates P_1 to P_4 at loads within the elastic limit is shown by the ordinates of shaded areas in Figures 1 and 2. These areas were plotted from extensometer readings taken at the points indicated. The gauge length was usually 2" and the extensometers near the edges of the plates and pin holes were centred about ⅜" from such edges; so that the short portions of the curves between these points and the edges of the metal were plotted to agree with the general form of the curves. As the pitch of the curves near the pin holes is very steep there is, in such cases, some uncertainty as to the value of the maximum stress. Furthermore, the stress at the edge of a 5¼″ pin hole may vary appreciably in a length of 2″. To overcome this difficulty an extensometer was devised as the tests progressed, which read over a ½″ gauge length, and which could be attached to the actual edge of an unloaded pin hole, thus giving the true maximum stress at such points. The readings of this extensometer indicated that the curves as plotted were nearly correct, although perhaps the maximum stresses shown are a trifle low on the average. A satisfactory check on the accuracy of the work is however provided by the consideration that, for any one plate, the stress areas at sections A, B, C, D, E and F should all be equal. The values of these areas for several plates of the series are shown in the following table. The value of E used in plotting the stresses was 29,800,000 lbs. per square inch.

TEST 9—Table 1

No. of Plate	Actual Load	Stress Area Sq. inches at Sections						Load estimated from stress area
		A	B	C	D	E	F	
P1	40,000	2.02	....	2.03	2.04	....	....	40,600
P2	40,000	1.95	1.97	2.02	2.00	1.97	1.93	39,560
P3	50,000	1.98	2.04	1.98	1.93	1.94	1.90	49,030
P4	50,000	2.06	1.95	1.95	1.98	2.02	2.02	49,925

As these results include all errors arising from observation, variations in dimensions or properties of the plates themselves, plotting and measuring the areas, and all other sources, they constitute a very good check on the accuracy of the work.

It will be noticed that the maximum fibre stress occurs, in every case at the edge of the loaded pin hole. As the diameter of the pin was 5″, ¼″ less than the transverse diameter of the hole, there was ample opportunity for the distortion of the metal. Doubtless the distribution of stress would be modified if the pin filled the hole completely. At the edges of the unloaded pin holes the maximum stress is somewhat less, running from 1.8 to 2.4 times the mean value for the section. In the body of the plates between the pin holes, where the distance between the centres of the holes is three diameters of the hole, the greatest stresses occur at the outer edges of the plate, and the variation of the stress from the edges to the centre is practically linear. On the other hand, when the centres of the holes are 6 diameters apart the stress at the section midway between them is nearly uniform across the width of the plate. In the case of the slotted plate, P_4, the maximum stress again occurs at the edge nearest the pin, and the distribution becomes more uniform at the succeeding sections, until at the end of the slot remote from the pin the stress at the outer edge of the plate is 13% greater than the mean.

These stress distributions are all at loads within the elastic limit of the most highly strained fibre. Whenever that limit is exceeded, the strains of course fail to give a measure of the stress, and the distribution of the latter cannot, therefore, be inferred from extensometer readings.

Elastic Limit and Yield Point.

The elastic limit and yield point were naturally reached first at the edge of the loaded pin holes. The elastic limit occurred in all these plates at mean stresses from 11,000 to 13,000 lbs. per sq. inch of net section; and the yield points at mean stresses from 20,000 to 22,000 lbs. per sq. in. of net section, or in the neighbourhood of the proposed working stresses of 21,600 lbs. per sq. in. These elastic limits and yield points were doubtless very local in character.

The raising of the elastic limit and yield point due to repeated loading is clearly shown in the case of Plate P_2 (Figure 1). Curves showing the relation between total load and elongation are plotted for points 1, 2 and 3, Section A A, and for four consecutive applications of the load. An interval of six days elapsed between the first and second applications of the load; and intervals of 18 hours and 24 hours respectively between the second and third, and third and fourth applications. The curves also show the total load expressed in lbs. per sq. inch of net section of the plate. Using this unit loading it will be seen that at point 3, the elastic limit was reached at a stress of about 11,000 lbs. per sq. inch, and was followed by a somewhat indefinite yield point at about 20,000 lbs. per sq. inch. The actual stress intensity at Point 3 can be inferred for this loading from the elongation which was about 0.0016″ on a gauge length of 2 inches, when elongation ceased to be proportional to total load. This corresponds to a stress intensity of about

$$\frac{0.0016 \times 30 \times 10^6}{2} = 24{,}000 \text{ lbs. per sq. inch,}$$

and as the stress intensity at the edge of the pin hole was probably some 20% to 30% greater than this, the maximum stress was about 30,000 lbs. per sq. inch, a reasonable value for the elastic limit. The maximum elongation at Point 3 for the first application of the load was 0.0048″, three times the above amount, indicating considerable local over-straining.

As already pointed out, the actual stresses in the plate cannot be inferred from the extensometer readings, either under the first or subsequent loadings, once the elastic limit has been passed, even locally. Following the first application of the load, there was a set of 0.0017″ at Point 3 and a set of 0.00015″ at Point 1. The latter does not necessarily mean that the metal at Point 1 had been overstrained, since the final configuration of the plate on removal of the load will depend on the strain of the most highly stressed fibres next the pin hole. An elastic strain at Point 1 will not entirely disappear when the load is removed from the plate, if the metal next the pin hole has been overstrained. Residual stresses will exist in the plate, and none of the observed elongations for the fibres at Point 1 suggest that the metal there was overstrained during any of the repeated loadings.

The raising of the elastic limit by the overstraining of the metal next the pin holes is shown by the continuously lengthening straight line portion of the curves for Point 3 for successive loads. Under the fourth application of the load the elastic limit and yield point became practically coincident at a load of 26,500 lbs. per sq. inch of net sectional area. Assuming the value oi the modulus of elasticity to remain unchanged (as is likely), the increment of stress at Point 3 during the application of the fourth load up to the new and artificial elastic limit, as indicated by the extensometer, would be about 56,000 lbs. per sq. inch, and the maximum stress at the edge of the pin hole, being about 25% greater, would approach 70,000 lbs. per sq. inch. But as the residual stresses due to previous overloading cannot be estimated, it is impossible to infer the actual distribution of stress in this case, even at loads within the new elastic limit.

Similar results are indicated by the load-elongation curves for certain points on Plates P_3 and P_4 (Figure 2), where the effects of two applications of the load are shown. The slow yielding of the most highly strained fibre under a constant load is shown by the curve for Point 3, Section A A, Plate P_4.

Permanent Set.

The largest permanent set observed after applying a load of 21,600 lbs. per sq. inch of net section did not exceed 0.003″ in a 10″ gauge length. It may, therefore, be stated that the permanent sets were of no practical consequence.

SECOND SERIES

As a testing load of 150,000 lbs. was insufficient to cause failure in the plates described above, a second series of tests was made in July, 1914, upon three pairs of plates of smaller section. The dimensions were as follows:—

2	Plates	P_5, 13″ x 3/8″ x 5′-8½″	Net Section,		3.0	sq.	ins.
2	"	P_6, 11″ x 3/8″ x 5′-6½″	"	"	2.25	"	"
2	"	P_7, 9″ x 3/8″ x 5′-6″	"	"	1.5	"	"

These plates were bored to correspond with plates P_1, P_2, P_3 of the first series, except that all pin holes were 5-1/32″ diameter. The corners at one end of each plate were clipped. The loads were applied through 5″ pins.

Specimens cut from the uninjured portions of the metal after testing gave the following results:—(See Table 2).

TEST 9—Table 2

	P5	P6	P7
Yield Point	36875	37950	36950
Ultimate Strength	65500	66600	65600
Elongation in 8″	28.4%	27.3%	28.4%
Reduction of area	56.6%	57.5%	56.7%

Loads were applied ininc rements of 5,000 and 10,000 lbs. until failure took place. In the case of Plates P_5 the load was released after an application of 24,000 lbs. per sq. inch of net section, to observe the permanent deformations. Extensometer readings were made at one loaded and one unloaded pin hole. Extensometer swith 2″ gauge length were placed as near as possible to the pin, and an extensometer with ½″ gauge length was attached to the edge of an unloaded pin hole. The elongations out to out of end pin holes were also observed by a scale reading to $\frac{1}{1000}$ of an inch. All these readings are plotted in Figures 3, 4, and 5.

As indicated by the extensometer, the elastic limit at the loaded pin holes is reached in all the plates of the series at loads of about 13,400 lbs. per sq. inch of net section. As the ratio of the maximum to the mean stress in this case is estimated at 2.8, as explained in discussing the first series, the maximum fibre stress when the elastic limit is reached would be 13,400 x 2.8 = 36,200 lbs. per sq. inch. At the edge of the unloaded pin holes the elastic limit is shown at about 18,000 lbs. per sq. inch of net section, as an average value for all the plates. As the ratio of the maximum stress at the edge of the pin hole to the mean stress across the section is, in this case, from 2.1 to 2.2 we have the value of the maximum fibre stress 18,000 x 2.15 (say) = 38,600 lbs. per sq. inch. These values for the maximum stress are so near the actual yield point of the steel (about 37,000 lbs.)as obtained from specimens cut from the plates, that it seems probable that the elastic limit in such cases means simply the point at which the most heavily stressed fibre reaches the yield point.

The local character of the elastic limit as obtained by sensitive extensometers at the above points is shown by the elongations back to back of pin holes, which show little or no departure from proportionality to load, until loads approximating 26,000 to 27,000 lbs. per sq. inch of net section are reached.

The following table (Table 3) shows the values obtained for the ratio of the maximum to the mean stress for one loaded and one unloaded pin hole, in the case of all the comparable plates of series 1 and 2.

TEST 9—Table 3

Plate	Section	Loaded Pin hole max./mean	Unloaded Pin hole max./mean
P1	12½″ x ½″	2.5	1.9
P2	12½″ x ⅝″	2.3	2.1
P3	15″ x ⅝″	3.1	2.3
P5	13″ x ⅜″	3.0	...
P6	11″ x ⅜″	2.9	2.2
P7	9″ x ⅜″	2.6	2.1

These results show no consistent variation with the widths of the plates.

The highest elongation noted back to back of pin holes at a load of 24,000 lbs. per sq. inch of net section was 0.08″ and the highest permanent set in the same distance after the above load was 0.02″. The first two plates tested, P7-1 and P-61, failed by dishing behind the pin at the end whose corners were clipped. The next three, P5-1, P6-2 and P7-2, had the clipped end blocked behind the pin hole so as to prevent dishing there. Of these P5-1 and P6-2 failed by dishing behind the pin at the square end. P7-2 (section 9″ x ⅜″) failed in tension across an unloaded pin hole. The remaining plate P5-2 had both ends blocked. It failed by splitting longitudinally behind the pin. The accompanying table (Table 4) gives the essential data as to the ultimate loads and modes of failure.

The following points made evident by this table may be noted:—

(1) Clipping the corners behind the pins increases the tendency to dish.

(2) Only two plates were loaded to anything like their capacity to resist tension. Of these, one, P7-1, withstood without failure in tension, a tensile stress practically as great as the ultimate strength of a specimen cut from the same plate. The other, P7-2, failed in tension under a tensile stress per sq. inch of net section 1.04 times as great as the ultimate strength of a speciment cut from the same plate.

(3) When the ends are unrestrained the ratio of total width or net width to thickness does not seem to have much effect upon the capacity to resist dishing behind the pins, that is to say, upon the strength of the plates. The wider plates, in fact, show a little higher strength than the narrower ones of the same thickness. From a comparison of the plates of

TEST 9—TABLE 4

Plate	Dimensions	Net Section Sq. ins.	Ultimate Load Lbs.	Mode of Failure	Pin Bearing Stress at Failure Lbs. per Sq. in.	Tensile Stress at Failure Lbs. per Sq. in.	Ratio Net Width to Thickness	Remarks
P5-1	13″ x 3/8″	3.0	108,800	Dished at square End	57.900	36,000	21.3	Blocked at clipped End
P5-2	do	do	145,000	Split behind Pin.	77,400	48,300	do	Blocked at both Ends
P6-1	11″ x 3/8″	2.25	99,000	Dished at clipped End	52,800	44,000	16	Ends free
P6-2	do	do	104,200	Dished at square End	55,600	46,500	do	Blocked at clipped End
P7-1	9″ x 3/8″	1.5	98,000	Dished at clipped End	52,300	65,300	10.7	Ends free
P7-2	do	do	103,500	Tension	55,000	68,730	do	Blocked at clipped End

this series with the large plates tested at Ambridge in September, 1914, it would seem that the ratio of diameter of pin hole to thickness of the plate is an important factor. Thus two plates 28" x 2" with 12" pin holes ($\frac{d}{t}=6$) stood a bearing pressure of 79,000 lbs. per sq. inch without sign of failure by dishing and may be credited with a bearing capacity of at least 80,000 lbs. per sq. inch. Two plates 26" x 1½" with a 12" pin hole ($\frac{d}{t}=8$) showed signs of dishing (although not enough to cause failure) at an average bearing stress of 71,700 lbs. per sq. inch. The average bearing stress at the point of failure by dishing for the three widths of plates included in series 2 is 56,000 lbs. per sq. inch and $\frac{d}{t}=13.3$. Plotting these results it is observed that a straight line is obtained whose equation is nearly represented by $p=100{,}000-3{,}300\frac{d}{t}$, where "p" is the bearing pressure to produce dishing "d" the diameter of the pin hole and "t" the thickness of the plate. This suggestion is made with all reserve, on account of the inadequacy of the data on which it is based. But on the other hand the relation seems too striking to be altogether accidental.

Table 5 gives the diameters of all pin holes before and after testing to failure.

The broad conclusion to be derived from the tests of series 2, as well as from the full sized plates tested at Ambridge is, that when the ratio of thickness to the diameter of the pin holes is sufficient to prevent dishing, the plates will develop a strength in tension per unit of net sectional area very nearly or quite equal to the ultimate strength of the metal of which the plates are composed; and that this is true notwithstanding the very uneven distribution of stress at loads within the elastic limit.

TEST 9—Table 5

Plate	Dimensions of Pin holes before and after failure							
	Loaded hole		Unloaded hole		Unloaded hole		Loaded hole	
	Hor.	Vert.	Hor.	Vert.	Hor.	Vert.	Hor.	Vert.
Original A 11 Plates	5.03	5.03	5.03	5.03	5.03	5.03	5.03	5.03
P_5-1	5.02	5.12	5.02	5.04	5.03	5.04	5.00	5.17
P_5-2	5.00	5.90	4.96	5.20	4.96	5.20	5.00	5.36
P_6-1	5.00	5.26	5.00	5.12	5.00	5.10	5.01	5.18
P_6-2	5.00	5.24	4.98	5.15	4.98	5.15	5.04	5.32
P_7-1	5.00	5.60	4.85	5.48	4.83	5.48	5.00	5.48
P_7-2	5.00	5.74	4.94	6.14 *	4.82	5.76	5.00	5.68

*Tension failure at section through this pin hole.

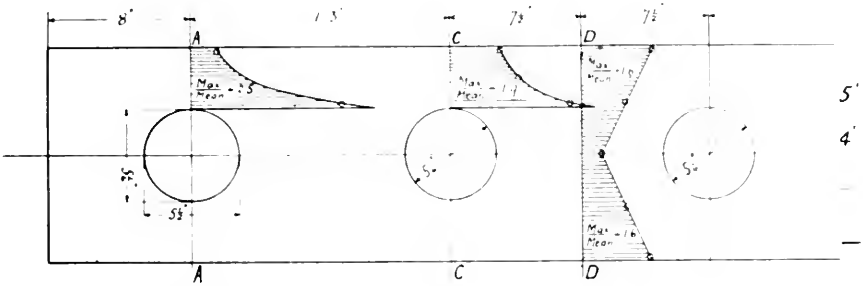
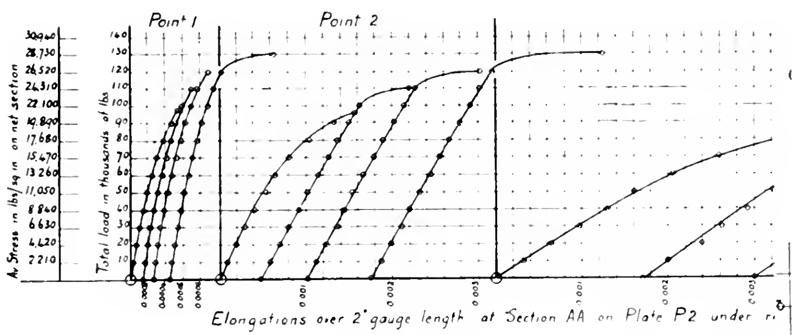


PLATE P1

12½" x ½" x 6'4" long   Total load 40 000 lbs -
Stress diagrams   1" = 10 000 lbs sq in
Gauge length 2"



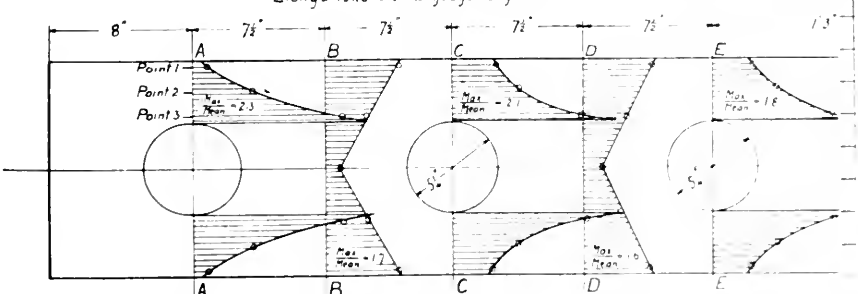
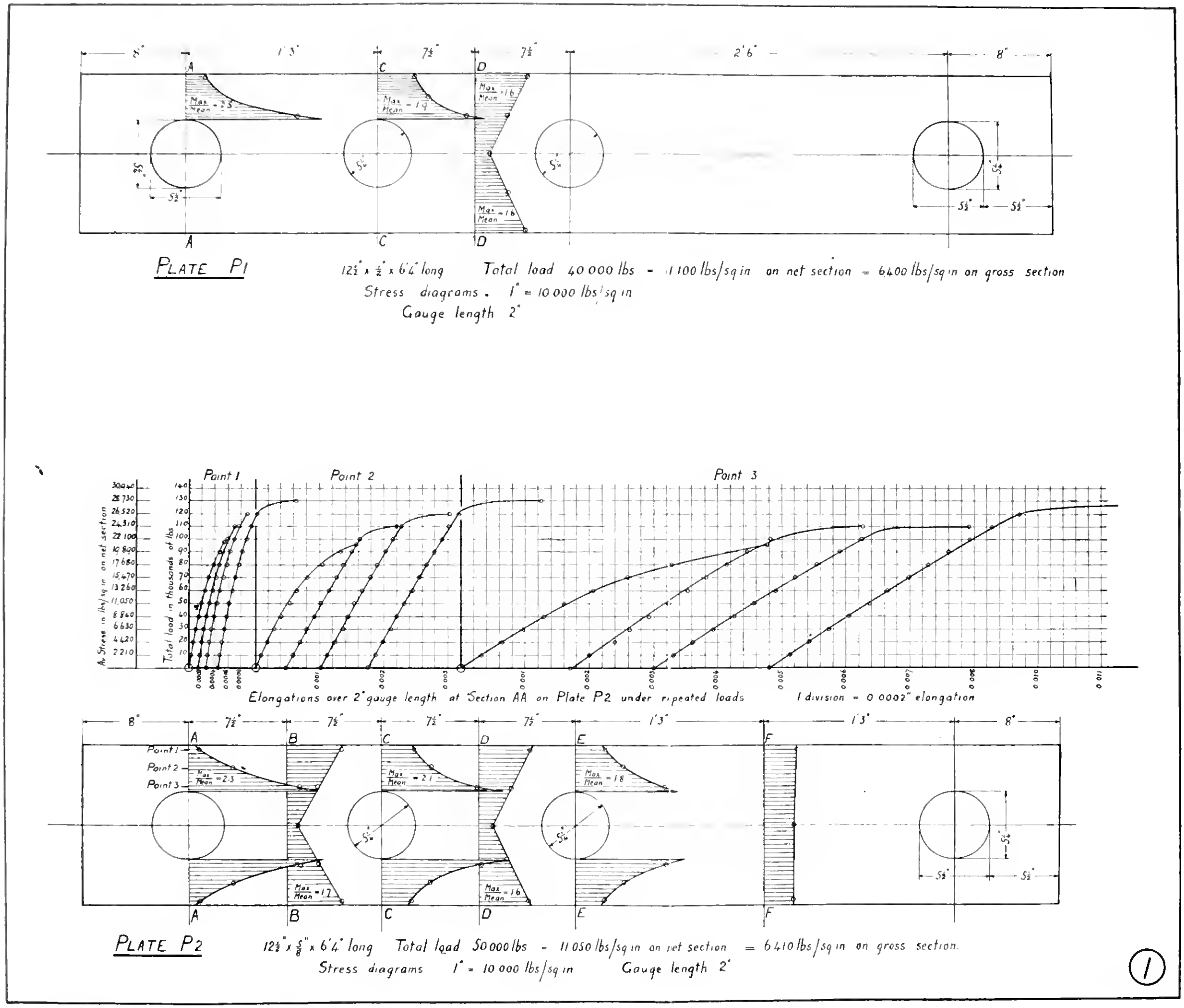


PLATE P2

12½" x ⅝" x 6'4" long   Total load 50 000 lbs - 11 050 lbs/sq in on
Stress diagrams   1" = 10 000 lbs/sq in   Gauge



TEST 9 — Figure 1 242

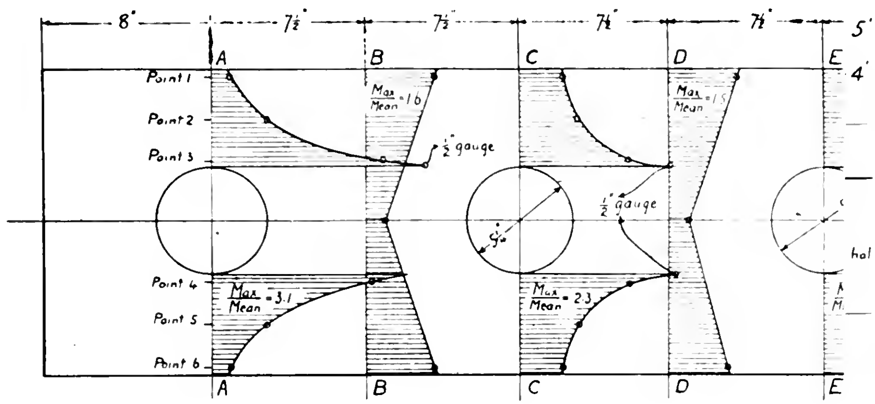


PLATE P3

15" x ⅝" x 6'4" long. Total load 50.000 lb:
Stress diagrams
Gauge length 2" e bo

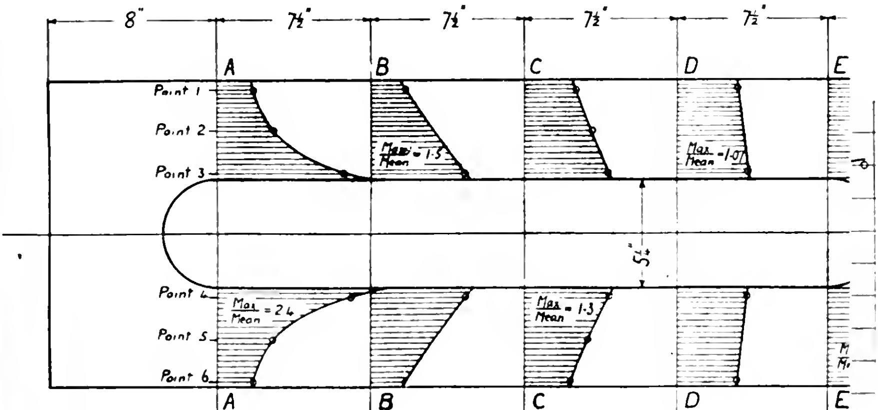


PLATE P4

15" x ⅝" x 6'4" long Total load 50.000 lbs
Stress diagrams
Gauge length 2"

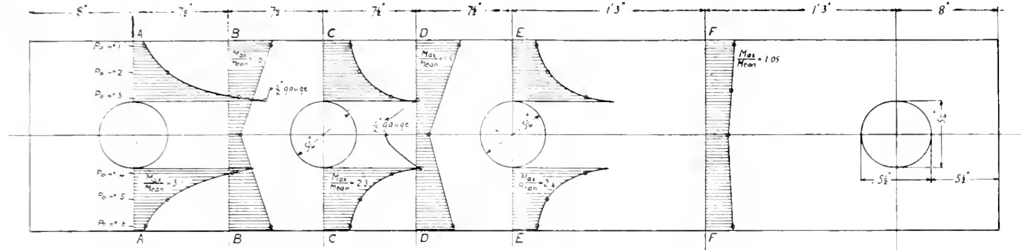
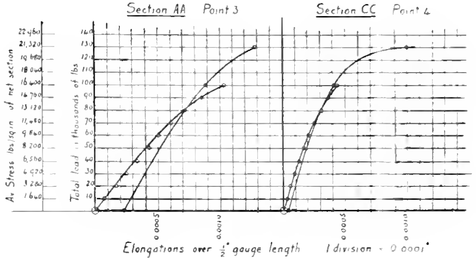


PLATE P3 15" x ⅝" x 6'4" long Total load 50 000 lbs = 8200 lbs/sq in on net section = 5330 lbs/sq in on gross section
Stress diagrams 1" = 10 000 lbs/sq in
Gauge length 2" except as noted

PLATE P3 Effect of repeated loading



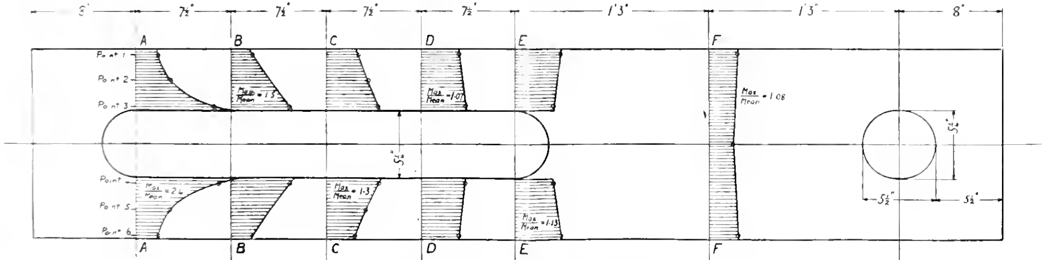
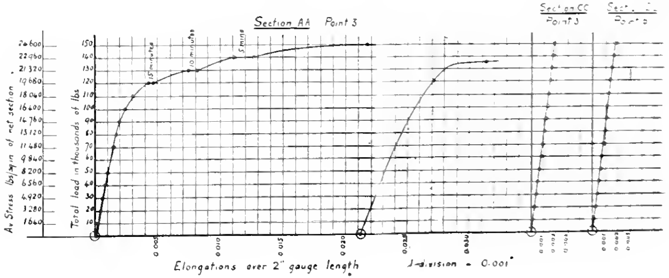
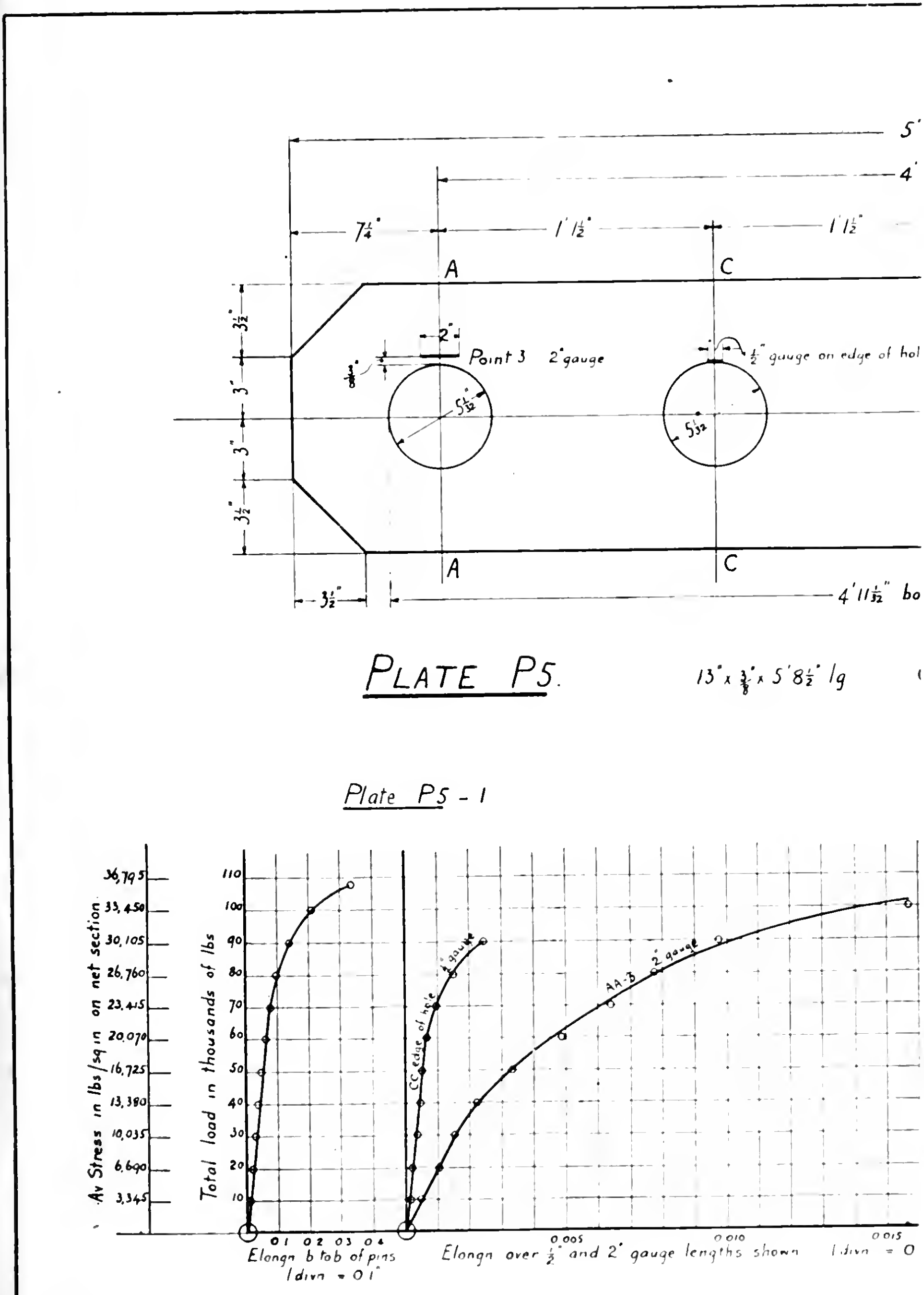


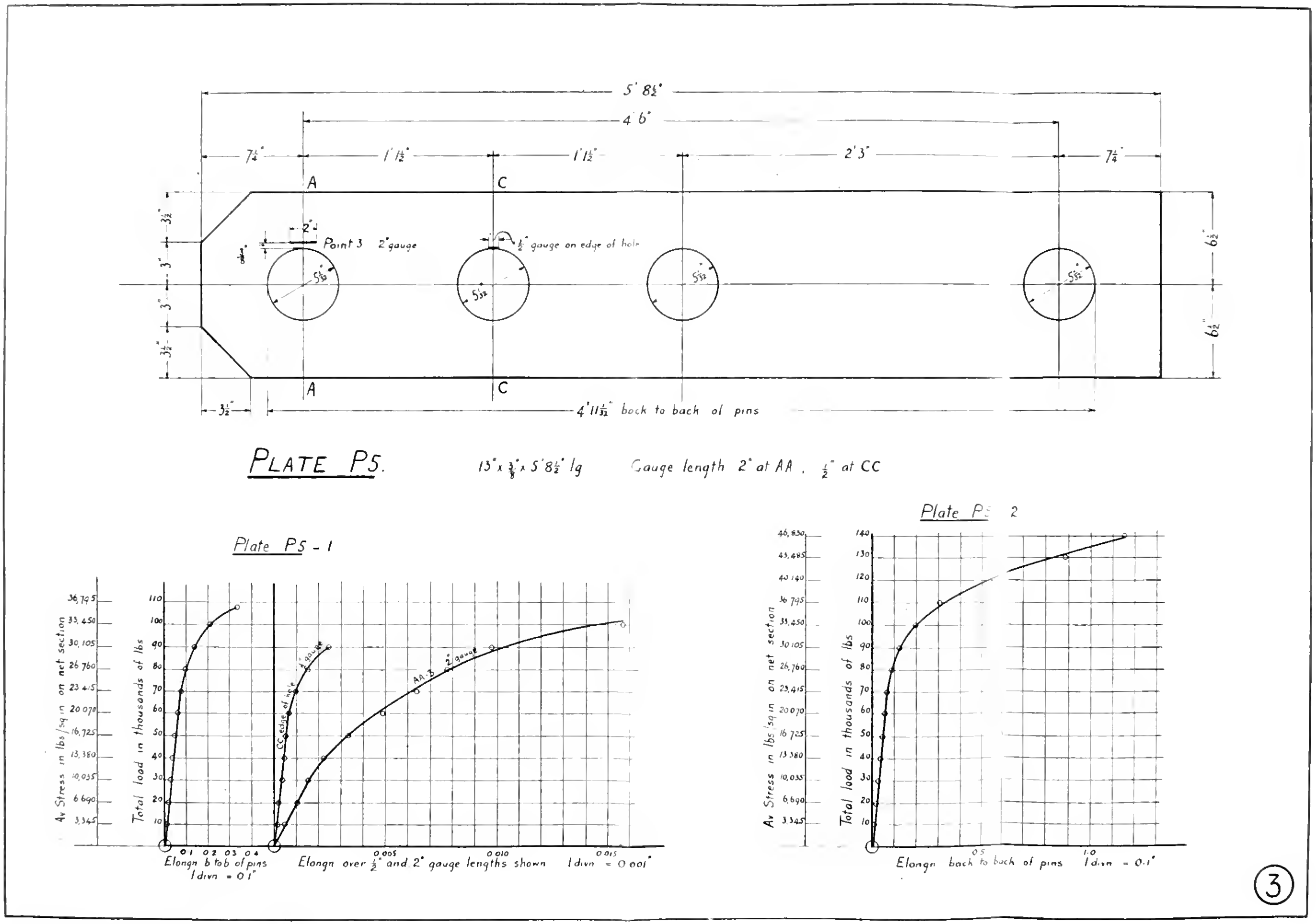
PLATE P4 15" x ⅝" x 6'4" long Total load 50,000 lbs = 8200 lbs/sq in on net section = 5330 lbs/sq in on gross section.
Stress diagrams 1" = 10,000 lbs/sq in
Gauge length 2" except as noted.

PLATE P4 Effect of repeated loading, and of time interval during loading First loading [illegible] lbs
Second loading 150 000 lbs Yielding at Section AA but not at Sections CC and EE



2


5'
4'
7¼"
1' 1½"
1' 1½"
A
C
2"
Point 3 2" gauge
½" gauge on edge of hol
3½"
3"
3"
3½"
⅜"
5 1/32"
5 1/32
A
C
3½"
4' 11 1/32" bo
PLATE P5.
13" x ⅜" x 5' 8½" lg
Plate P5 - 1
Av Stress in lbs/sq in on net section
36,795
33,450
30,105
26,760
23,415
20,070
16,725
13,380
10,035
6,690
3,345
Total load in thousands of lbs
110
100
90
80
70
60
50
40
30
20
10
CC edge of hole ½" gauge
AA-3 2" gauge
0.1 0.2 0.3 0.4
Elongn b tob of pins
1 divn = 0.1"
0.005
0.010
0.015
Elongn over ½" and 2" gauge lengths shown 1 divn = 0



TEST 9 — Figure 3

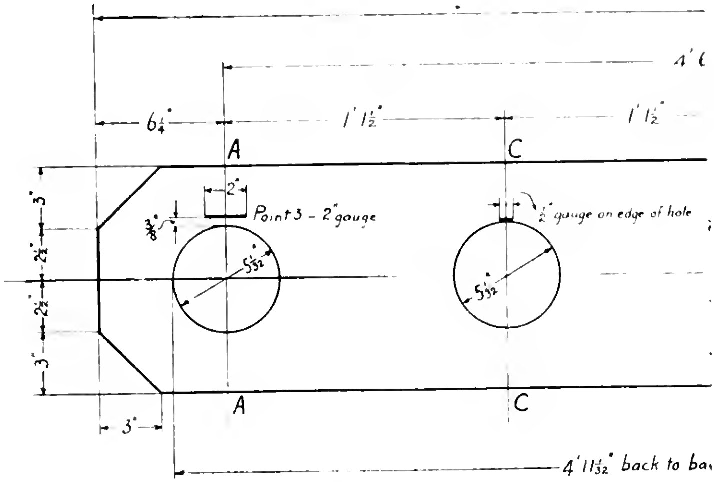
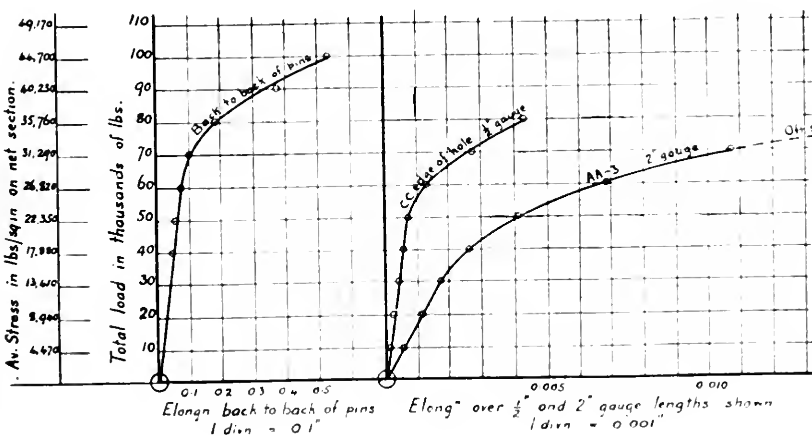
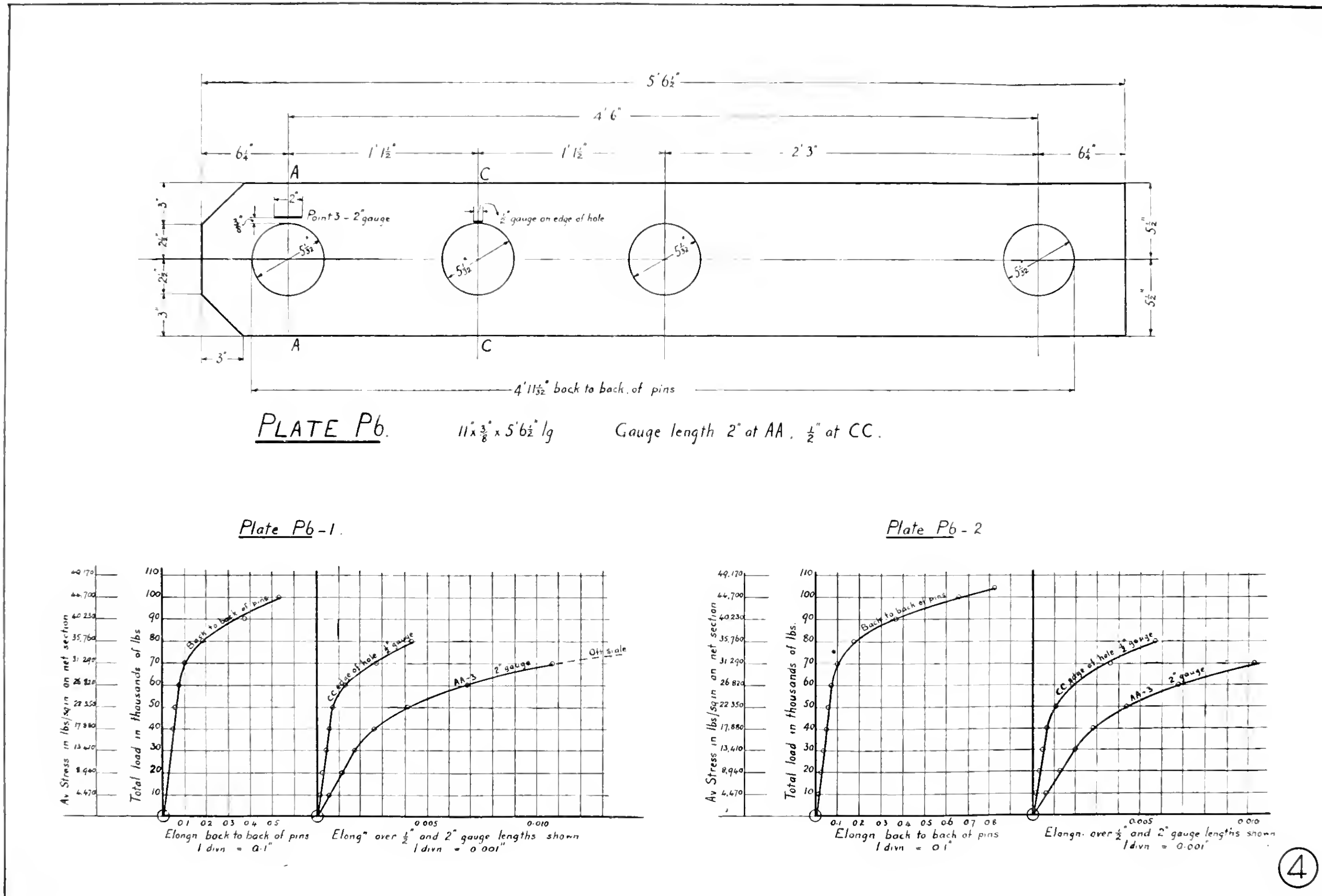


PLATE P6. $11'' \times \frac{3}{8}'' \times 5' 6\frac{1}{2}''$ lg

Plate P6-1.





5' 6½"
4' 6"
6¼"
1' 1½"
1' 1½"
2' 3"
6¼"
A
C
Point 3 - 2" gauge
½" gauge on edge of hole
A
C
4' 11$\frac{1}{32}$" back to back of pins
PLATE P6.
11 x ⅜" x 5' 6½" lg
Gauge length 2" at AA, ½" at CC.
Plate P6-1.
Back to back of pins
CC edge of hole ½" gauge
AA-3 2" gauge
Off scale
Av Stress in lbs/sq in on net section
Total load in thousands of lbs
Elongn back to back of pins
1 divn = 0.1"
Elong" over ½" and 2" gauge lengths shown
1 divn = 0.001"
Plate P6-2
Back to back of pins
CC edge of hole ½" gauge
AA-3 2" gauge
Av Stress in lbs/sq in on net section
Total load in thousands of lbs.
Elongn back to back of pins
1 divn = 0.1"
Elongn. over ½" and 2" gauge lengths shown
1 divn = 0.001"
4

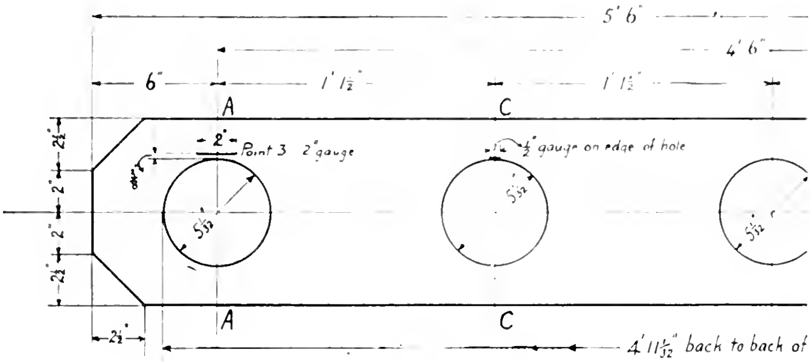
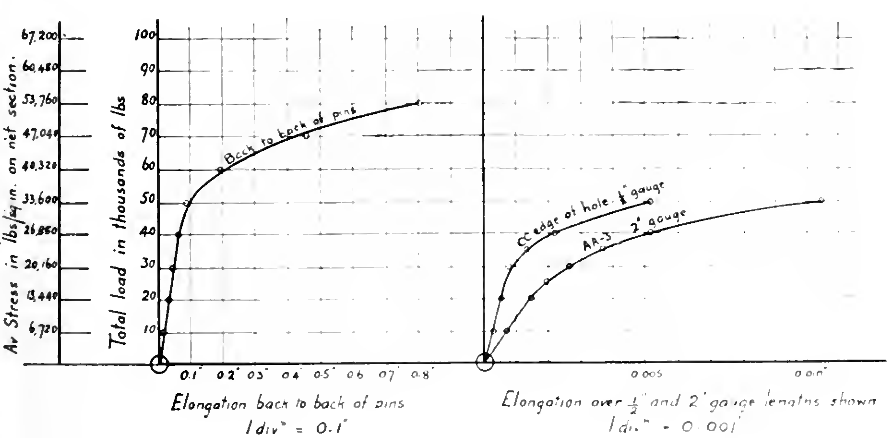


PLATE P7 $9" \times \frac{3}{8}" \times 5' 6"$ long Gauge len

Plate P7 - 1



TEST 9 — Figure 5

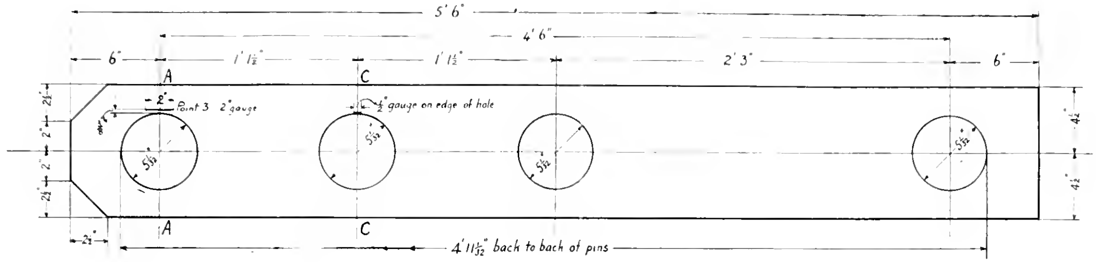


PLATE P7 — 9" x 3/8" x 5' 6" long — Gauge length 2" at AA ; ½" at CC.

Plate P7 - 1

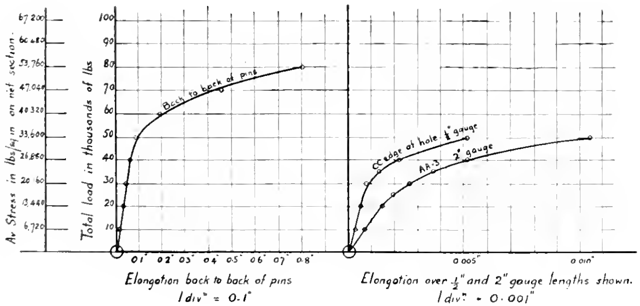
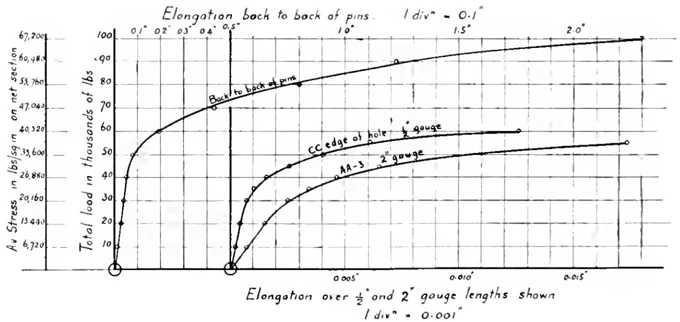


Plate P7 - 2



5

10.—TESTS ON 2—28 x 2″ AND 2—26 x 1½″ PLATES TO ASCERTAIN THEIR ULTIMATE STRENGTH

The plates were tested on the large testing machine of the American Bridge Co., at Ambridge, Pa., in September, 1914, under the supervision of Professor H. M. Mackay, who reported on the results of the tests in part, as follows:—

The detail dimensions of the plates are shown in Fig. No. 1.

The object was to ascertain the ultimate strength in tension and also to observe the behaviour of the plates under load, and more especially at loads of 18,000 lbs. and 25,000 lbs. per square inch of net section.

The Ambridge testing machine is so arranged that, in the case of large test pieces, the load comes on very rapidly. It is, therefore, difficult to stop at any predetermined load below the yield point. It is impossible to hold the load stationary. The above mentioned loads were, on this account, considerably exceeded in some cases.

A brief account of the individual tests follows:—

Plate 1. Nominal cross section 28″ x 2″.

Actual thickness 2.02″.

Actual net section (least) 32.56 sq. ins.

At 19,500 lbs. per sq. in. of net section scaling was distinctly noticeable back of and at the sides of the loaded pin holes.

At 25,000 lbs. per sq. in. the scaling was more pronounced about the loaded pin holes. No scaling was noticeable about the unloaded holes.

At 55,000 lbs. per sq. in. of net section (about 32,000 lbs. per sq. inch of gross section) scaling became general over the body of the plate.

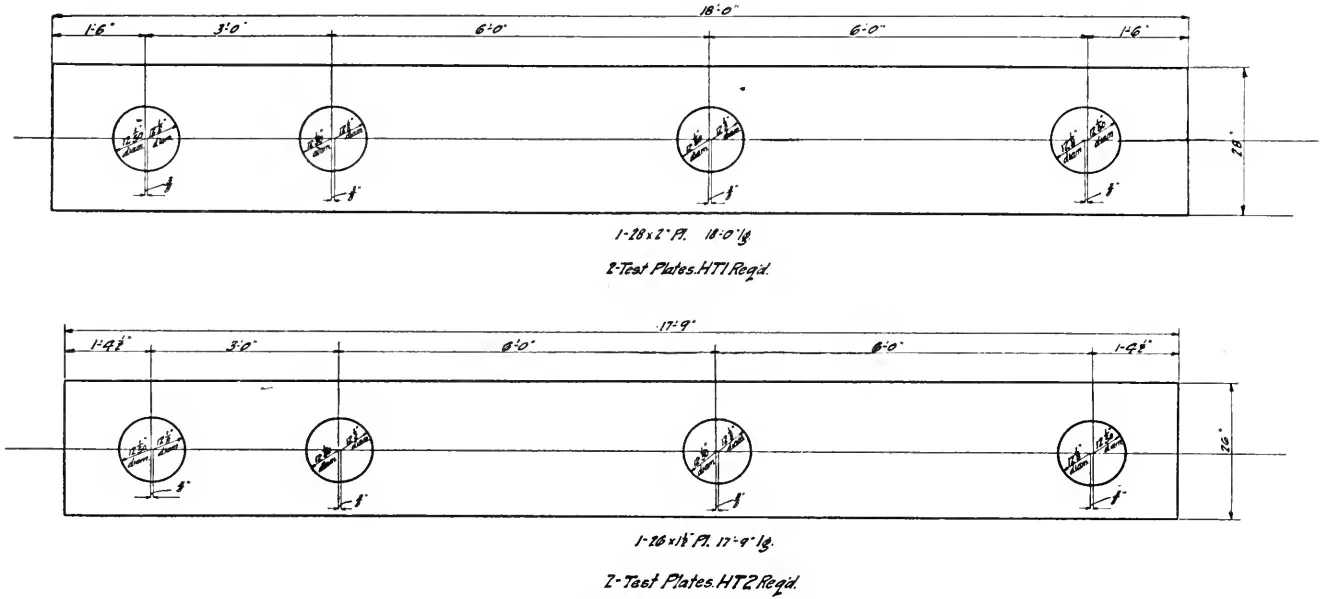
At 57,800 lbs. per sq. inch the plate fractured transversely across the loaded pin hole at the "dead" end. The fracture was coarsely crystalline with some traces of included slag.

Plate 2. Nominal dimensions 28″ x 2″.

Actual thickness 2.02″.

Actual net section (least) 32.60 sq. ins.

At 19,900 lbs. per sq. in. moderate scaling had occurred back of the loaded pin holes. There was no noticeable scaling around the unloaded pin holes, and on release of the load no permanent deformation could be detected in the unloaded holes with a micrometer gauge reading to one thousandth of an inch.



TEST 10—Figure 1

At 29,400 lbs. per sq. inch scaling was heavy about the loaded pin holes, and had extended to unloaded holes. Upon release of the load, the unloaded pin holes showed permanent deformations of 0.016″ transversely and 0.048″ longitudinally.

At 46,900 lbs. per sq. inch, scaling was general over the body of the plate.

At 58,600 lbs. per sq. inch the plate failed by splitting axially behind the loaded pin hole at the "dead" end. Fracture coarse crystalline.

Plate 3. Nominal dimensions 26″ x 1½″.

Actual net section (least) 21.15 sq. inches.

At 23,800 lbs. per sq. inch moderate scaling occurred back of loaded pin holes.

At 29,800 lbs. per sq. inch heavy scaling occurred at loaded pin holes, moderate scaling at other pin holes.

At 60,300 lbs. per sq. inch the plate fractured transversely across the unloaded pin hole nearest the "dead" end. Fracture coarse crystalline. A slight tendency to "dish" was noticed at the "live" end.

Plate 4. Nominal dimensions 26″ x 1½″.

Actual net section (least) 21.09 sq. inches.

At 18,000 lbs. per sq. inch slight scaling occurred back of loaded pin holes. Upon releasing the load no permanent set was perceptible in the unloaded pin holes.

At 26,100 lbs. per sq. inch scaling was noticed at all pin holes. The permanent deformation of the unloaded pin holes was 0.006″ transversely and 0.004″ longitudinally.

At 61,900 lbs. per sq. inch the plate fractured transversely through the loaded pin hole at the "dead" end. A tendency to dish was also evident, amounting to a deflection of about ½ inch from a straight line drawn transversely across the plate behind the loaded pin hole. Fracture was coarsely crystalline.

The ends of the plates behind the pins were not restrained by blocking or in any other way. While both 1-½″ plates showed some tendency to "dish," no such tendency appeared in the 2″ plates. The ratio of net width through the pin hole to thickness, in the former plates was 9.35, and in the latter 8.0. It may be noted that, in the case of the small plates tested at McGill, the only ones with unblocked ends which did not fail by dishing had the above ratio about 10.6

The following Table 1. gives the reduction of area at a right section across all pin holes.

TEST 10—Table 1

Plate No.	Dimensions Nominal Inches	Ultimate strength lbs./sq. in. Net section	Reduction of area %			
			P.H. 1	P.H. 2	P.H. 3	P.H. 4
1	28 x 2	57,800	11.2	13.1	14.0	19.2x
2	28 x 2	58,600	10.6	10.25	11.6	14.1x
2	26 x 1½	60,300	14.6	14.0	17.6x	17.0
4	26 x 1½	61,900	13.4	13.1	14.4	17.7x

x—Point of Failure.

11.—TESTS ON LINKS REMOVED FROM THE HANGERS USED IN RAISING THE SUSPENDED SPAN IN 1916

After the loss of the first span on September 11th, 1916, careful measurements were made on all the links to ascertain if any had been seriously over-stressed. The measurements showed that all had received a permanent set, particularly the ones on the South-East and North-West corners, but none of an amount that would in itself condemn them for use when raising the second span. As, however, there existed a prejudice against using these links again, it was decided to have several of them tested to destruction at the Ambridge Plant of the American Bridge Co.

Each link was built up of 2—30 x 1⅛ plates stitch-riveted together, as shown in Figure No. 1.

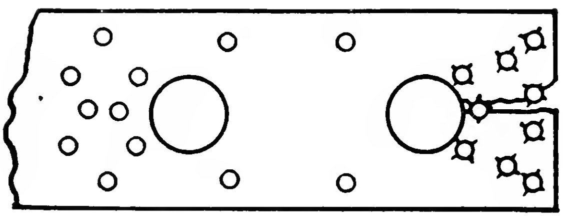
The ultimate strength developed was surprisingly low and as there seemed to be little doubt that this was largely influenced by the stitch-rivets, rather than by any injury which the links may have received in the accident, it was decided to order an entirely new set, and to avoid the use of stitch-rivets. The new links were single plates 28″ x 1½″ as shown on plate C1 and were assembled as shown on plate XCIII.

Professor H. M. Mackay was present when the tests were made on the links at Ambridge and reported as follows:

The following Table 1. summarizes the results as regards the ultimate strength figured on the basis of a net sectional area of 39.375 square inches.

TEST 11.—Table 1

Member	Maximum load	Ultimate Strength lbs. per sq. inch	Remarks
ELH_2E	1,475,000	37500	Split behind slotted hole.
ELH_2F	1,500,000	38100	" " round "
ELH_3C	1,500,000	38100	One plate cracked.
	1,620,000	41100	Both plates split behind pin hole.
ELH_3D	1,550,000	39400	Split behind pin hole.
ELH_3E	1,575,000	40000	" " "
ELH_3F	2,090,000	53100	One plate split behind pin hole.
			" " dished about ⅝″.
ELH_4C	1,635,000	41500	Split behind pin hole.
ELH_4D	1,570,000	40000	" " " "



Sketch showing arrangement of Rivets
and Typical Method of Failure.

TEST II Fig. 1.

The mode of failure was very similar in all plates, except as noted above. Usually a crack first developed suddenly and with a sharp report from the end of the plate either to the rivet on the axis behind the pin hole, or to one of the rivets nearest the end and adjacent to the axis of the plate. Upon continued application of the load the crack would then extend from the rivet in question to the pin hole.

In the case of ELH2E, the first member tested, the loading pins were not parallel owing to the condition of the bearings. The divergence was about 5/16 inch in possibly about 6" of length, with an indeterminate divergence due to the shape of the pins. This member gave the lowest result. For subsequent tests the machine was corrected. In other tests some of the loaded holes were as nearly as possible at right angles to the axis, while in others variations of the bearing surface of the pin hole from 1/64 to 3/64 from the perpendicular were noted. It is not possible, however, to trace any relation between this inclination and the ultimate strength or mode of failure.

In some cases, too, one of the two plates riveted together projected at the end beyond the other, the maximum amount of such projection being about two inches. So far as could be observed, or inferred from the fracture, the longer of such plates cracked first.

Tests were made later at McGill University on fourteen model plates. These were, except as regards length, one-fourth the linear dimensions of the actual hangers and were designed to show the effect of the following variables:—

(1) Sheared vs. Finished Edges.

(2) Round vs. Oblong Pin holes.

(3) Riveted vs. Plain Plates.

The results showed no appreciable difference as between sheared and finished edges or as between round and oblong holes. The plain plates developed on the average an ultimate strength equal to 99% of the test specimens, while the riveted plates developed only 79.5% of the specimens. Excepting one case of dishing all the riveted plates failed by splitting behind the pin holes in the characteristic way, while none of the plain plates failed that way all breaking transversely.

12.—TESTS ON PLATES IN TENSION TO INVESTIGATE THE EFFECT OF INCLINING THE PLATE TO THE AXIS OF LOAD

Four Plates 28″ x 1½″ were used for each hanger when lifting the suspended span In 1917. The stresses in these plates are shown on plate CI and the manner in which they were arranged on plate XCIII.

The design was criticized by Mr. Joseph Mayer of the staff of the Board of Engineers, who claimed, that under certain unfavorable conditions of loading, very heavy bending stresses would result at the ends of the plates, which combined with the direct stress would give a stress in excess of the elastic limit of the material used; that this stress might alternate from one side of the plate to the other during the lifting operation, and finally that the knowledge of the behaviour of plates under such condition of loading was not sufficient to warrant their use in the manner proposed. His argument was based on the assumption that the bending stress which might be expected was expressed by the formula $S=\frac{1}{R}\,\frac{dT}{aL-2}$ where S = the bending stress in lbs. per square inch due to the end moment, R = the net section modulus (across the loaded pin hole), d = the relative motion of the center of bearing of the two ends of the plate, T = the total load, $a=\sqrt{\frac{T}{EI}}$, I = the gross moment of inertia, E = Young's modulus = 29,000,000 L = the distance center to center of end pins.

There was no disputing the accuracy of the calculations required to develop the formula for the bending stress but there did seem to exist reasonable grounds to question the accuracy of the assumptions on which they were made. A test was, therefore made at McGill University on four plates, one-fourth the linear dimensions, except as to length, of the 28 x 1½″ plates, with a view to discovering if these bending stresses did exist and if so what effect they had on the ultimate strength of the plate.

A copy of Professor H. M. Mackay's report, under whose supervision this test was made, follows:—

This Report describes a series of tests on four plates 7″ x ⅜″ x 5′ 3¼″ over all, and bored with holes for 3 inch pins. The end pin holes were 4′ 6″ c. to c. of pins, and each hole was bored to two radii 1½″ and 1-9/16″ respectively, from centre 5/32″ apart. The three intermediate holes were circular, and 3⅛″ in diameter. The object of the tests was to find how the strength of the plates was affected, when loaded through the end pins, by inclining the plates to the axis of the load, and also to find how the stress distribution on a section through the loaded pin hole was affected by such inclination.

Two specimens cut from the same stock as the plates were tested with the following results:—

A Yield Point 30,600 lbs./sq. in.
Ultimate Strength, 63,200 lbs./sq. in.

B Yield Point 30,200 lbs./sq. in.
Ultimate Strength 63,400 lbs./sq. in.

The first two tests (Plates T15-1 and 2) were made as follows:—

The plate was assembled in the testing machine with the upper end placed centrally on the pin (in the axis of the machine). The lower end was deflected ½ inch, say northerly, and a load of 36,000 lbs. was applied, or about 25,800 lbs. per sq. inch of net section. This load was held for a short time, released and then the lower end of the plate was deflected ½ inch off centre in the opposite direction, and the same load applied. The elongation back to back of pin holes was noted in each case. After repeating the above loading thirty or more times, the plate was tested to destruction.

Plate 1 failed by fracture across a loaded pin hole at 92,500 lbs. total load, or 63,800 lbs. per sq. inch of net section, a little above the ultimate strength of the test specimen. *Plate 2* failed by dishing behind the pin hole at 90,000 lbs. total load or 62,070 lbs. per sq. inch of net section, about 0.4% below the average ultimate strength of the test specimens. The ultimate bearing stress was, however, 80,000 lbs. per sq. inch. From previous tests on plates an ultimate bearing strength for structural steel was deduced $=100{,}000-3300\dfrac{d}{t}$, when d = diameter of pin and t = thickness of plate. This would give an ultimate bearing strength in the present case of 73,600 lbs. per sq. inch, which was exceeded. It is apparent, therefore, that the ultimate strength of these plates whether as regards tension or bearing capacity was not lowered by the repeated applications of the inclined load.

The third test was designed to measure the actual stresses on a section through a loaded pin hole with the plate inclined to the axis of the load. For this purpose mirror extensometers with ½ inch gauge lengths were used, one on each face of the plate. As the plate bends on application of load, all mirrors attached to it rotate. A third mirror, was, therefore, clamped to the plate as near to the extensometers as possible to measure the amount of such rotation and furnish a correction to the readings of the extensometers. Two complications were encountered in this measurement. First, in nearly all plates, owing to kinks, bends, accidental eccentricities, etc., the observed stresses on the two faces of a plate are seldom the same no matter now carefully the load is centered. Second, the stress distribution across a section through a pin hole is far from uniform, the stress at the edge of the hole often being as much as three times the mean stress. The first of these difficulties was overcome by loading the plate centrally to begin with, and measuring the deformations in that case, and subsequently comparing these with the deformations when one end was offset. To obviate the second, the extensometers were set as nearly as possible at the point of mean stress on the transverse section through the pin hole. Care had to be taken, however, to keep the loads low so that the elastic limit at no point should be exceeded. For if the elastic limit were once exceeded at any point, subsequent extensometer readings might be incapable of interpretation.

Table 1 gives the measured stresses under a central load on both faces of the plate on a section through the loaded pin hole. The extensometers were placed four-tenths of the distance from the edge of the hole to the edge of the plate, at the point where former experience showed the mean stress in the plate might be expected. As the extensometers bear, not on a point, but over a line of finite length, they can only be placed approximately. However, the agreement of the mean stress and the measured stress computed for E=29,000,000 lbs. per sq. inch agrees very closely and constitutes a satisfactory check on the working of the instruments. It will be seen that although the plate was unusually straight there is a considerable difference between the stresses on the two faces of the plates under a central loading.

Table 2* gives the corresponding results when the top of the plate is offset 5/16″ to the south. The agreement of the mean measured stress with the actual mean is not quite so close but is fairly satisfactory, when it is considered that an error of one one-hundred-thousandth inch in measurement means about 600 lbs. per square inch in stress.

The excess of stress on the north side gives the total fibre stress due to bending plus that indicated in table I as due to initial causes and is, except in the case of the 8,000 lbs. load, greater than that due to bending alone. The greatest value observed is 2,900 lbs. per sq. inch or 29 per cent of that theoretically due to bending.

Table 3 corrects these stresses for the inequality due to initial causes as observed under a central load, as far as the data are available. In the case of the 14,000 and 16,000 lb. load the inequality for a central load is assumed the same as for 10,000 and 12,000 lb. load and the measured bending stresses may vary considerably from those indicated, but should undoubtedly lie between the values of Table II and Table III.

TEST 12—Table I—Plate T_{15}-3, Centrally Loaded

Total Load	Mean Stress lbs. per sq. in.	Measured Stress North Side	Measured Stress South Side	Mean Measured Stress	Excess North Side
8000	5520	5220	5800	5510	-290
10000	6900	7540	6380	6960	+580
12000	8270	8700	7540	8120	+580

TEST 12—Table 2—Plate T15-3—Top Offset 5/16″ to South

Total Load	Mean Stress per sq. in.	Measured Stress North Side	Measured Stress South Side	Mean Measured Stress	Excess North Side	Theoretical Bending Stress	Measured % Theoretical
8000	5520	6960	4060	5510	1450	8780	16.5
10000	6900	8700	4060	6380	2320	9260	25
12000	8270	11020	5800	8410	2610	9670	27
14000	9650	12180	6380	9280	2900	10100	29
16000	11020	13340	9280	11310	2030	10500	20

TEST 12—Table 3—Mean Values

Load	Total Excess North Side	Excess due Central Load	Excess due to Bending	Theoretical Bending	Observed % Theoretical
8000	1450	—290	1740	8780	20
10000	2320	580	1740	9260	19
12000	2610	580	2030	9670	21
14000	2900	(580)	2320	10100	23
16000	2030	(580)	1450	10510	14

It seems safe therefore, to conclude that up to the loads taken, the observed bending stresses do not amount to more than from 20 to 30 per cent of those calculated from the formula $S = \frac{I}{R}\,\frac{d\,T}{aL\text{-}2}$. This formula depends for its validity on the assumption that the axis of the plate at the point of bearing will remain perpendicular to the axis of the pin. The writer believes that this will not usually be the case. The bending moments considered can be developed only by the eccentricity of the resultant of the pressure on the pin. In the case of the plates in question and for a 5/16″ offset up to total loads (T) about 16,000 lbs., the resultant of the pin pressure, in order to develop the bending stresses indicated would fall outside the middle third of the thickness of the plate, and the plate would tend to "tip" on the pin. At a load of 16,000 lbs., the above resultant would fall about the middle third. Very careful measurements show that at a load of 16,000 lbs., the thickness of the metal back of the pin hole increased 0.001″, indicating some flow. Moreover, while the inclination of the plate to the axis of the pin at the initial load was 0°-20′, the inclination as shown by a mirror clamped on the centre line of the pin, was 0°-8′ at a load of 16,000 lbs. Plates 1 and 2, as observed before, clearly indicate flow of metal under the conditions pertaining to them. The writer believes the tests indicate this as the most probable reason why the theoretical bending stresses are not realized, and that they are therefore no more likely to prove harmful to the ultimate strength, than the very high stresses obtaining within the elastic limit at the edge of the pin holes.

The fourth Plate T15-4 was given a larger offset, viz.,—½ inch, and the total load was carried to 29,000 lbs. This plate was not straight, and under an axial load of 12,000 lbs., the observed stress on the north side at the point of measurement (transverse axis of lower pin) was 3,860 lbs. per sq. inch greater than the mean observed stress.

The upper end was then offset ½ inch to the South, so that the fibre stress from bending on the north side at the point of measurement, would be added to bending stress due to initial curvature. The accompanying table (Table 4) summarizes the results. Comparing columns 2 and 3 it will be seen that the observed mean stress is anywhere up to 10% less than the calculated mean stress. This is no doubt due, as previously mentioned, to the difficulty of predicting the point of mean stress on a transverse section through the pin hole, and making the extensometer bear at that point, if correctly predicted. Column 4 gives the maximum observed stress and column 5 the excess of the stress on the north side above the mean. This excess stress is certainly due in part to initial curvature, but it is impossible to say just how much is so caused. If however it be all attributed to bending due to the ½ inch offset, it does not in any case exceed about 30% of the theoretical stress caused in that way. The true percentage is certainly less, and probably much less. These results therefore seem to confirm strongly those obtained in the previous tests.

TEST 12—Table 4—Plate T15-4

Load	Mean Stress Calculated Per sq. in.	Mean Stress Observed	Max. Observed Stress North Side	Excess Max. Over Mean	Theoretical Bending Stress	Total Excess % Theoretical
16000	11030	10150	14500	4350	16840	25.8
18000	12400	11600	15660	4060	17490	23.2
20000	13800	13050	17400	4350	18090	24.0
22000	15150	14210	19900	5690	18700	30.4
24000	16500	15950	21460	5510	19230	28.8
26000	17900	16820	22600	5780	19840	29.1
28000	19300	18850	24400	5550	20370	27.2
29000	20000	20300	26100	5800	20700	28.0

13.—TESTS ON LEAD IN BEARING

Before deciding upon the final dimensions of the lead bearing, used to carry the weight of the suspended span at Sillery (See page 138 and Plate LXXXVIII), a test was made at McGill University to observe the behaviour of lead under heavy loads. For the purpose of the test a small model, to represent the conditions of loading at Quebec, was made. It consisted of a $9\frac{1}{2}$ x $\frac{3}{4}''$ plate, $1'\ 4\frac{1}{2}''$ long with four plates $3\frac{1}{4}''$ x $\frac{3}{4}''$ riveted to one face and so placed as to leave an opening $3''$ x $10''$ x $\frac{3}{4}''$ deep. Four pieces of $\frac{1}{8}''$ sheet lead 2—15/16″ wide of lengths varying from 5″ to 10″ with the shorter piece on top were placed in this opening and covered with a plate 2—15/16″ x $\frac{3}{4}''$ x 9—15/16″ long. The load was then applied in increments, the top or loose plate being removed after each increment to observe the behaviour of the lead.

There was very little flow up to a load of 75,000 lbs., or 5,000 lbs. per sq. inch on the 3″ x 5″ piece of lead. The length of this piece was then 5.1″. At a load of 105,000 lbs. the length was 5.4″. Measurements were not made beyond this point, the upper piece becoming imbedded into the lower pieces and the second layer of lead came into bearing.

Another test was made on a single piece of lead 3″ x 5″. At a load of 105,000 lbs. its length had increased to 5.4″ after holding the load for 3 minutes. At a load of 160,000 lbs. its length after 3 minutes was $6\frac{1}{4}''$ thus giving a load of about 8,500 lbs. per sq. inch on the new bearing area. In both tests the lead was observed to extrude through the clearance at the sides of the plate, but even under the heaviest loads applied equilibrium was established.

The lead bearing was used under the suspended span to avoid putting a bending stress in the end post as it rotated from its camber position to its position under full load. It is obvious that this condition would be fully attained if the pressure per square inch over the whole area provided for the lead was sufficient to cause it to flow, and from the results of the experiment it was thought that a pressure of 5,000 lbs. per square inch would be sufficient to give this condition.

The lead was therefore confined to a space 7-$\frac{3}{4}''$ x 3′ 0″, which, under the total dead load of 1,410,000 lbs. on each bearing (See Plate CI), would give a pressure of 5,050 lbs. per square inch, and was arranged as shown on plate LXXXVIII, the sheet metal angles ELG1—B13 being provided to prevent it from extruding through the clearance between the casting and the bearing plate.

When the bearing was removed after carrying the load for about six weeks it was found that the lead did not completely fill the space provided for it, from which it may be concluded that the object, of avoiding the bending stresses in the end post, was not completely attained and that while this bearing served the purpose for which it was designed better results would have been obtained if the area of the space provided for the lead had been reduced about 25 per cent.

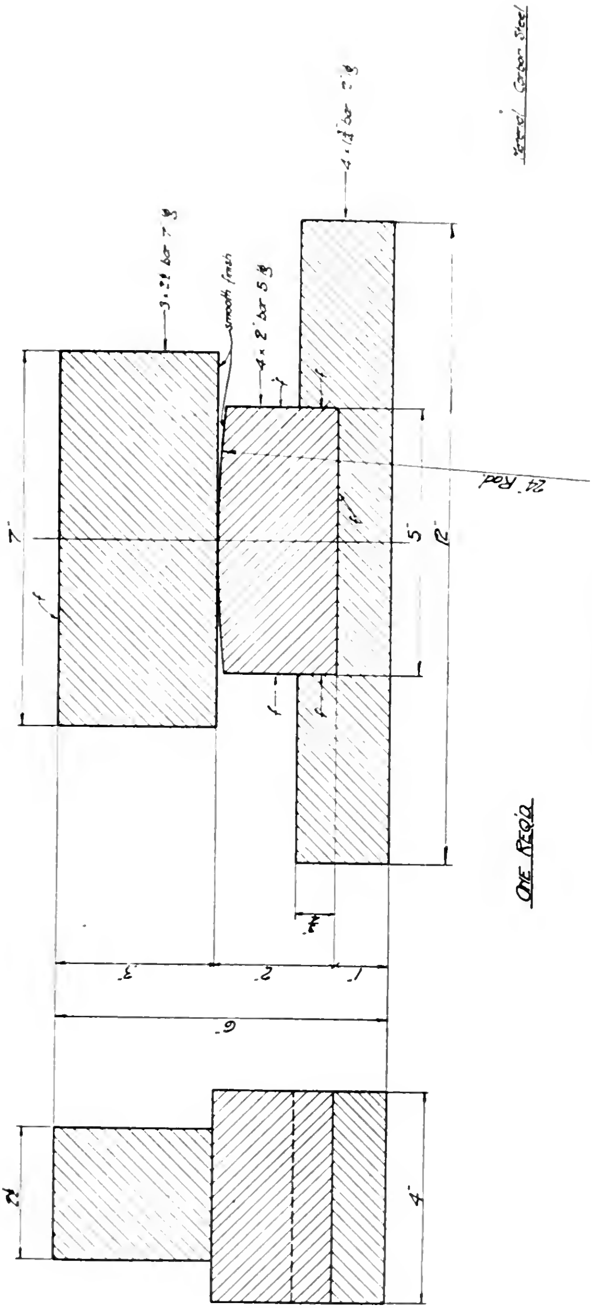
14.—TEST ON STEEL ROCKER BEARING

A full sized test was made at McGill University on a rocker bearing representing a short section of the bearing used under each corner of the suspended span to carry its weight while being lifted from the scows to its final position in the bridge (see Page 138, and Plates LXXXVIII and CI.) The specimen was made of ordinary rolled carbon-steel instead of rolled nickel steel as used for the bearings themselves. Four test pieces cut from the same stock as used for the specimen showed an average yield point of 30,500 and an ultimate strength of 63,500 lbs. per sq. inch. Figure No. 1 shows the dimensions of the test specimen and the results of the tests are summarized in the following Table 1.

TEST 14.—Table 1

Load Lbs.	Load Lbs. per Lineal Inch	Deformation 1/1000 inches		Width of Contact Inches	
		Convex Surface	Flat Surface	Convex Surface	Flat Surface
125,000	50,000	4.5	3.0	1.0	1.0
175,000	75,000	4.5	6.5	1.4	1.7
217,000	86,800	5.2	10.3	1.7	2.2

All measurements were made after the specimen had been removed from the machine and therefore represent the permanent set of the material. The width of contact given was clearly defined on the steel by reflected light. No measurements were made to obtain the amount of deformation or the width of contact under the load.



TEST 14—Figure 1

APPENDIX "B"

NOTES ON THE QUEBEC BRIDGE STRESSES

by

A. L. HARKNESS, B.A.Sc., A.M.E.I.C.

LIVE LOAD STRESSES

Specification:—The live load for which the bridge will be calculated is as follows,—

Train Load. Two Class E60 Engines, followed or preceded, or followed and preceded by a train load of 5,000 lbs. per foot per track, on one or two tracks. Where empty cars weighing 900 lbs. per lineal foot of track in any part of a train produce in any member larger strains than the uniform load of 5,000 lbs. such empty cars shall be assumed.

A Sidewalk Live Load of 500 lbs. per lineal foot for each of two sidewalks; to be used for members receiving their maximum stress from a length of moving load covering two panels or less.

The maximum live load stresses in the different members of the bridge and the condition of loading giving these stresses are shown on plate No. CIII.

The direct stresses, in the Anchor and Cantilever Arm Trusses, are entirely determinate as will be at once apparent from the outline diagram and by consideration of the fact that the member $CM_{14}M_{16}$ is built with a sliding connection at CM_{16} which makes it impossible for this member to carry direct stress.

Formulæ were developed expressing in terms of moments and elements of the truss, the stresses in all the members. Owing to the similarity of the different panels it was possible to arrange the calculations in tables thus greatly reducing the amount of labor involved. The formulæ used and characteristic cases showing the development of the formulæ and the method of tabulating the calculations for stresses will be given.

Notation: The stresses in the members are represented by the capital letters T, D, H, P, U, L and Q, their lengths by the small letters t, d, h, p, u, l and q, while the perpendicular distances from the point of moments to the members are represented by the italic letters *t, d, u, l* and *q*.

The loads are represented by the capital letters M, B, F, G, K, N and E. The small letter "m" represents the distance covered by the load .M.—b, f, g, k and n represent panel lengths and "e." represents the space covered by the locomotives. The italic letters m, b, f, g, k, n and e represent the distance from the points of moments to the centers of gravity of the different loads. The moments are represented thus Mm, Bb, Ff, Gg, Kk, Nn, and Ee.

R represents the end reaction from the Suspended Span.
r " " distance from panel point I to the end of the Cantilever arm.,
w " " weight of the uniform load = 5000 P.L.F.,
x " " variable distance from the point of moments to the locomotive,
a " " length of the Anchor Arm,
c " " " " " Cantilever Arm,
s " " " " " Suspended Span,
i the Angle between the top chord and the horizontal,
j " " " " bottom chord and the horizontal,
o the length of the end panel C_{1-0} of the Cantilever Arm,

z represents the ratio $\dfrac{p+h}{p_1+h_1}$

y represents the ratio $\dfrac{h_1}{(p+h)\ (f+g)}$

For purposes of reference each panel point was given a mark as shown on plates XXIX, XXXII and XXXIII. This mark preceded by "A" for the Anchor Arm, "C" for the Cantilever Arm and "X" for the Suspended Span, serves to locate any point in the trusses, as for example: AM_{12}, CU_4, XL_5. A member joining any two points is referred to by combining the marks for the two points, as for example AM_4U_6, CM_4M_5, XU_2L_4.

Moments: Nn, Kk, Gg, Ff and Bb represent the moments, about the panel point indicated, of all the loads to the left of that panel point, which produce stress in the member under consideration, e.g., when calculating the stress in the member "D_1", Fig. No. 1. Nn is the moment about panel point V of all the loads between panel points V and I. It is obvious that loads to the left of panel point I do not effect the stress in the member D_1.

N, K, G, F and B represent the loads and n, k, g, f and b the distance to their centers of gravity respectively.

Mm in Fig. I represents the moments about the panel point I of all the loads from that point to the end of the Cantilever Arm including the reactions at C_1 and XL_0 from the uniform live load on the Suspended Span. As before "M" represents the total load and "m" the distance to its center of gravity.

Mm in Figures 3 and 4 represents the moment about the far end of the Suspended Span, of all the loads from that point to the panel point in the Cantilever Arm beyond which they no longer effect the stress in the member under consideration.

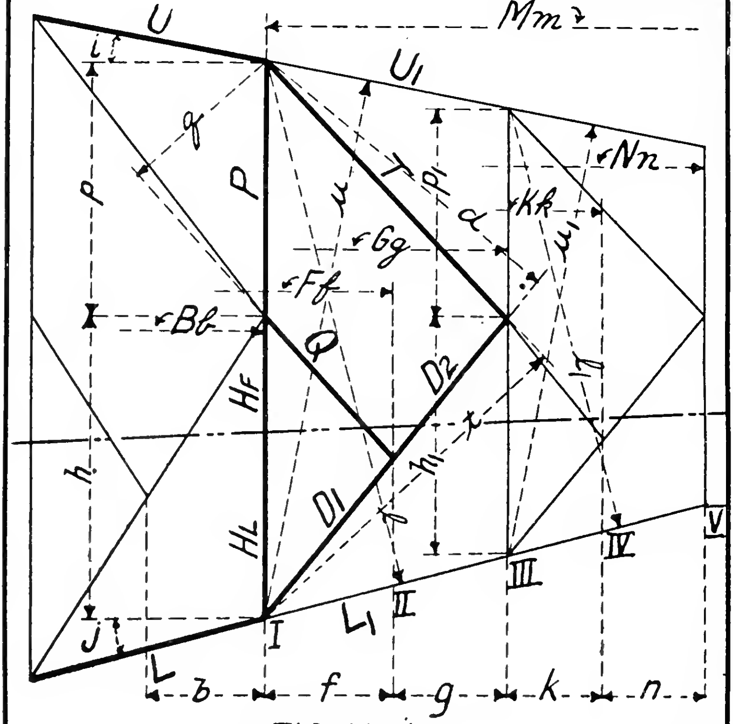


FIG. No. 1

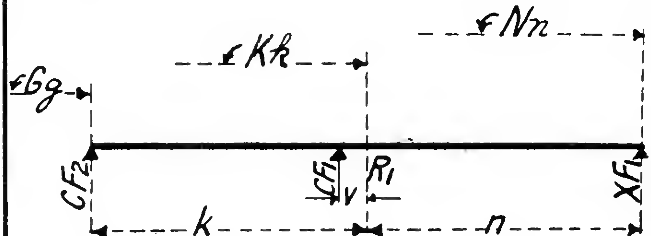


FIG. No. 2

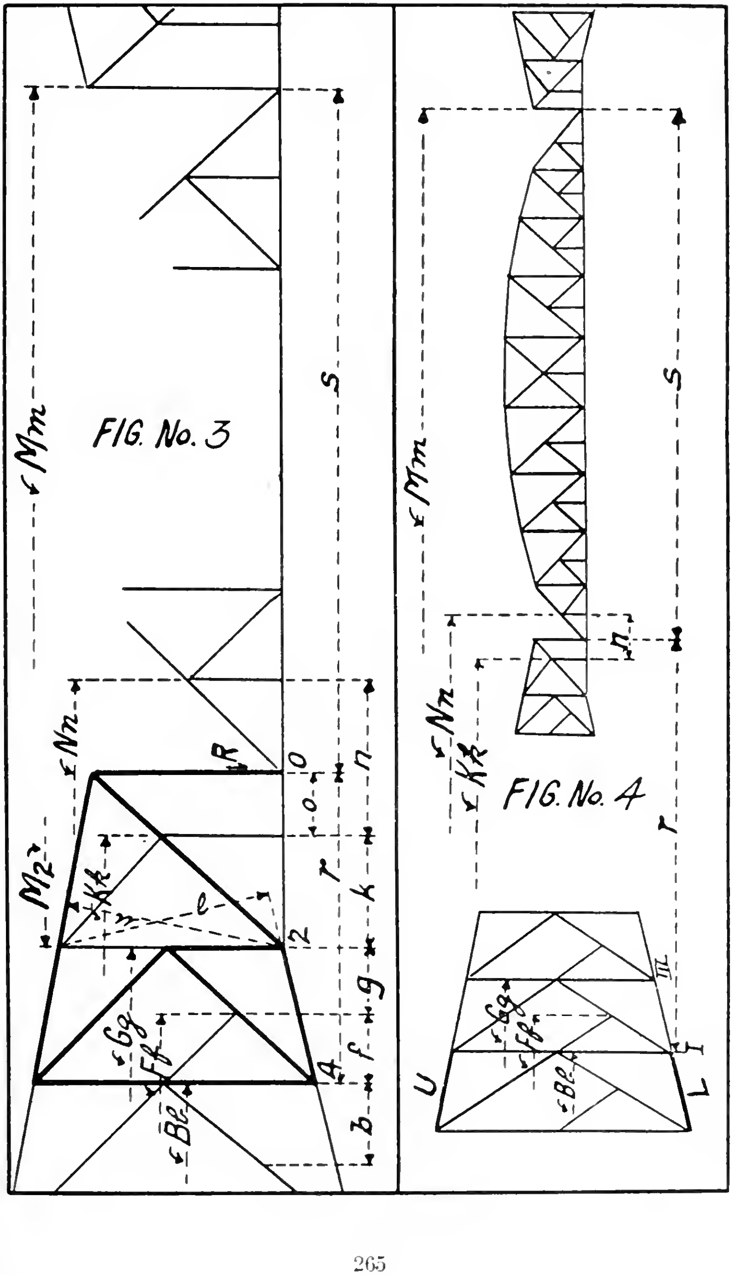


FIG. No. 3
FIG. No. 4
Mm
Nn
Kk
Gg
Ff
Bb
M2
s
r
o
n
k
g
f
b
A
O
R
U
L
I
III

CANTILEVER ARM

Development of Formulæ, —

STRESSES IN SUB MEMBERS.

Refer to Figure No. 1.

SUB HANGERS AND POSTS.

Let. R_{11} be the stress in the sub vertical $F_{11}M_{11}$

$$R_{11}=\frac{Gg—Kk—Kg}{g}+F—B—\frac{Ff—Bb—Bf}{f}$$

$$=Bb\,\frac{1}{f}\;—Ff\,\frac{f+g}{fg}\;+Gg\,\frac{1}{g}$$

SUB DIAGONALS.

$$Q=R_{11}\,\frac{q}{h}$$

Stress in Sub Member $M_1\,F_1$.

The variation in the length of the channel span structure, due to change of temperature and to the live load on the bridge, was provided for at the junction of the cantilever arms and the suspended span, the floorbeam "CF_1" moving with the Cantilever Arm and "XF_1" with the Suspended Span. To allow for this expansion and contraction the stringers in the panel "C_1X_1" were fixed at X_1 and were supported and free to slide on a bracket cantilevered out from the Floorbeam CF_1 (see plate XXVIII), the maximum distance from the center of bearing of the stringer to the center of the floorbeam being 2.15 feet.

In calculating the live load stresses in the sub members of the panel $C_{0\text{-}2}$ this condition was considered but was neglected when calculating the stresses in the main members, the panel C_1X_1 then being considered as of its normal length of 55.5 feet without regard to the eccentricity at "CF_1."

HANGER M_1L_1

Refer to Figure No. 2

$$M_1L_1\;=R\,\frac{k}{k-v}$$

$$R=Gg\,\frac{1}{k}—Kk\frac{k+n}{kn}\;+\;Nn\,\frac{1}{n} \qquad v=\;2.15\text{ feet}$$

$$M_1L_1=Gg\frac{1}{k—v}\;—Kk\;\frac{k+n}{n(k—v)}\;+Nn\;\frac{k}{n(k—v)} \qquad \begin{array}{l} k=44.15\;\;\text{"} \\ n=54.04\;\;\text{"}\end{array}$$

$$M_1U_2=M_1L_1\;\;\frac{\text{Length }M_1U_2}{\text{" }\;U_2L_2}$$

$$M_2U_2=M_1L_1\;\;\frac{\text{Length }M_1U_1}{\text{" }\;U_2L_2}$$

Main Truss Members: The development of the formulæ for the top and bottom chords and for the members M_4F_4 and M_1F_1 will be given as an illustration of the method followed.

BOTTOM CHORD

Refer to Fig. No. 4 and to panel I-III of Fig. No. 1

$$Ll=Rr+\frac{Nn-Kk-Kn}{n}(r-o)+K(r-o-k)+Qq$$

$$=Rr-Kk\left(1+\frac{r-o}{n}\right)+Nn\frac{r-o}{n}+Qq$$

$$R=\frac{1}{S}\left\{Mm-Kk-K(s+o)-\frac{Nn-Kk-Kn}{n}(s+o)\right\}$$

$$=\frac{1}{S}\left\{Mm-Kk\left(1-\frac{s+o}{n}\right)-Nn\frac{s+o}{n}\right\}$$

$$Q=R_{II}\frac{q}{h} \qquad q=\frac{pf}{q}$$

$$R_{II}=Bb\frac{1}{f}-Ff\frac{f+g}{fg}+Gg\frac{1}{g}$$

$$Qq=Bb\frac{p}{h}-Ff\frac{p}{h}\frac{f+g}{g}+Gg\frac{p}{h}\frac{f}{g}$$

$$Ll=Mm\frac{r}{s}-Kk\frac{r}{s}\left(1-\frac{s+o}{n}\right)-Nn\frac{r}{s}\frac{s+o}{n}-Kk\left(1+\frac{r-o}{n}\right)+Nn\frac{r-o}{n}$$

$$+Bb\frac{p}{h}-Ff\frac{p}{h}\frac{f+g}{g}+Gg\frac{p}{h}\frac{f}{g}$$

$$=Mm\frac{r}{s}+Bb\frac{p}{h}-Ff\frac{p}{h}\frac{f+g}{g}+Gg\frac{p}{h}\frac{f}{g}-Kk\frac{(n-o)(r+s)}{ns}-Nn\frac{o(r+s)}{ns}$$

Loads to the left of I do not effect the stress in the member L. Equation therefore becomes:—

$$Ll=Mm\frac{r}{s}-Ff\frac{p}{h}\frac{f+g}{g}=Gg\frac{p}{h}\frac{f}{g}-Kk\frac{(n-o)(r+s)}{ns}-Nn\frac{o(r+s)}{ns}$$

TOP CHORD:

$$Uu=Rr+\frac{Nn-Kk-Kn}{n}(r-o)+K(r-o-k)-R_{II}f$$

Loads to the left of II do not effect the stress in the member U

$$Uu=Mm\frac{r}{s}-Gg\frac{f}{g}-Kk\frac{(n-o)(r+s)}{ns}-Nn\frac{o(r+s)}{ns}$$

STRESS IN MEMBER M_4F_4 (Refer to Figs. Nos. 1 and 3) (consider panel 4-2 as panel I-III of Fig. No. 1).

$$H_F \doteq D_1 \frac{h_1}{d} + R_1$$

$$= D_1 d \frac{d}{(p+h)\ (f+g)} \frac{h_1}{d} + R_1$$

$$= D_1 d y + R_1$$

$$D_1 d = Rr + \frac{Nn - Kk - Kn}{n}(r-o) + K\ (r-o-k) + Bb - L_1 l + Qq$$

$$= Rr - L_1 l + Qq + Bb - Kk\ (1+\frac{r-o}{n}) + Nn\frac{r-o}{n}$$

$$L_1 l = L_1 l_1 \frac{p+h}{p_1+h_1} = L_1 l_1 z$$

$$= Rz\ (k+o) + R_1 zk$$

$$D_1 d = R\left\{r - z\ (k+o)\right\} - R_1 zk + Qq + Bb - Kk(1+\frac{r-o}{n}) + Nn\frac{r-o}{n}$$

$$R = Mm\frac{1}{s} - Kk\frac{1}{s}(1-\frac{s+o}{n}) - Nn\frac{1}{s}\frac{s+o}{n}$$

$$R_1 = Gg\frac{1}{k} - Kk\frac{k+n}{kn} + Nn\frac{1}{n}$$

$$Qq = Bb\frac{p}{h} - Ff\frac{p}{h}\frac{f+g}{g} + Gg\frac{p}{h}\frac{f}{g}$$

$$D_1 d = Mm\frac{1}{s}\left\{r - z\ (k+o)\right\} + Bb\ (1+\frac{p}{h}) - Ff\frac{p}{h}\frac{f+g}{g} + Gg\left\{\frac{p}{h}\frac{f}{g} - z\right\}$$

$$-Kk\left\{\frac{1}{s}(1-\frac{s+o}{n})\left\{r - z\ (k+o)\right\} - z\frac{k+n}{n} + (1+\frac{r-o}{n})\right\}$$

$$-Nn\left\{\frac{1}{s}\frac{s+o}{n}\left\{r - z\,(k+o)\right\} + z\frac{k}{n} - \frac{r-o}{n}\right\}$$

$$R_1 = \quad -Bb\frac{b+f}{bf} + Ff\frac{1}{f}$$

$$H_F = M_4F_4 = Mm\frac{y}{s}\left\{r - z\,(k+o)\right\} - Bb\left\{\frac{b+f}{bf} - y\ (1+\frac{p}{n})\right\}$$

$$+Ff\left\{\frac{1}{f} - y\frac{p}{n}\frac{f+g}{g}\right\} - Gg\ y\left\{z - \frac{p}{h}\frac{f}{g}\right\}$$

$$+Kk\frac{y\ (n-o)}{ns}\left\{z\ (s+o+k) - (s+r)\right\}$$

$$+Nn\frac{yo}{ns}\left\{z\ (s+o+k) - (s+r)\right\}$$

STRESS IN M_1F_1: Refer to Fig. No. 1.

$$H_F = D_1 \frac{h_1}{d} + R_1$$

$$= D_1 d \frac{1}{d} \frac{h_1}{d} + R_1 \qquad d = \frac{(h+p)\ (f+g)}{d}$$

$$= D_1 dy + R_1 \qquad \text{since } y = \frac{h_1}{(h+p)\ (f+g)}$$

$$D_1 d = Mm - L_1 l + Qq$$

$$L_1 l = L_1 l_1 z \qquad z = \frac{p+h}{p_1+h_1}$$

$$L_1 l_1 = Mm - M\ (f+g) + Gg - Bb - B\ (f+g) + Q_1 q_1$$

$$Q_1 q_1 = R\ \text{IV} \frac{p_1}{h_1} k$$

$$R\ \text{IV} = Gg \frac{1}{k} - Kk \frac{k+n}{kn} + Nn \frac{1}{n}$$

$$Q_1 q_1 = Gg \frac{p_1}{h_1} - Kk \frac{p_1}{h_1} \frac{k+n}{n} + Nn \frac{p_1}{h_1} \frac{k}{n}$$

$$L_1 l_1 = Mm - (M+B)\ (f+g) - Bb + Gg \left(1 + \frac{p_1}{h_1}\right) - Kk \frac{p_1}{h_1} \frac{k+n}{n}$$
$$+ Nn \frac{p_1}{h_1} \frac{k}{n}$$

$$L_1 l = Mmz - (M+B)\ z\ (f+g) - Bbz + Ggz \left(1 + \frac{p_1}{h_1}\right)$$
$$- Kkz \frac{p_1}{h_1} \frac{k+n}{n} + Nn\ z \frac{p_1}{h_1} \frac{k}{n}$$

$$Qq = Bb \frac{p}{h} - Ff \frac{p}{h} \frac{f+g}{g} + Gg \frac{p}{h} \frac{f}{g}$$

$$R_1 = -Bb \frac{b+f}{bf} + Ff \frac{1}{f}$$

$$H_F = y \left\{ Mm + Mmz + (M+B)\ z\ (f+g) + Bbz + Ggz \left(1 + \frac{p_1}{h_1}\right) \right.$$
$$\left. + Kkz \frac{p_1}{h_1} \frac{k+n}{n} - Nnz \frac{p_1}{h_1} \frac{k}{n} + Bb \frac{p}{h} - Ff \frac{p}{h} \frac{f+g}{fg} + Gg \frac{f}{g} \right\}$$
$$- Bb \frac{b+f}{bf} + Ff \frac{1}{f}$$

$$H_F = Mm\, y\ (1-z) + (M+B)\ yz\ (f+g) - Bb \left\{ \frac{b+f}{bf} - y \left(z + \frac{p}{h}\right) \right\}$$
$$+ Ff \left\{ \frac{1}{f} - y \frac{p}{h} \frac{f+g}{g} \right\} - Gg\, y \left\{ z \left(1 + \frac{p_1}{h_1}\right) - \frac{p}{h} \frac{f}{g} \right\}$$

$$+Kk\ y\ z\ \frac{p_1}{h_1}\ \frac{k+n}{kn}\ -\ Nn\ yz\ \frac{p_1}{h_1}\ \frac{k}{n}$$

GENERAL FORMULAE:

Formulae for all the members in the truss were developed as illustrated above and are as follows:— Refer to Fig. 1.

$$Tt = Mm(1-z) + Mz\ (f+g) - Gg\ \frac{f}{g} - Kkz\ \frac{k+n}{n} + Nnz\ \frac{k}{n}$$

$$P = T\ \frac{p_1}{t}$$

$$D_2d = Mm\ (1-z) + Mz\ (f+g) - Gg\left\{z\ (1+\frac{p_1}{h_1}) + \frac{f}{g}\right\} + Kkz\ \frac{p_1}{h_1}\ \frac{k+n}{n}$$

$$-Nn\ z\ \frac{p_1}{h_1}\ \frac{k}{n}$$

$$D_1d = Mm\ (1-z) + Mz\ (f+g) - Ff\ \frac{p}{h}\ \frac{f+g}{g} - Gg\left\{z\ (1+\frac{p_1}{h_1}) - \frac{p}{h}\ \frac{f}{g}\right\}$$

$$+Kk\ z\ \frac{p_1}{h_1}\ \frac{k+n}{n} - Nn\ z\ \frac{p_1}{h_1}\ \frac{k}{n}$$

$$H_L = D_1\ \frac{h_1}{d}$$

$$H_F = Mm\ y\ (1-z) + (M+B)\ y\ z\ (f+g) - Bb\left\{\frac{b+f}{bf} - y\ (z+\frac{p}{h})\right\}$$

$$+Ff(\frac{1}{f} - y\ \frac{p}{h}\ \frac{f+g}{g}) - Gg\ y\left\{z\ (1+\frac{p_1}{h_1}) - \frac{p}{h}\ \frac{f}{g}\right\}$$

$$+Kk\ y\ z\ \frac{p_1}{h_1}\ \frac{k+n}{n} - Nn\ y\ z\ \frac{p_1}{h_1}\ \frac{k}{n}$$

$$R_{II} = Bb\ \frac{1}{f} - Ff\ \frac{f+g}{fg} + Gg\ \frac{1}{g}$$

$$Q = R_{II}\ \frac{g}{h}$$

Refer to Figure No. 1 and to panel I-III of Figure No. 1.

$$Uu = Mm\ \frac{r}{s} - Gg\ \frac{f}{g} - Kk\ \frac{(n-o)\ (r+s)}{ns} - Nn\ \frac{o\ (r+s)}{ns}$$

$$Ll = Mm\ \frac{r}{s} - Ff\ \frac{p}{h}\ \frac{f+g}{g} + Gg\ \frac{p}{h}\ \frac{f}{g} - Kk\ \frac{(n-o)\ (r+s)}{ns} - Nn\ \frac{o\ (r+s)}{ns}$$

Refer to Figure No. 3:

$$U_oL_o = Mm\ \frac{1}{s} - Nn\ \frac{s+o}{ns}$$

$$U_oU_2 = U_oL_o \times \frac{\text{length } U_oU_2}{\text{" } U_2L_2}$$

$$U_oM_1 = U_oL_o \times \frac{\text{length } U_oL_2}{\text{" } U_2L_2}$$

$$\left.\begin{aligned} M_1L_2 &= M_2\frac{1}{d} \\ L_2L_4 &= M_2\frac{1}{l} \\ U_2U_4 &= M_2\frac{1}{u} \end{aligned}\right\} M_2 = Mm\,\frac{k+o}{s} - Kk\frac{(n-o)\ (s+o+k)}{ns} - Nn\,\frac{o\ (s+o+k)}{ns}$$

$$M_2F_2 = Mm\,\frac{y\ (k+o)}{s} - Gg\ \left(\frac{g+k}{gk} - y\right) - Kk\left\{\frac{1}{k} - \frac{y\ (n-o)\ (s+o+k)}{ns}\right\}$$
$$- Nn\,\frac{yo\ (s+o+k)}{ns}$$

$$M_2U_4 = T;\text{and } Tt = Mm\frac{1}{s}\left\{r - z(k+o)\right\} - Gg\left(\frac{f}{g} + z\right) + K_k\,\frac{(n-o)}{ns}\left\{z\ (s+o+r)\right.$$
$$\left. - (s+r)\right\} + Nn\,\frac{o}{ns}\left\{z\ (s+o+k) - (s+r)\right\}$$

$$U_4M_4 = M_2U_4\quad \frac{\text{Length } U_2M_2}{\text{"}\quad M_2U_4}$$

$$M_2M_3 = M_2U_4\quad \frac{\text{Length } M_2L_4}{\text{"}\quad M_2U_4}$$

$$M_3L_4 = D_1;\ \text{and } D_1d = M_m\frac{1}{s}\left\{r - z\ (k+o)\right\} - Ff\,\frac{p}{h}\,\frac{f+g}{g} - Gg\ \left(z - \frac{p}{h}\,\frac{f}{g}\right)$$
$$+ Kk\,\frac{n-o}{ns}\left\{z\ (s+o+k) - (s+r)\right\}$$
$$+ Nn\,\frac{k}{ns}\left\{z\ (s+o+k) - (s+r)\right\}$$

$$F_4L_4 = M_3L_4\quad \frac{\text{Length } M_2L_2}{\text{"}\quad M_2L_4}$$

$$M_4F_4 = Mm\ \frac{y}{s}\left\{r - z\ (k+o)\right\} - Bb\left\{\frac{b+f}{bf} - y\ \left(1 + \frac{p}{h}\right)\right\}$$
$$+ Ff\left\{\frac{1}{f} - y\frac{p}{h}\,\frac{f+g}{g}\right\} - Gg\ y\left\{z - \frac{p}{h}\ \frac{f}{g}\right\}$$
$$+ Kk\,\frac{y\ (n-o)}{ns}\left\{\ z\ (s+o+k) - (s+r)\ \right\}$$
$$+ Nn\,\frac{yk}{ns}\left\{\ z\ (s+o+k) - (s+r)\right\}$$

TABLE No. 1—COEFFICIENTS FOR THE FORMULAE

	Member	Refer to Fig. No.	Unit of Coeff.	COEFFICIENTS Mm	M or M+B	Bb	Ff	Gg	Kk	Nn	Lever Arm in ft.
SUB. VERTICALS	M1L1	2	10^{-3}	....	....	23.810	43.262	19.452	....	....	....
	M3F3	"	"	....	....	39.215	83.659	44.444	....	....	....
	M5F5	"	"	....	....	32.258	67.972	35.714	....	....	....
	M7F7	"	"	....	....	25.000	56.250	31.250	....	....	....
	M9F9	"	"	....	....	23.810	47.620	23.810	....	....	....
	M11F11	"	"	....	....	"	"	"	....	....	....
	M13F13	"	"	....	....	"	"	"	....	....	....
	M15F15	"	"	....	....	"	"	"	....	....	....
TOP CHORD	UoU2	3		UOLO	x .7966	....	....	....	....	....	....
	2–4	3	1	.10157	....	....	....	....	.6451	.4567	81.60
	4–6	1 & 4	1	.1766	....	....	....	1.1333	.6890	.4875	102.30
	6–8	"	1	.2687	....	....	....	1.1072	.7430	.5258	127.75
	8–10	"	1	.3812	....	....	....	1.2500	.8088	.5724	158.80
	10–12	"	1	.5123	....	....	....	1.0000	.8856	.6268	195.04
	12–14	"	1	.6438	....	....	....	1.0000	.9626	.6812	231.27
	14–16	"	1	.7750	....	....	....	1.0000	1.0394	.7356	267.51
BOTTOM CHORD	L2L4	3	1	.10157	....	....	....	....	.6451	.4567	81.05
	4–6	1 & 4	1	.1766	....	....	1.8685	.9928	.6890	.4875	101.61
	6–8	"	1	.2687	....	....	1.8415	.9677	.7430	.5258	126.89
	8–10	"	1	.3812	....	....	1.9630	1.0905	.8088	.5724	157.74
	10–12	"	1	.5123	....	....	1.7424	.8712	.8856	.6268	193.73
	12–14	"	1	.6438	....	....	1.7408	.8704	.9626	.6812	229.71
	14–16	"	1	.7750	....	....	1.7396	.8698	1.0394	.7356	265.70
TENSION DIAGONALS	M2U4	3	1	.04925	....	....	....	2.3871	.1198	.0848	73.25
	4–6	1	1	—.2485	73.66	....	....	1.1072	2.6635	1.4150	90.79
	6–8	"	1	—.2430	89.50	....	....	1.2500	2.6195	1.3762	111.85
	8–10	"	1	—.2282	103.17	....	....	1.0000	2.7638	1.5353	133.67
	10–12	"	1	—.1858	99.62	....	....	1.0000	2.3716	1.1858	143.38
	12–14	"	1	—.1565	97.15	....	....	1.0000	2.3130	1.1565	150.61
	14–16	"	1	—.1354	95.38	....	....	1.0000	2.2708	1.1354	156.06

TABLE No. 1—Continued

Compression Diagonals	UoM1	3	1	UoLo	x 1.1470	....	....	....	....	....	....
	M2M3	3	1	M2U4	x 1.0742	....	....	....	....	....	....
	4–5	1	1	—.2485	73.66	....	....	3.4494	2.3330	1.2395	84.33
	6–7	"	1	—.2130	89.50	....	....	3.5793	2.2895	1.2028	103.66
	8–9	"	1	—.2282	103.17	....	....	3.3000	2.4110	1.3393	123.36
	10–11	"	1	—.1858	99.62	....	....	3.2190	2.0662	1.0330	130.79
	12–13	"	1	—.1565	97.15	....	....	3.1633	2.0132	1.0067	136.14
	14–15	"	1	—.1354	95.38	....	....	3.1212	1.9752	.9876	140.08
	M1L2	3	1	.10157	....	....	....	....	.6451	.4567	56.67
	M3L4	3	1	.01925	....	....	1.8685	.2640	.1198	.0848	68.19
	5–6	1	1	—.2185	73.66	....	1.8414	1.3744	2.3333	1.2395	84.33
	7–8	"	1	—.2130	89.50	....	1.9630	1.2388	2.2895	1.2928	103.66
	9–10	"	1	—.2282	103.17	....	1.7424	1.4288	2.4110	1.3393	123.36
	11–12	"	1	—.1858	99.62	....	1.7408	1.3486	2.0662	1.0330	130.79
	13–14	"	1	—.1565	97.15	....	1.7396	1.2935	2.0132	1.0067	136.14
	15–16	"	1	—.1354	95.38	....	1.7567	1.2424	1.9752	0.9876	140.08
Hangers	UoLo	3	10^{-3}	1.5625	....	....	....	....	....	18.666	....
	M2F2	3	"	1.0258	....	....	....	58.155	17.295	4.611	....
	4–4	3	"	.4357	....	58.333	22.688	2.309	1.0627	.7484	....
	6–6	1	"	—1.7980	533.0	48.110	18.938	9.944	16.880	8.967	....
	8–8	"	"	—1.4488	533.6	36.200	13.300	7.386	13.650	7.172	....
	10–10	"	"	—1.1803	533.6	36.780	14.796	7.392	13.470	6.928	....
	12–12	"	"	— .9970	534.6	36.607	14.468	7.237	11.088	5.534	....
	14–14	"	"	— .8610	534.5	36.490	14.240	7.117	11.077	5.538	....
Hangers	F4L4	....	....	M3L4	x .6032						
	6–6	....	....	5–6	x .6101						
	8–8	....	....	7–8	x .6180						
	10–10	....	....	9–10	x .6386						
	12–12	....	....	11–12	x .7017						
	14–14	....	....	13–14	x .7492						
Posts	U2M2	....	....	M4F1	x .3539						
	4–4	....	....	M2U4	x .5693						
	6–6	....	....	4–6	x .5751						
	8–8	....	....	6–8	x .5822						
	10–10	....	....	8–10	x .6038						
	12–12	....	....	10–12	x .6702						
	14–14	....	....	12–14	x .7211						
Sub. Diagonals	M1U2	....	....	M4F1	x .6805						
	M3M4	....	....	M3F3	x .6555						
	5–6	....	....	5–5	x .6510						
	7–8	....	....	7–7	x .6412						
	9–10	....	....	9–9	x .6371						
	11–12	....	....	11–11	x .6035						
	13–14	....	....	13–13	x .5768						
	15–16	....	....	15–15	x .5588						

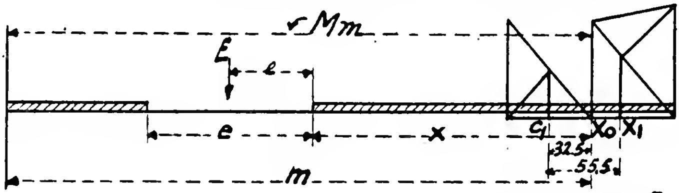


FIG. № 5

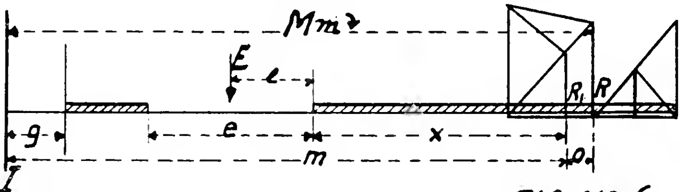


FIG. № 6

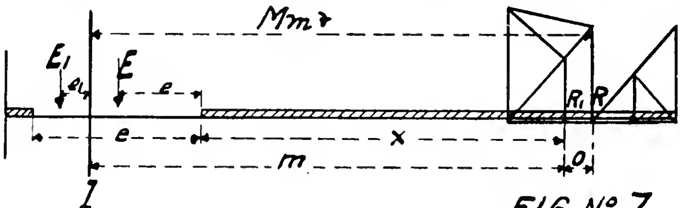


FIG. № 7

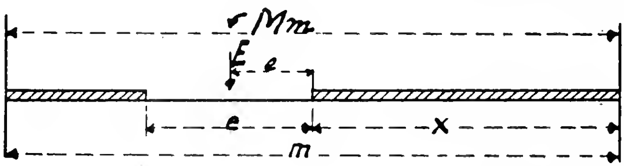


FIG. № 8

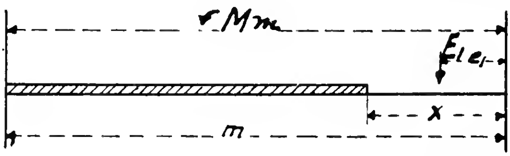


FIG. № 9

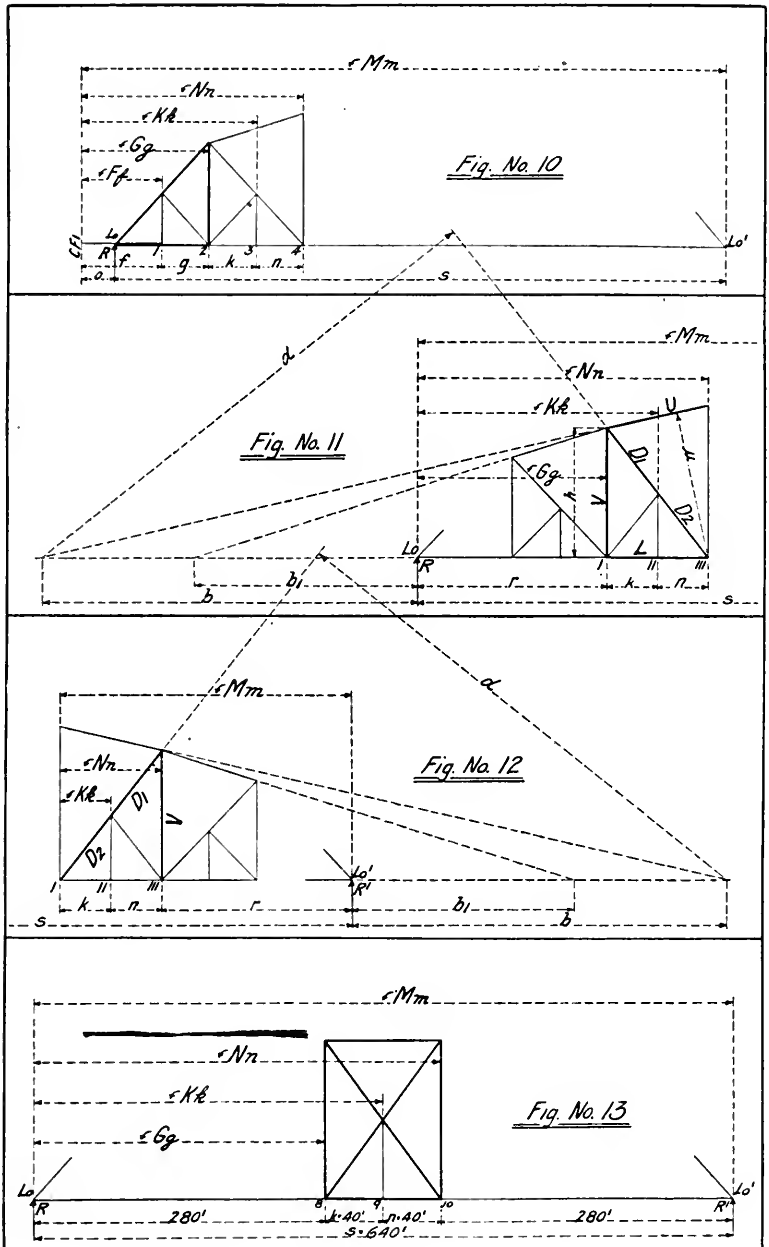
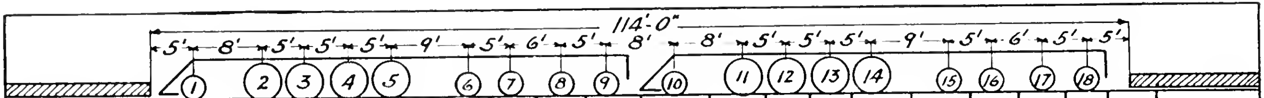


Fig. No. 10
Mm
Nn
Kk
Gg
Ff
Lo
R
f
g
k
n
o
s
Lo'
Fig. No. 11
Mm
Nn
Kk
Gg
U
D1
D2
V
L
Lo
R
b
b1
r
k
n
s
d
Fig. No. 12
Mm
Nn
Kk
D1
D2
V
Lo'
R'
k
n
r
b1
b
s
d
Fig. No. 13
Mm
Nn
Kk
Gg
Lo
R
Lo'
R'
280'
k·40'
n·40'
280'
s·640'



Distance from Wh 1	0	8	13	18	23	32	37	43	48	56	64	69	74	79	88	93	99	104	108
Axle Loads	30	60	60	60	60	39	39	39	39	30	60	60	60	60	39	39	39	39	
Sum of axle loads	30	90	150	210	270	309	348	387	426	456	516	576	636	696	735	774	813	852	852
Moments	0	240	690	1440	2490	4920	6465	8553	10488	13896	17544	20124	23004	26184	32448	36123	40767	44832	49092

MOMENT TABLE.

The Moments may be expressed as follows:—

1. For Figs. 3 and 4,

Refer to Figure No. 5,

Let "Mm" be the moment of all the loads between the points indicated,

Let "E" be the weight o the locomotives,

Let "e" be the distance to the c. of g. of the locomotives,

Let "w" be the uniform load per lineal foot of track.

Assume locomotives headed towards the main pier,

$$Mm = w\frac{m^2}{2} + (Ee - w\frac{e^2}{2}) + (E - we)\,x + w\,(\frac{55.5}{2} - \frac{32.5}{2})\,32.5$$

$$= w\frac{m^2}{2} + 282x + 18470.$$

$$\frac{dMm}{dx} = 282.$$

2. For Fig. No. 1,

Refer to Figure No. 6,

R = Reaction at the end of the Cantilever Arm from the uniform load on the Suspended Span

R_1 = Reaction at CF_1 from the Uniform load in panel C_1X_1.

$$Mm = \frac{w}{2}\,(m^2 - g^2) - we\left(m - x\,\frac{e}{2}\right) + E\,(m - x - e) + R_1m + R\,(m + o)$$

$$= \frac{w}{2}\,(m^2 - g^2) + (E - we)(m - x) - (Ee - w\frac{e^2}{2}) + (R + R_1)\,m + Ro$$

$$= \frac{w}{2}\,(m^2 - g^2) + 282\,(m - x) + 1715m + 19652.$$

$$\frac{dMm}{dx} = -282$$

3. For Figure No. 1

When g = o.

$$Mm = \frac{w}{2}\,m^2 + 282\,(m - x) + 1715\,m + 19652.$$

$$\frac{dMm}{dx} = -282$$

4. For Figure No. 1

Refer to Figure No. 7,

Let "E_1" be the weight of the locomotives to the left of panel point I.

Let "e_1" be the distance to the c. of g. of E_1.

$$Mm = \frac{w}{2}\,x^2 + wx\,(m - x) + E\,(m - x - e) + E_1e_1 + R\,(m + o) + R_1m$$

$$= wx\,(m - \frac{x}{2}) + E\,(m - x) - Ee + E_1e_1 + (R + R_1)\,m + Ro$$

$$= wx\,(m - \frac{x}{2}) + 852\,(m - x) + E_1e_1 + 1715m - 12838$$

$$\frac{dMm}{dx} = w\ (m - x - 852 + E_1$$

5. For Figure No. 1

Refer to Fig. No. 8,

$$Mm = w\ \frac{m^2}{2} + Ee + Ex - w\ \frac{e^2}{2} + w\ e\ x$$

$$= w\ \frac{m^2}{2} + Ee + (E - w\ e)\ x - w\ \frac{e^2}{2}$$

$$= w\ \frac{m^2}{2} + 282\ x + 16602$$

$$\frac{dMm}{dx} = 282$$

6. For Figure No. 1

Refer to Figure No. 9,

$$Mm = w\ \frac{m^2}{2} - w\ \frac{x^2}{2} + E_1 e_1$$

$$\frac{dMm}{dx} = E_1 - w\ x$$

7. Moment table for locomotives, page 276.

A moment table of the wheel concentrations of the locomotives was used from which the load E_1 and the moment E_1e_1 of Figures 7 and 9 could be easily obtained.

Maximum Stress in any member. The position of the locomotive giving the maximum stress in any member was found by means of an equation obtained by differentiating the equation for the stress in that member. The first differential is equal to zero and contains only expressions representing the loads and the coefficients for the moments, thus giving an equation which was very easily solved.

As an example refer to the calculations for the stress in the member M_4F_4, which follows, and to figures 5, 8 and 9.

The first differential of the equation for the stress in this member is:—

$$.4357 \times 282 - 58.333\ (E_B - wx) + 22.688\ (E_F - wx)$$
$$- 2.3090\ (E_G - wx) + 1{,}0627\ (E_K - wx) + .7484 + 282 = 0.$$

where E_B, E_F, E_G, and E_K represent the weight of the locomotives to the left of panel points 4, 3, 2 and 1 respectively, x the distance from the point of moments to the uniform load preceding the locomotives and $w = 5000$ lbs. per lineal foot of track.

To obtain the condition for the maximum stress in the member it is then only necessary to find a position of the locomotives such that the sum of the positive factors of the equation falls between the sums of the negative factors, obtained by first considering E_B as the weight of the locomotive to the left of the panel point 4 and second by considering that the weight E_B includes the weight of the wheel concentrated at that point.

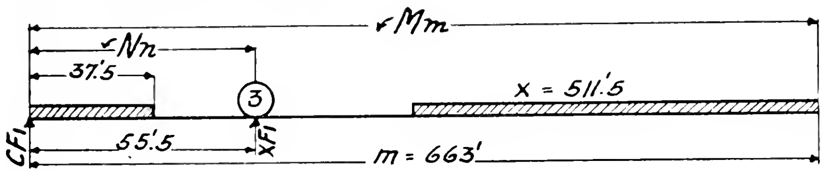
Stresses,—The full calculations for the stress in the members UoLo M_2L_2, M_4F_4 and the Hangers H_F (see Fig. 1) follow and will serve to illustrate the method followed for calculating the stresses in all the members of the truss.

The clause in the specification stating that the locomotives were to be followed or preceded or followed and preceded by the uniform load made it necessary to figure all the stresses twice, first with the locomotives headed towards the main pier, as herein, and second with the locomotives headed towards the center of the span, the maximum of these two conditions being taken as the max'mum live load stress in the member. The difference in the stresses thus obtained was found to be very slight and would hardly justify the additional amount of labor involved.

STRESS IN UoLo

(See Figure No. 3.)

$UoLo = 1.5625\ Mm - 18.666\ Nn$



CRITERION:

(See Figures Nos. 5 and 9.)

$1.5625 \times 282 = \underline{\underline{441.}}$

$$\begin{array}{rr} 90 & 150 \\ \underline{90} & \underline{90} \\ (\ 0, & 60) \times 19.06 = \underline{\underline{0,}}\ \ \underline{\underline{1140.}} \end{array}$$

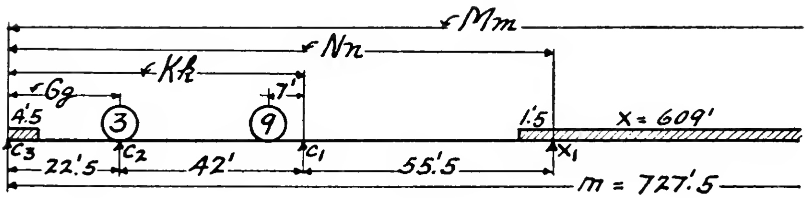
STRESS

$$\begin{array}{lrl} w\dfrac{m^2}{2} & = 1098920 & \\ 282x = & 144240 & \\ & \underline{18470} & \\ Mm & = 1261630 \times 1.5625 = & 1971.2 \\ & 690 & \\ & \underline{6890} & \\ Nn & 7580 \times 18.666 = & \underline{141.5} \\ & & 1829.7 = UoLo \end{array}$$

Stress in M^2F^2

(Refer to Figure No. 3.)

$M_2F_2 = 1.0258\ Mm - 58.155\ Gg + 17.295\ Kk - 4.611\ Nn$



Criterion:

See Figures Nos. 5, 8 and 9.

282×1.0258 = 289	90	150			
426	90	90			
300	(0	60)×	58.155 =	0000	3480
126×17.295 = 2180			282× 4.611 =	1300	1300
2469				1300	4780

Stress

$w\frac{m^2}{2} = 1323140$

282x = 171740

18470

Mm 1513350×1.0258 = 1552.2

10488

2982

1400

Kk 14870×17.295 = 257.2 1809.4

690

456

Gg 1146×58.155 = 66.6

$w\frac{m^2}{2}$ 36000

282+1.5 423

16602

Nn 53025×4.611 = 244.5 311.1

1498.3 = M_2F_2

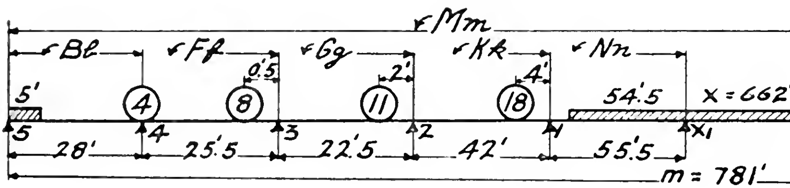
STRESS IN M_4F_4

(See Figure No. 3.)

$$M_4F_4 = .4357\ Mm - 58.333\ Bb + 22.688\ Ff - 2.3090\ Gg + 1.0627\ Kk + .7484\ Nn$$

CRITERION:

282 × .4357 = 122					
387		150	210		
242.5		115	115		
144.5 × 22.688 = 3280		(35	95) × 58.333 = 2040		5550
852		516			
565		355			
287 × 1.0627 = 305		161 × 2.309		= 372	372
282 × .7484 = 211 =	3918			2412	5922



Bb	Ff	Gg	Kk	Nn
1440	8553	17544	44832	75255
637	194	1032	3408	15370
2077	1275	1837	2887	16602
	10022	20413	51127	107227

$$w\frac{m^2}{2} = 1524900$$

$282x = 186684$

18470

$Mm = 1730054 \times .4357 = 753.8$

$10022 \times 22.688 = 227.4$

$51127 \times 1.0627 = 54.3$

$107227 \times .7484 = 80.3$ 1115.8

$2077 \times 58.333 = 121.2$

$20413 \times 2.3090 = 47.1$ 168.3

$947.5 = M_4F_4$

TABLE No. 2.—MAXIMUM LIVE LOAD STRESS H_F

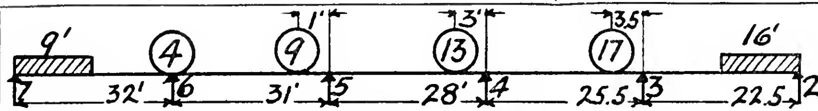
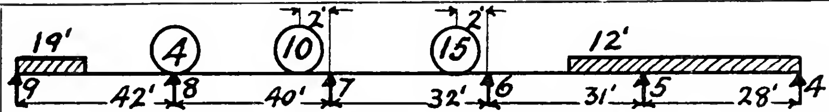
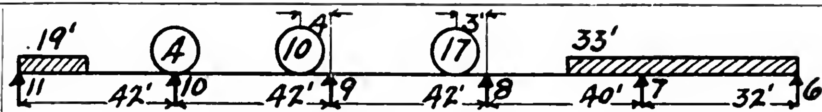
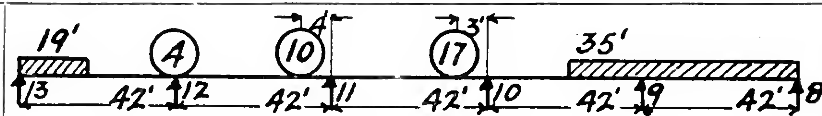
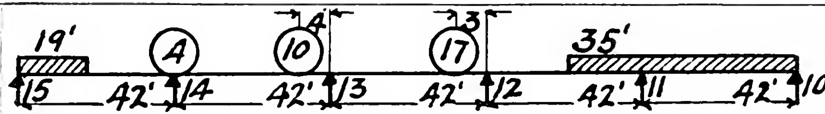
Member	m m − x x $m - \frac{x}{2}$ 5x	5 × (m—x) 852 (m—x) 1715m E_1e_1 Sum −12838 Mm	M+B	Bb	Ff	Gg	Kk	Nn	H_F
M_6F_6	149 91 58 120 290	34800 77530 255550 1440 369320 12838 356482 −1.7980	 2902 +533.0	1440 1238 2678 −48.140	10488 426 2635 13549 +18.938	23004 1908 3895 28807 —9.944	40767 2846 5040 48653 +16.880	48303 4512 16602 69417 − 8.967	1546.7 256.6 821.2=2624.5 640.9 128.9 286.5 622.4=1678.7 945.8
M_8F_8	221 91 130 156 650	101400 77530 379000 1440 559370 12838 546532 −1.4488	 3312 +533.6	1440 3088 4528 —36.200	13896 912 6887 21695 +13.300	32448 1470 9928 43846 −7.386	55063 3384 16602 75049 +13.650	75049 28196 1960 105205 −7.172	1767.0 288.5 990.3=3045.8 791.8 163.9 323.8 736.6=2016.1 1029.7

TABLE No. 2—Continued

M10F10	305 91 214 198 1070	211860 77530 523050 1440 813880 12838 801042 −1.1803							
			3732 +533.6	4528 −36.780	13896 1824 7077 22797 +14.796	40767 2439 11068 54274 −7.392	68890 9306 16602 94798 +12.470	94798 35584 2560 132942 −6.928	1991.2 337.6 1182.0=3510.8 945.5 166.5 401.2 921.0=2434.2 1076.6
M12F12	389 91 298 240 1490	357600 77530 667200 1440 1103770 12838 1090032 −0.9970							
			4152 +534.6	4528 −36.607	22797 +14.468	54274 −7.237	70560 9870 16602 97032 +11.088	97032 47124 4410 148566 −5.543	2219.5 329.9 1075.8=3625.2 1087.7 165.7 392.8 823.4=2469.6 1155.6
M14F14	473 91 382 282 1910	538620 77530 811200 1440 1428790 12838 1415952 −0.8610							
			4572 +534.5	4528 −36.490	22797 +14.240	54274 −7.117	97032 +11.077	148566 − 5.538	2443.5 324.6 1074.6=3842.7 1219.0 165.2 386.4 822.7=2593.3 1249.4

ANCHOR ARM

The live load stresses in the members of the Anchor Arm were calculated for the following conditions of loading.

Case I The Locomotives and the uniform load of 5000 P.L.F. on the channel span with the locomotives placed to give the maximum uplift on the anchorage. No load on the Anchor Arm.

Case II A uniform load of 5000 P.L.F. on the channel span. No load on the Anchor Arm.

Case III A uniform load of 5000 P.L.F. from the outer end of the Anchor Arm to a point giving the maximum stress in the member under consideration and a uniform load of 900 P.L.F. (representing unloaded cars) on the balance of the Anchor Arm. No load on the channel span.

Case IV The locomotives on the Anchor Arm in the position giving the maximum stress in the member under consideration, with a uniform load of 5000 P.L.F. extending from the outer end of the Anchor Arm to the locomotives and a uniform load of 900 P.L.F. from the locomotives to the main pier.

Case V The live load on one track only. The tracks are spaced 32' 6" c. to c. while the trusses are spaced 88 ft. c. to c. hence under a live load on one track only the one truss receives 68.47% of the load and the other truss the balance, i.e., 31.53%. Under this difference of loading the bottom chord of the one truss shortens more than the chord of the other truss and this tends to throw the outer end of the bottom chord lateral system off the center line. This tendency is restrained by the wind anchorage pin (see plate XXVIII) and thus a shear in the bottom lateral system is produced resulting in stresses in the bottom laterals and a decrease in the stresses of the more heavily loaded chord and an increase in the other chord. This condition of loading gives the only live load stress in the bottom laterals but in no case gives a maximum stress in the chords.

The maximum stress in the truss members is produced by either one of the conditions of loading described above or by a combination of them. Case I gives the maximum stress in the chords, the Anchorage eyebars and some of the members towards the outer end of the Anchor Arm. Combining cases 1 and III, gives the maximum stress in the Diagonals M_3L_4, M_4M_5 and M_5M_6, and in the hangers F_4L_4 and F_6L_6. Combining cases II and IV gives the maximum stress in the remainder of the web members.

Formulæ, for calculating the stresses, similar to those used for the Cantilever Arm were developed. The method used was, with but minor changes, the same and reference will therefore be made only to the points in which they differ.

Formulae: Refer to Figure No. 1 and consider it as a part of the Anchor Arm. The notation used for the Anchor Arm is the same with the exception of the moments. Mm is used to represent the moment about the panel point Ao, of all the loads on the Anchor Arm. Nn, Kk, Gg, Ff and Bb are used to represent the moments about the panel points indicated, of all the loads from that panel point to the vertical post over the main pier. "r" is used to represent the distance from the panel point "Ao" to the panel point I of Figure I and "a" to represent the length of the Anchor Arm = 515 feet.

As an illustration the formula for the member D_1 (Figure I) will be given:—

$$D_1 d = Mm\frac{1}{a}\left\{z\,(a-r+f+g)-(a-r)\right\}-Nn\,\frac{p_1}{h_1}\,z\,\frac{k}{n}+Kk\,\frac{p_1}{h_1}\,z\,\frac{k+n}{n}$$
$$-Gg\left\{z\left(1+\frac{p_1}{h_1}\right)-\frac{p}{h}\frac{f}{g}\right\}-Ff\,\frac{p}{h}\,\frac{f+g}{g}+Bb\left(1+\frac{p}{h}\right)$$

Similar formulæ were developed for all the members and were used for calculating the stresses for both Case III and Case IV.

Calculations: The calculations for the stress in the member M_9L_{10} for both Cases III and IV follow and will serve as an illustration of the method used throughout the Anchor Arm.

Substituting the value of the coefficients the formula for this member becomes:—

$$M_9L_{10}=0.2737Mm-1.4480Nn+2.4820Kk-1.4006Gg-1.7188Ff$$
$$+1.8594Bb$$

Case III Let w represent the uniform load of 5000 P.L.F.
" w_1 " " " " 900 "
" x " " distance from the panel point Ao to the end of the 5000 lb. load.

The moments may then be expressed as follows,—

$$Mm=\frac{w_1}{2}a^2+\frac{w-w_1}{2}x^2+\frac{w}{2}(50.5-23)\,23=\frac{w-w_1}{2}x^2+120930$$

$$\frac{dMm}{dx}=(w-w_1)\,x$$

$$Nn=\frac{w_1}{2}\,(a-r+f+g+k+n)^2+\frac{w-w_1}{2}\,x-r+f+g+k+n)^2$$

$$\frac{dNn}{dx}=(w-w_1)\,(x-r+f+g+k+n)$$

$$Kk=\frac{w_1}{2}\,(a-r+f+g+k)^2+\frac{w-w_1}{2}(x-r+f+g+k)^2$$

$$\frac{dKk}{dx}=(w-w_1)\,(x-r+f+g+k)$$

$$Gg = \frac{w1}{2}(a-r+f+g)^2 + \frac{w-w1}{2}(x-r+f+g)^2$$

$$\frac{dGg}{dx} = (w-w_1)\ (x-r+f+g)$$

$$Ff = \frac{w1}{2}(a-r+f)^2 + \frac{w-w1}{2}(x-r+f)^2$$

$$\frac{dFf}{dx} = (w-w_1)\ (x-r+f)$$

$$Bb = \frac{w1}{2}(a-r)^2 + \frac{w-w1}{2}(x-r)^2$$

$$\frac{dBb}{dx} = (w-w_1)\ (x-r)$$

Maximum Stress: The condition of loading producing the maximum stress in the member may be found by placing the first differential of the equation equal to zero and solving for .x.

Since the concentrations at all panel points AF_1 to AF_9 inclusive, produce compression in the member M_9L_{10} and all concentrations AF_{10} to AF_{12} inclusive produce tension, it is obvious that the uniform load "w" ends somewhere between AF_9 and AF_{10}.

The equation for the maximum stress in the member therefore is—

$x\ (0.2737-1.4480+2{,}4820-1.4006-1.7188)+191 \times 1.4480$
$-221 \times 2.4820+263 \times 1.4006+305 \times 1.7188=0$
or $x=342.7$.

Having obtained the value of "x" the stress in the member may then be obtained as follows:

Mm	$=2.05 \times 342.7 = 240760$			
	$120930 = 361690 \times 0.2737 =$		99000	
Nn	$.45 \times 324^2 = 47240$			
	$2.05 \times 151.7^2 = 47180 = 94420 \times 1.4480 =$			136720
Kk	$= .45 \times 294^2 = 38900$			
	$2.05 \times 121.7^2 = 30360 = 69260 \times 2.4820 =$		171900	
Gg	$= .45 \times 252^2 = 28575$			
	$2.05 \times 79.7^2 = 13025 = 41600 \times 1.4006 =$			58270
Ff	$= .45 \times 210^2 = 19845$			
	$2.05 \times 37.7^2 = 2915 = 22760 \times 1.7188 =$			39120
Bb	$= .45 \times 168^2 = \ldots\ldots\ 12700 \times 1.8594 =$		23615	
			294515	234110
			234110	
		$D_1d =$	60405	
		$d =$	129.16	
		$M_9L_{10} =$	467. 7	

Combining Case I and Case III the total stress in the member for this condition of loading becomes:

Case I $=711.0$
Case III $=467.7$

1178.7

Case IV. The moments may be expressed as follows:—

1. Refer to Figure 5 and

let $m=a=515$ feet
" X_o represent A_o of the Anchor Arm
" X_1C_1 be the stringer panel between the Anchor Arm and the Approach Span $=50.5$ feet,

let x be the length of the uniform load of w ($=5000$ P.L.F.) on the Anchor Arm.

Then $(m—x—e)=(a—x—e)$ will be the length of the uniform load w_1 ($=900$ P.L.F.)

$$\text{Then } Mm=w_1\frac{a^2}{2}+(w-w_1)\frac{x^2}{2}—w_1e\left(x+\frac{e}{2}\right)+Ee+Ex+\frac{w}{2}(50.5-23)23$$

$$=(w-w_1)\frac{x^2}{2}—749.4\ x+164170$$

$$\frac{dMm}{dx}=4.1\ x+749.4$$

2. Refer to Figure 8 and

let m = the distance from the point of moments to the post over the main pier.
let x = the length of the uniform load w
Then m—x—e " " " " " w_1

$$Mm=w_1\frac{m^2}{2}+(w—w_1)\frac{x^2}{2}—w_1e\left(x+\frac{e}{2}\right)+Ee+Ex$$

$$=(w—w_1)\frac{x^2}{2}+w_1\frac{m^2}{2}+749.4\ x+43244$$

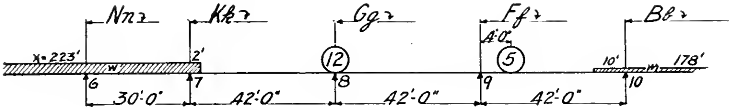
$$\frac{dMm}{dx}=4.1x+749.4$$

3. Refer to Figure 9 and
let m = the distance from the point of moments to the post over the main pier.
let x = the distance from the point of moments to the uniform load w_1.

$$Mm=\frac{w_1}{2}(m^2—x^2)+E_1e_1.$$

$$\frac{dMm}{dx}=E_1—w_1x.$$

Calculations for the member M_9L_{10}.



Mm	Nn	Kk	Gg	Ff	Bb
			509.4		
1663.7	880.6	757.6	449.4	241.2	0

1663.7 x 2737 = 455	880.6 x 1.4480 = 1270	1270
757.6 x 24820 = 1880	449.4 x 1.4006 = 630	
2335	509.4 x 1,0006 =	715
	241.2 x 1,7188 = 415	415
	2315	2400

Moments:

Mm	Nn	Kk	Gg	Ff	Bb
	2100	16			
101940	47230	38900		2490	
167120	23980	1500	20124	1080	
164170	43240	43244	26112	19384	
433230	116550	83660	46236	22954	12700

433230 x .2737 = 118570
83660 x 2.4820 = 207650
12700 x 1.8594 = 23620 = 349840

116550 x 1.4480 = 168780
46236 x 1.4006 = 64760
22954 x 1.7188 = 39450 = 272990

76850 = D_1d
129.16 = d
595 = M_9L_{10} Case IV
653 = M_9L_{10} " II
1248 = Maximum live load stress in the member.

SUSPENDED SPAN

The formulae used for calculating the stresses in the truss members of the Suspended Span and the value of the coefficients for the different members will be given.

Figs 10, 11, 12, & 13.

Refer to Fig. No. 10 note, $g = k = n$

$$L_0M_1 = \left\{ Mm\frac{1}{s} - Ff\,\frac{s+o}{fs} \right\} \frac{\text{length } L_0U_2}{\text{" } U_2E_2}$$

$$L_0L_2 = \left\{ Mm\frac{1}{s} - Ff\,\frac{s+o}{fs} \right\} \frac{\text{length } L_0L_2}{\text{" } U_2L_2}$$

$$M_1U_2 = \left\{ Mm\frac{1}{s} - Ff\left(\frac{s+o}{fs} - \frac{f+g}{2fg} \right) - Gg\frac{1}{2g} \right\} \frac{\text{length } L_0U_2}{\text{" } U_2L_2}$$

$$U_2L_2 = Ff\,\frac{f-g}{2fg} - Gg\,\frac{g+k}{2gk} + Nn\,\frac{1}{2n}$$

Refer to Fig. No. 11 note, $k = n$

$$D_1d = Mm\frac{b}{s} - Nn\,\frac{b+r}{2n}$$

$$D_2d = Mm\frac{b}{s} + Kk\left(\frac{b+r}{n} + 2 \right) = Nn\left(\frac{b+r}{n} + 1 \right)$$

$$V = Mm\frac{b_1}{s\,(b_1+r)} - Nn\frac{1}{2n}$$

$$Uu = Mm\frac{r+k+n}{s} - Nn$$

$$Lh = Mm\,\frac{r}{s} - 2Kk + Nn$$

Reversals,— Refer to Fig. No. 12.

$$D_1d = Mm\,\frac{s+b}{s} - Nn\left(\frac{b+r}{2n} + 1 \right)$$

$$D_2d = Mm\,\frac{s+b}{s} - Kk\left(\frac{b+r}{n} + 2 \right)$$

$$V = Mm\frac{s+b_1}{s\,(b_1+r)} - Nn\,\frac{1}{b_1+r}\left\{ \frac{b_1+r}{2n} + 1 \right\}$$

CENTRE PANEL

The rivets connecting the members M_9L_{10} and M_9L_{10} to the gusset plates at L_8 and L_{10} were not driven until after the truss had been lowered from its camber position and was supported entirely on the staging bents under the four corners, L_0. Hence these members çarry no dead load stresses; and the dead load stresses in the other members of this panel are determinate. With the members all connected into the truss system this panel has a redundant member and the live load stresses were therefore calculated according to the elastic deformation of the members. The equations for the different members are given in the table of coefficients.

TABLE No. 3

	Member	Refer to Fig. No.	COEFFICIENTS Unit = 10^{-3}				
			Mm	Ff	Gg	Kk	Nn
Bottom Chord	L_0L_2	10	1.451	17.699			
	2–4	11	1.451			28.572	14.286
	4–6	11	2.245			22.100	11.050
	6–8	11	2 991			19.140	9.570
	8–10	13	+4.452		+1.696	—12.297	+1.696
Top Chord	U_2U_4	11	2.354				11.586
	4–6	11	3.048				9.759
	6–8	11	3.957				9.112
	8–10	13	+4.638		—1.687	—5.905	—1.687
Main Diagonals	L_0M_1	10	2.132	26.011			
	M_1U_2	10	2.132	7.281	20.986		
	U_2M_3	11	1.166				16.240
	M_3L_4	11	1.166			41.988	37.232
	U_4M_5	11	1.220				15.656
	M_5L_6	11	1.220			36.155	33.733
	U_6M_7	11	1.625				14.966
	M_7L_8	11	1.625			31.508	30.721
	U_8M_9	13	—1.126			+10.040	+2.868
	M_9L_8	13	+0.806			—20.862	+2.861
Main Verticals	U_4L_4	11	0.855				14.286
	U_6L_6	11	0.964				12.500
	U_8L_8	13	+0.640			—8.100	—2.340
REVERSALS — Diagonals	U_2M_3	12	5.922				20.994
	M_3L_4	12	5.922			41.988	
	U_4M_5	12	3.642				18.078
	M_5L_6	12	3.642			36.155	
	U_6M_7	12	2.412				15.754
	M_7L_8	12	2.412			31.508	
	U_8M_9	13	+0.806				—12.589
	M_9L_8	13	—1.126				+18.318
REVERSALS — Verticals	U_4L_4	12	4.338				17.771
	U_6L_6	12	2.886				14.414
	U_8L_8	13	—1.003				+10.802
Sub-Hangers	U_2L_2	10		6.376	30.770		15.385
	M_1L_1	10		49.287	30.770		
	M_3L_3	10			30.770	61.538	30.770
	M_5L_5	11			28.571	57.143	28.571
	M_7L_7	11			25.000	50.000	25.000
	M_9L_9	12			25.000	50.000	25.000
Sub-Diagonals	L_2M_1				M_1L_1 x	.6823	
	L_2M_3				M_3L_3 x	.6823	
	L_4M5				M_5L_5 x	.6321	
	L_6M_7				M_7L_7 x	.6297	

DEAD LOAD STRESSES

Specification:—

The dead load for which the bridge will be calculated is as follows),—

The weight of all material remaining in the completed bridge and a snow load of 100 lbs per lineal foot of each track, 150 lbs per lineal foot of each sidewalk and 150 lbs per lineal foot of each bottom chord of the cantilever and anchor arms.

The weight of railway floor above stringers to be assumed at 600 lbs per lineal foot of each track and the weight of sidewalk floor above stringers to be assumed at 200 lbs per lineal foot of each sidewalk.

The dead load stresses were revised several times as the design developed. The final stresses, as used for determining the sections of the members and the weight of the material producing these stresses and its distribution to the different panel points, are given on plates Nos. CVI, CIX, CXIV. The weight of the steel given was estimated from preliminary shop drawings and sketches and to the weight thus obtained an addition of 2% was made to allow for further increase in weight as the shop drawings were completed and to allow for overrun in the weight of the steel. It has developed however that the 2% addition was just about sufficient to cover the increase in the final estimated weight as taken from the approved shop drawings and that the weight of the steel in the completed structure is about .65% in excess of the weight for which the stresses were calculated.

Table No. 3 gives the estimated weight, including the 2% addition, used for calculating the stresses and also the shipping weight. It will be noted that a large percentage of the increase occurs at points where it would not effect the stresses in the main structure.

Table No. 3 also gives the percentage of nickel steel used in the structure. The floor, swaybracing and top chord supporting trusses were built of carbon steel while the truss members and laterals were built of carbon or nickel steel as noted on plates XXIX, XXXII and XXXIII.

The calculations for the stresses in the Cantilever Arm will be given, those for the Anchor Arm are similar and were obtained by the same formulæ with the necessary changes in the coefficients. The only difference was that it was first necessary to obtain the reaction on the anchorages from the weight of the channel span, and then consider it as a concentrated load. The calculations for the suspended span were made in the usual way. The center panel was made determinate by leaving the connection at one end of the lower diagonals open until after the span was completely erected and resting on the end supports, thus insuring that this member would carry no dead load stress.

TABLE No. 4

	CONTRACT	Estimated Weight Used For The Dead Load Stresses					Shipping Weights
		S. Span	C. Arm	A. Arm	App. Spans	Total	
97	Field Bolts						33
101	Main Anchorage			1491		1491	1457
102	Wind Anchorage			120		120	117
103	App. Spans, Trusses and Bracing				1737	1737	1699
104	Eye Beam Stringers	265	540	465	165	1435	1412
105	Sub Floorbeams	187	384	340	185	1096	1064
106	Main Stringers	984	1760	1571	564	4879	4814
107	Main Floorbeams	1715	3667	3310		8692	8655
108	Floor Laterals	144	267	252	53	716	686
109	C. S. Grillage—Main Shoe			1163		1163	1261
110	Forged Sleeves			455		455	422
111	Main Shoe			2128		2128	2099
112	Main Post			10049		10049	9656
113	Link at top of Main Post			1069		1069	1164
114	Bottom Chord, A. Arm			11135		11135	11182
115	Diag. Comp. Members, "			9557		9557	9291
116	" Tens. " "			4540		4540	4373
117	Vert. Comp. " "			3219		3219	3104
118	" Tens. " "			2896		2896	2786
119	Riv. Sub. " "			779		779	804
120	Portal, "			612		612	712
121	Laterals, "			1001		1001	1177
122	Sway Bracing, "			3022		3022	3191
123	Bottom Chord, C. Arm		10073			10073	9893
124	Diag. Comp. Members, "		8154			8154	8130
125	" Tens. " "		3793			3793	3747

126	Vert. Comp. Members, C. Arm	...	3484	...	...	3484	3413
127	" Tens. " "	...	2432	...	...	2432	2327
128	Riv. Sub. " "	...	957	...	...	957	912
129	Laterals, "	...	1409	...	...	1409	1426
130	Sway Bracing, "	...	2881	...	...	2881	2913
131	Top Chord Supporting Trusses	...	914	952	...	1866	1939
132	Carbon Eyebars	...	5969	7511	...	13480	13617
133	Nickel "	1425	482	...	...	1907	1936
134	Carbon Pins	...	592	1271	...	1863	1896
135	Nickel Pins	162	490	504	...	1156	1139
136	Top Chord and End Post, S. Span	2801	...	...	...	2801	2767
137	Built Web Members "	1752	...	...	...	1752	1687
138	Laterals and Sw. Br., "	1546	30	...	...	1576	1511
139	Sidewalk Stringers	42	74	72	28	216	208
140	Handrail	32	58	50	23	163	361
141	Sidewalk Brackets	47	84	83	39	253	257
142	Pin Caps	4	47	53	...	104	92
143	" Bolts	3	22	22	...	47	47
144	Stairs	...	...	...	...	...	161
145	Track Iron	...	...	...	...	...	33
146	Concrete Reinforcement	...	...	...	...	...	9
147	Filler Rings	...	...	...	...	...	25
148	Brake	...	17	...	...	17	18
149	Field Rivets	...	...	...	...	...	1303
	Total Weight	11109	48580	69692	2794	132175	132929
	%Nickel Steel	56	48	6	0	25.5	24.5

Weights are given in 1000 lb. units.

Calculations for the Cantilever Arm dead load stresses —

FORMULAE. Refer to Fig. No. 1

$$Q = R_{11} \frac{g}{h}$$

$$Uu = Mm - R_{11}f$$

$$Ll = Mm + Qq \qquad q = \frac{pf}{q}$$

$$= Mm + R_{11}\frac{p}{h}f$$

$$Tt = Mm - R_{11}f - U_1u \qquad u = u_1z$$

$$= Uu - U_1u_1z$$

$$D_2d = Mm - R_{11}f - L_1l_1z = Uu - L_1l_1z$$

$$D_1 = D_2 + R_{11}\frac{d_1}{h}$$

$$H_L = D_1\frac{h_1}{d} + \text{Conc. } L_1$$

$$H_F = H_L + \text{Conc. } F_1$$

$$P = T\frac{p_1}{t} + \text{Conc. } U_1$$

PANEL 0—2. Refer to Fig. No. 3.

$$U_oU_2 = M_2\frac{1}{u}$$

$$U_oM_1 = M_2\frac{1}{d}$$

$$M_1L_2 = U_oM_1 + R_1 \frac{\text{lenth } M_1L_2}{\text{'' } U_2L_2}$$

Table No. 4. gives the elements of the truss and the coefficients of the formulae required for calculating the stresses.

Table No. 5. gives the total concentrations at the different panel points and the moments. It will be noted that the concentrations differ slightly from those shown on plate No. CIX but the difference was not considered great enough to warrant another revision of the stresses with the exception of the sub members carrying one panel load only.

Table No. 6 gives the calculations for the stresses. Concentrations under R_{11} are as given on plate No. CIX for the reason given in the last paragraph.

Elements of the truss.

$$\cos i = 0.9798 \qquad t = \frac{(p+h)\ (f+g)}{t}$$

$$\cos j = 0.9732 \qquad d = \frac{(p+h)\ (f+g)}{d}$$

$$u = (p+h) \cos i \qquad z = \frac{p+h}{p+h_1}$$

$$l = (p+h) \cos j \qquad y = \frac{h_1}{(p+h)\ (f+g)}$$

TABLE No. 5—ELEMENTS OF THE TRUSS

	PANEL	0–2	2–4	4–6	6–8	8–10	10–12	12–14	14–16
Elements of Truss-feet	p	38.947	48.754	60.808	75.518	92.680	109.843	127.005	145.000
	h	44.333	55.658	69.577	86.563	106.381	126.198	146.016	165.000
	p+h	83.280	104.412	130.385	162.081	199.061	236.041	273.021	310.000
	f+g	65	48	59	72	84	84	84	84
	t	....	68.417	84.727	104.341	125.082	138.280	152.270	166.853
	d	95.525	73.496	91.224	112.594	135.546	151.598	168.454	185.894
	q	56.672	36.482	45.292	55.500	67.773	75.799	84.227	92.203
Coefficients	*u*	81.60	102.30	127.75	158.80	195.04	231.27	267.51	303.74
	l	....	101.61	126.89	157.74	193.73	229.71	265.70	301.69
	t	....	73.25	90.79	111.85	133.67	143.38	150.61	156.06
	d	56.668	68.19	84.33	103.66	123.36	130.79	136.14	140.08
	q/h	.6805	.6555	.6510	.6412	.6371	.6006	.5768	.5588
	p/h	.8785	.8760	.8740	.8724	.8712	.8704	.8698	.8788
	z	....	1.2537	1.2485	1.2430	1.2282	1.1858	1.1565	1.1354
	d_1/h	.7412	.7015	.6889	.7228	.6371	.5972	.5768	.5633
	p_1/t	....	.5693	.5754	.5822	.6038	.6702	.7214	.7613
	h_1/d	.5717	.6032	.6101	.6180	.6386	.7017	.7492	.7850
	y	.010101	.008846	.007235	.005964	.005177	.005366	.005503	.005608

TABLE No. 6a—MOMENTS

Panel Point	Conc. lbs x 10^3	Shear lbs x 10^3	Lever ft.	Mom. ft.- lbs x 10^6	RIIf ft.-lbs x 10^6	Mm-RIIf ft.-lbs x 10^6	Mm ft.- lbs x 10^6
0	3391.5	3391.5	65	220.5	......	......	......
1	356.3	......	42	......	15.0	......	......
2	575.3	4323.1	48	207.5	......	220.5	235.5
3	161.2	......	25.5	......	4.1	......	......
4	661.5	5145.8	59	303.6	......	443.0	447.1
5	202.6	......	31	......	6.3	......	......
6	899.4	6247.8	72	449.8	......	750.7	757.0
7	249.3	......	40	......	10.0	......	......
8	1222.1	7719.2	84	648.4	......	1206.8	1216.8
9	287.2	......	42	......	12.1	......	......
10	1576.5	9582.9	84	805.0	......	1865.2	1877.3
11	347.4	......	42	......	14.6	......	......
12	1893.1	11823.4	84	993.2	......	2682.3	2696.9
13	377.4	......	42	......	15.8	......	......
14	2361.8	14562.6	84	1223.2	......	3690.1	3705.9
15	501.8	......	42	......	21.1	......	......
16	915.6	15980.0	0	0	......	4929.1	4950.2

TABLE No. 6b—COEFFICIENTS AND MOMENTS

Panel	CONCENTRATIONS lbs × 10^3				MOMENTS ft. lbs × 10^6							
	U_1	F_1	L_1	R_{11}	Mm	$R_{11}f\frac{p}{h}$	Ll	Uu	U_1u_1z	L_1l_1z	Tt	D_2d
0–2	123.6	126.4	223.5	353.0	235.5	....	....	235.5	....	....	....	....
2–4	195.5	164.8	187.2	162.0	447.1	3.6	450.7	443.0	295.2	295.2	147.8	147.8
4–6	278.6	178.1	305.3	202.2	757.0	5.6	762.6	750.7	553.0	562.7	197.7	188.0
6–8	388.1	207.0	450.3	248.6	1216.8	8.7	1225.5	1206.8	933.0	948.0	273.8	258.8
8–10	521.9	223.8	595.1	281.5	1877.3	10.5	1887.8	1865.2	1482.0	1505.2	383.2	360.0
10–12	645.8	225.0	746.1	342.5	2696.9	12.7	2709.6	2682.3	2211.8	2238.2	470.5	444.1
12–14	791.8	233.9	900.8	371.3	3705.9	13.8	3719.7	3690.1	3102.0	3133.5	588.1	556.6
14–16	....	....	....	496.2	4950.2	....	....	4929.1	4190.5	4223.0	738.6	706.1

TABLE No. 7—STRESSES

lbs $\times 10^3$

L	U	T	D_2	D_1		H_L		H_F	P		Q
....	2702	....	3890	264	4154	2375 224	2599	2725	125 128	253	210
2905	2886	2018	2168	113	2281	1376 187	1563	1728	1149 196	1345	106
4436	4330	2177	2230	140	2370	1446 305	1751	1930	1253 279	1532	132
6010	5876	2448	2497	180	2677	1654 450	2101	2311	1425 388	1813	160
7770	7600	2867	2918	183	3101	1980 595	2575	2799	1731 522	2253	179
9744	9564	3282	3395	208	3603	2528 746	3274	3499	2200 646	2846	206
11795	11598	3904	4088	218	4306	3226 901	4127	4361	2816 792	3608	214
14000	13795	4733	5010	283	5323	....	....	....	....	....	277

OTHER STRESSES

The specification for, the amount of, and notes on the calculations of the Impact, Wind, Temperature, Traction, Brake, Friction, Torsion and Secondary Stresses are given on the following plates,—

Impact,	Floor — Plates XXV and XXVI,	
	Trusses — Plates XXIX, XXXII and XXXIII,	
Wind,	Trusses and Laterals, Plates CV, CVII and CXI,	
	Sway Bracing, Plates XXXV and CXV,	
Temperature,	Plates	CVIII, CXII, XXXIV and CXIII,
Traction,	"	CV, CVIII and CXII,
Brake,	"	CV, CVIII and CXII,
Friction,	"	CV, CVIII and CXII,
Torsion,	"	CV,
Secondary,	"	CX, XXXIV and CXIII.

Plate CVI shows the lateral displacement of the structure under a wind of 30 lbs. per square foot, normal to the bridge, and the horizontal and vertical displacements of the trusses under the maximum variation in temperature.

Plate XXVII shows the provision made in the floor and in the trusses to permit the free expansion and contraction of the structure, due to change in temperature and to the deformations under direct stresses.

Torsion.

As pointed out when discussing the live load stresses in the anchor arm, with one track only loaded the one truss carries 68.47% of the load while the other carries 31.53% resulting in unequal deflections in the two trusses. Owing to the absence of top chord laterals in the Cantilever and Anchor Arms no direct stresses due to this cause result in the truss members of this portion of the structure except as noted under the Anchor Arm live load stresses. Under such a loading the four corners of the suspended span will not all be at the same elevation and a condition may arise when two diagonally opposite corners will be at the same elevation with the other two corners at a higher or lower elevation. The maximum difference in elevation for the live load specified was found to be 1.1 inches. A solution for the determination of these stresses was developed by Mr. Cyril Batho, M.Sc., B. Eng., then Associate Professor of Applied Mechanics of McGill University, which has been fully described by him in " Engineering " of London, Eng. of Oct. 15th, 1915, pages 392 and 393.

The maximum torsion stresses exist with the live load as shown for the maximum stress in the laterals on Plate CV.. These stresses are only a very small percentage of the total stresses in the truss members and were therefore calculated for only one condition of loading (see plate CV) and the stress thus obtained considered as the torsion stress in the members regardless of the condition of loading giving the maximum live load stress in the individual members.

TOTAL STRESSES—SECTIONS OF MEMBERS

Loads used to determine section of members.

Specification:—

All the co-existing loads and stresses and the deformations shall determine the sections of the different members with the following restrictions:—

The sidewalk live load shall be used for floorbeams and sidewalk stringers and members receiving their maximum strain from a length of moving load covering two panels or less.

Stresses produced by a difference of temperature of 25° fahr. between the outer webs exposed to the sun and the other webs of compression members shall be considered as secondary stresses. These stresses and the stresses produced by a difference of 25 degrees Fahr. between the temperature of a shaded chord and the average temperature of a chord exposed to the sun shall be assumed to co-exist with one-half the wind load of 30 lbs. per sq. ft. normal to the bridge.

Unit Stresses.—

Specification: All parts of the structure shall be proportioned so that the sum of the maximum stresses produced by the loadings specified, including impact, shall not exceed the following amounts in pounds per square inch.

For Carbon Steel:

	lbs. per sq. in.
Tension:	
Eyebars	20000
Riveted Members	18000
Including secondary stresses	24000
Compression:	
Short members with $\frac{l}{r}$ — 50 and under	14000
Long members with $\frac{l}{r}$ over 50	$17500 - 70\frac{l}{r}$
Including Secondary stresses	18000

Shear:

Webs of plate girders, gross section 10,000

Laterals and Sway Bracing: Take both systems in calculation of strains disregarding reversal of strains. For compression . . $16{,}000 - \frac{l}{r}$

Bending Stresses:

All bending stresses in compression member produced by the weight of the member itself and by loads applied on the member shall be considered as primary stresses. All such members shall be proportioned so that the greatest fibre stress due to this bending and axial strain together will not exceed the allowed units for the axial stress in that member.

Alternate Stresses:

Members subject to alternate tension and compression shall be proportioned for either stresses and their sections shall be made equal to the sum of the two sections.

Erection Unit Stresses:

Increase the unit stresses given above by 20% except in the case of shear in the webs of plate girders where a unit shear of 13,000 lbs. per sq. inch will be allowed.

For Nickel steel:

Increase the unit stresses given for carbon steel by 40%.

Material Stress Sheets,

Floor: The total stresses and the material used are shown on the following plates:

Plate XXV Track Girders, Sub floorbeams and stringers.

Plate XXVI Main Floorbeams.

The material required to carry the loads specified for the permanent structure was not sufficient in the floorbeams of the cantilever arm to support the traveller during the erection of this portion of the structure, without exceeding the unit stresses specified for erection and it was necessary to increase the thickness of the web plates of the single web floorbeams and the lengths of the cover plates of all the floorbeams to provide for these stresses.

Trusses: A summary of all the stresses, the total stress used for determining the sections of the members, the bending stress in the compression members due to their own weight, the allowable unit stresses and the material used are shown on the following plates.

Plate XXIX Suspended Span, Truss, Laterials and Sway Bracing, also shows typical sections of the different members.

Plate XXXII Cantilever Arm, Truss and Laterals.

The sections of all sub-members supporting the front legs of the traveller during erection were determined by the erection stresses. These members and the floorbeams mentioned above were the only parts of the permanent structure requiring additional material to carry the erection loads.

Plate XXXIII	Anchor Arm, Truss and Laterals,
Plate XXXV	Cantilever Arm, Sway Bracing,
Plate CXV	Anchor Arm, Sway bracing.

Plate VII shows the cross sections and dimensions of the members of the Cantilever Arm. The sections of the members of the Anchor Arm are similar.

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Oct. 10, 1914.—Quebec Bridge Anchor Arm Diagonal and Posts. Describes the longest web members of 515 foot trusses which are erected in several units.

Nov. 14, 1914.—Ultimate Strength of Carbon and Nickel-Steel Models of Quebec Bridge Members. Tension tests of reinforced steel plates and of alternating stress members and compression tests of four struts.

Jan. 16, 1915.—Erection of New Quebec Bridge. H. P. Borden. The north anchor arm has been completely placed by 1000 ton traveller since Aug. 1st, 1914 with exception of two upper panels.

Apr. 17, 1915.—Provision for Traction Stresses in Quebec Bridge. C. A. Norton. Types of trusses used. Illustrated.

July 24, 1915.—New Methods Involved in Building World's Largest Bridge. Details of Quebec Erection.

Sept. 16, 1916.—Suspended Span of New Quebec Bridge Falls into River While Being Hoisted Last Monday Morning. Also editorial. Account of failure and suspected cause.

Sept. 23, 1916.—I — Breakage of Casting of Rocker-Joint Bearing Responsible for Quebec Bridge Disaster. II — Revolutionary Methods Used to Float and Hoist Center Span of Quebec Bridge.

Oct. 7, 1917.—What Was the Cause of the Initial Failure at the Quebec Bridge? Computations and specifications for steel rocker casting, also editorial.

ENGINEERS' CLUB OF PHILADELPHIA

October, 1916.—Pertinent Remarks on the Quebec Bridge Accident. William P. Parker. A study of the cause of the failure.

IRON AGE

Sept. 21, 1916.—Quebec Bridge Disaster Charged to Casting. One of the four cast steel bearings believed to have collapsed.

JOURNAL OF FRANKLIN INSTITUTE

Sept, 1913.—Design of Large Bridges with Special Reference to the Quebec Bridge. Ralph Modjeski. Discusses the more important questions which should be considered by designers of such structures; referring to structures designed and constructed by the Author, particularly the Quebec Bridge.

SCIENTIFIC AMERICAN

Feb. 12, 1910.—The Design of the New Quebec Bridge. Line drawings and criticism of the proposed plans.

Supplement. May 10, 1913.—The Reconstruction of the Quebec Bridge. G. Kuwaschein. Illustrates and describes a design proposed by a Russian engineer, giving calculations, showing a large saving and advantages.

RAILWAY AGE GAZETTE

Sept. 26, 1913.—Adopted Design of the Quebec Bridge. Ralph Modjeski. Discussion of elements considered in designing the longest span in the world.

Sept. 11, 1914.—The Erection Equipment of the Quebec Bridge H. P. Borden. The method adopted and the traveller built for this 1800 ft. span.

Dec. 24, 1915.—Progress on the Erection of the New Quebec Bridge. H. P. Borden.

May 26, 1916.—The Season's Work on the Quebec Bridge. A. J. Meyers. Programme for completion of structure during 1916.

Sept. 22, 1916.—The Cause of the Quebec Bridge Disaster. Failure of bearing.

Sept. 28, 1917.—Quebec Bridge Central Span Successfully Hoisted. A. J. Meyers. Roller or key bearings used as supports during raising instead of rocker bearings and steel castings.

RAILWAY ENGINEER — LONDON

Dec., 1916.—The Quebec Bridge. Critical discussion of the cause of failure of lifting arrangements.

RAILWAY AND LOCOMOTIVE ENGINEERING

Nov., 1916.—Jacking the Quebec Bridge. The lifting jacks; their number and position.

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MONTREAL

1918

H H Vaughan

CONTENTS

INSTRUCTIONS FOR PREPARING PAPERS, ETC.

In writing papers, or discussions on papers, the use of the first person should be avoided.

They should be legibly written on foolscap paper, on one side only, with a margin on the left side.

Illustrations, when necessary, should be drawn on the dull side of tracing linen to as small a scale as is consistent with distinctness. Black ink only should be used. They should be drawn so that all details and lettering will show distinctly when reduced to a height of 7 inches.

When necessary to illustrate a paper for reading, diagrams or lantern projections may be furnished. Diagrams must be bold, distinct, and clearly visible in detail for a distance of thirty feet.

Papers which have been read before other Societies, or have been published, cannot be read at meetings of this Institute.

All communications must be forwarded to the Secretary of *The Institute*, from whom any further information may be obtained.

TRANSACTIONS — VOL. XXXII — PART II.

PREFATORY NOTE.

This volume (Vol. XXXII, Part II) bridges the activities of the Canadian Society of Civil Engineers and The Engineering Institute of Canada. The papers presented are those for which arrangements had been made under the Canadian Society of Civil Engineers. At the Annual Meeting held January 21st-22-23rd, 1918, the Report of the Committee on Society Affairs was adopted which changed the name to The Engineering Institute of Canada and brought into operation a decidedly different method of procedure in respect of the presentation of papers.

Under the new ruling, all papers presented at any branch have equal status dependent upon their intrinsic value. Although the new name was adopted at the Annual Meeting it was not until April 15th, 1918, that the Bill changing the name was passed by the House of Commons. Between the time of the Annual Meeting and the legalizing of the new name, the Montreal Branch was started as part of the newly adopted regulations. Under the Montreal Branch the programme of papers previously arranged was carried out and these are included in this volume.

The authors' names are given with their title in The Institute although at that time the title was not official. In the text the former appellation has not been changed.

RECENT ADVANCES IN CANADIAN METALLURGY

By ALFRED STANSFIELD, D. Sc., F.R.S.C., M.E.I.C.

February 14, 1918

In writing of "recent" advances it becomes necessary to decide on some definite period of time on which to report, and this surprising war has given such an impetus to Canadian metallurgy that it will be appropriate to regard the commencement of the war as the epoch from which to measure these advances.

Metal Markets in War Time.—The first effect of the war was to restrict the regular operation of metallurgical plants and to stop all new developments. This followed naturally from uncertainty in the financial situation and an immediate lack of money. Looking back at the situation it seems strange that markets should fall, and metal production decrease, when it must have been certain that nearly all the metals would be needed in increasing quantities for warlike operations. After about six months this demand became apparent, the prices of metals began to rise, and their production was consequently stimulated. An early development was caused by theShell Committee's need of brass for cartridge cases. Copper for this purpose could be obtained without great difficulty, but zinc of a suitable degree of purity was very difficult to obtain even at a greatly enhanced price. Zinc ores had been mined in British Columbia for a number of years, but it had not been found practicable to smelt them in this country, and the zinc ores, or concentrates, were shipped to smelters in the American zinc districts. After the outbreak of the war, the American smelters refused to accept ore shipments from Canada; a refusal which was considered by many as being caused by German control or interest in the smelters. Whatever was the cause the situation was clear: the Shell Committee needed zinc, and the British Columbia miners had the ore, while the American smelters were unwilling or unable to convert the one into the other. Zinc differs from many metals in the fact that the smelting process yields a metal which is marketable without the need of any refining. The purity of the "spelter" depends mostly upon the character of the ore from which it is produced, thus "high-grade" spelter from specially pure ores will contain less than 0.1% of total impurity, while "Prime Western" spelter, obtained from leady ores, may contain as much as 1.5% of lead. This difference in the purity of different brands of spelter is not objectionable, since the uses to which they are put vary greatly in requirements; "Prime-Western" spelter is amply good enough for galvanizing, while "high-grade" spelter is none too good for making brass for cartridge cases. The British Columbia zinc ores are in general mixed with ores of lead, and it would be impossible by the usual smelting process to obtain a spelter that would meet the requirements of the Shell Committee.

A process was under investigation, at that time, for obtaining zinc from these complex ores by roasting, extracting the zinc as sulphate by means of sulphuric acid and water, and separating metallic zinc from this solution by electrolysis, using insoluble anodes. This process was not new, but it had failed hitherto owing to technical difficulties in the leaching of the ore, the purification of the solution and the production by electrolysis of compact zinc that could be remelted. The cost, moreover, was too high to make it profitable in view of the low price of zinc before the war. A Commission was appointed by the Minister of Militia to investigate the situation with respect to the supplies of copper and zinc. This Commission concluded that the electrolytic process offered the greatest probability of filling the need, and made arrangements with the Consolidated Mining and Smelting Company which led to the establishment at Trail of a plant for producing electrolytic zinc from the British Columbia ores. At the present time this plant has a capacity of about 300 tons of zinc per week.

Before the war the writer carried out an elaborate investigation for the Department of Mines on the electric smelting of zinc ores, but practical success was not reached. During the war this research has been taken up again for a mining company with more success, both in the laboratory and in a small plant at Shawinigan Falls. Commercial success has not yet been obtained, but it is expected that the work will be resumed during the present year. The writer has also been interested in the production of zinc oxide for paint by the Wetherill process; he helped to design and operate an experimental plant in Montreal, and following on from this a full-sized plant has been erected at Notre Dame des Anges.

The metallurgy of zinc, as well as that of lead, copper and silver, has been materially improved during recent years by the application of the flotation process by means of which the zinc minerals can be separated more perfectly from the gangue, and in some cases from other metallic minerals.

Copper Situation.—The copper situation, investigated by the Commission, was entirely different: copper ores are mined and smelted in British Columbia on an enormous scale for the production of copper matte, and in some cases the matte is bessemerized for the production of metallic copper. Crude copper, however, is of no use until it has been refined, and all the copper derived from British Columbia was sent to the United States, as ore, matte, or blister copper, in order to be refined in the American refineries. The problem before the Commission was the establishment in Canada of an electrolytic refinery, so that Canadian copper could be made available for Canadian needs of that metal. The problem was complicated by American interests in the Canadian copper smelters, by contracts with American smelters and refiners, by heavy freight charges between British Columbia and the points at which the copper would be used, and by the lack in Canada of rolling mills and other works for turning the refined copper into sheets, rods, wires and other forms required by the Canadian market. Although the price of copper rose to a very high figure, the Shell Committee was able to obtain supplies of pure copper from the States, and there was not sufficient urgency to justify the Government in establishing a copper refinery in British Columbia in face of the natural obstacles already mentioned. In the meantime the Consolidated Mining and Smelting Company established at Trail a small refinery, which is now in operation, and has an output of about 10 tons per day, and the copper and zinc from Trail have both been used in the Dominion Copper Products plant in Montreal for making brass for the manufacture of cartridge cases. It must not be supposed that the erection of this small refinery solves the general problem outlined above, although it may serve by way of example to help in settling it at some later date. The copper furnaces at Trail smelt ores of gold and silver as well as ores of copper, and the resulting crude copper is so rich in the precious metals that the Company finds it more satisfactory to refine the metal at Trail, thus securing the gold and silver contents, instead of shipping the crude metal to distant refineries which would entail very serious difficulties in ascertaining and obtaining credit for the gold and silver that is alloyed with the copper. For economical operation, a copper refinery should have a very large output, say 100 tons a day. The whole output of copper from British Columbia amounted in 1913 to about 23,000 tons, or about 60 tons per day, so that a practically unanimous co-operation of all the producing companies would be needed to support a single refinery. We may hope that in course of time a large refinery will be operated in British Columbia for the treatment of Western ores, but this will depend on the erection in that Province of mills for working up the refined copper and of the growth of a market in Western Canada that will absorb a large part of the output of such a refinery. The production of copper in British Columbia has increased steadily during recent years as the following figures show, this increase being largely due to the development of the Granby Company's smelter at Anyox. Recently the copper smelter at Ladysmith has changed hands and has again been put into operation. This increase of copper production will tend towards the establishment of a large refinery on the coast. The introduction of the flotation process has modified copper metallurgy in recent years. Low grade ores, which were at one time smelted in blast furnaces, or even treated by wet (chemical) methods, are now crushed and ground to a fine powder and concentrated by flotation. The flotation concentrate, on account of its powdery condition, can be treated most easily by roasting in mechanical furnaces and smelting in reverberatory furnaces.

PRODUCTION OF COPPER IN BRITISH COLUMBIA IN TONS

1913	1914	1915	1916
23,000	21,000	28,000	32,000

Outside of British Columbia the largest production of Canadian copper is from the Sudbury district, where it occurs in association with nickel. The production in Ontario, nearly all being from the Sudbury district, is about half the production in British Columbia, but its separation and refining is dependent on the separation of the nickel and copper, and can only be considered as a by-product from the extraction of nickel.

Nickel Production and Refining. — At the beginning of the war two companies, — the International Nickel Company and the Mond Nickel Company, – were operating in the Sudbury district. These companies smelted the ore to a matte containing from 20% to 30% of nickel and copper, and then bessemerized this to obtain a matte of about 80% of nickel and copper, containing little else than sulphide of nickel and sulphide of copper in proportions depending on the relation between these metals in the original ore. The Mond Company shipped their matte to England for treatment by the Mond process at their works at Clydach, in South Wales, while the International Company treated their matte by the "salt cake" process at their refinery at Constable Hook in New Jersey. It has for a long time been regarded as an economic injustice to Canada that the copper-nickel matte should be sent to the States for refining, but it was maintained by the Company officials that the cost of refining it in Canada would be very much higher than in New Jersey. After the outbreak of war, attention was repeatedly called to the situation, because it was feared that the nickel passed into German hands, and as a result of this outcry the International Company agreed to erect in Canada a refinery sufficiently large for the British requirements of nickel. This refinery is now being built at Port Colborne in Ontario and should be in operation early in the present year; it will have an initial output of 7,500 tons of nickel per annum. In the meantime a new company has entered the field. The British America Nickel Corporation, a strong British-Canadian company which is controlled by the Imperial Government, has acquired about 17,000 acres of mineral land in the Sudbury district, and early in 1917 began the construction of a large smelting and refining works at the Murray mine. This company will smelt the ore to matte, and bessemerize the matte, on substantially the same lines as the older companies; but the production of refined nickel and copper from the matte will be carried out by the Hybinette electrolytic process which is now in operation in Norway. The new plant is expected to be in operation in 1919, and will have an output of 6000 tons of refined nickel per annum. In 1913, the last whole year before the war, nickel matte containing 25,000 tons of nickel was sent abroad to be refined; during 1917 the output was about 42,000 tons, and by 1919 we may expect a production of some 13,000 tons of nickel in Canada in addition to perhaps three times this amount in matte refined abroad.

During the year 1914 one million tons of nickel-copper ore was mined and smelted in the Sudbury district, and the resulting matte contained 22,700 tons of nickel and 14,400 tons of copper. Some 400,000 tons of iron, equal to half the Canadian production of pig iron, was slagged in the furnaces and thrown over the dumps, and 300,000 tons of sulphur was discharged into the atmosphere. This sulphur would produce one million tons of sulphuric acid, which is equal to one-fourth of the consumption of the United States, but it is not considered worth while to save this at the present time on account of the cost of transportation of the acid to points at which it could be used. Projects have been considered, however, for saving the sulphur in the elemental state so that it could be shipped easily, or for collecting the SO_2 gas in the liquid state and using it for the manufacture of wood pulp. With regard to the waste of iron, it may be remembered that more than half the nickel produced is used in the manufacture of nickel steel and with respect to this part of the output, there is therefore no need to separate from the nickel the iron which was originally associated with it. Attempts to make a nickel pig-iron which could be converted directly into nickel steel were met in the past by the difficulty that the copper, which is almost always present in the ore, was supposed to be harmful in the steel. During recent years, however, it has been found that if the copper is present in moderate amounts, not more than one third of the nickel content, a nickel-copper steel is entirely satisfactory and may be expected to replace the usual nickel steel for many purposes. Many tons of this steel have recently been made from Sudbury ore, by the process devised by Mr. Colvocoresses; and its mechanical properties and suitability for many purposes will soon be ascertained. It should be possible to employ this process on a large scale, although of course it is limited to ores with a small content of copper. Care will be taken, by suitable admixture of the ores and otherwise, to maintain a uniform product;and it may be added that the small amount of precious metals contained in the ore cannot be saved if this process is used.

In connection with the advances in the nickel industry mention must be made of the investigation of the Ontario Nickel Commission, which was appointed in 1915 and brought out its report in 1917. This Commission has investigated the ores, smelting processes, refining methods, properties and uses of nickel, not only in Canada, but in all parts of the world where nickel ores are mined. They have considered the subject from many points of view, and have produced a very valuable report. Geo. T. Holloway, the metallurgist of the Commission, died soon after the completion of his work.

The production of iron and steel in Canada fell off considerably at the beginning of the war, but recovered later, and at the present time, owing to the large demands for the manufacture of munitions, is larger than ever. This will be seen from the following table:—

PRODUCTION OF IRON AND STEEL IN SHORT TONS*

	1912	1913	1914	1915	1916	1917
Canadian Iron Ore	216,000	307,000	245,000	398,000	340,000	190,000
Pig Iron	1,014,000	1,129,000	783,000	914,000	1,169,000	1,187,000
Steel ingots and castings	957,000	1,169,000	828,000	1,020,000	1,454,000	1,735,000
Electric furnace steel			61	5,600	43,000	50,000

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*The figures for production in this paper have been taken from the reports by John McLeish, who kindly furnished the writer with approximate data for 1917. The amounts, in most cases, have been stated in even thousands, as in this form they are far more easily read and compared.

Electric Iron Smelting.—The production of steel has advanced faster than the production of pig-iron, the latter being restricted by the need of importing the ore, and in some cases even the fuel, for smelting in Canadian furnaces. The amount of ore mined in Canada is but a small proportion of the amount smelted, but if we include Newfoundland ore, as we reasonably may, we would find that rather more than half the iron is derived from Canadian ore. Just before the war the writer visited Sweden in order to study the electric iron smelting industry in that country, and made a report to the Mines Branch in the fall of 1914. At that time it did not appear that electric iron smelting could be undertaken profitably in this country, but in the year 1916 he made a fresh study of the situation in connection with a report by Mr. John Dresser on "Part of the District of Lake St. John, Quebec", and came to the conclusion that there is hope for electric pig-iron in Canada. In considering the possibility of smelting electrically the titaniferous magnetites of that district it appeared that although pig-iron produced in that way could never compete, as regards price, with the product of the blast-furnace using coke as fuel, still it should be possible to make a limited production of high class white pig-iron which would command a high price and thus sell at a profit. Besides producing a very desirable quality of pig-iron, and incidentally helping the steel makers, this would have the advantage of utilizing ores which are at present worthless, and of employing water-power which now runs to waste.

Electric Furnace Steel.—Although electric smelting of iron ores is still in the future, the electric furnace has been employed very largely for making steel, as is shown in the table. The electric furnace has not been used in this country to any extent for making new steel, but mostly for remelting steel scrap into shell billets. For this purpose several furnaces are in operation in Montreal and district and at Toronto a large plant containing ten 6-ton furnaces has been built by the Munitions Board for remelting shell turnings and other scrap. During the period under consideration the Armstrong Whitworth Company have built up an electric melting plant for the production of all kinds of tool steel in their works at Longueuil. The electric steel plant at Belleville has made a quantity of very good steel from titaniferous iron ore, but at that time it was found cheaper to employ steel scrap than to smelt ore. The plant has recently been fully occupied with the production of ferro molybdenum for the British Government.

Ferro Alloys.—There has been, for a number of years, a moderate production of ferro-alloys in Canada amounting in 1913 to 8000 tons, in 1914 to 7,500 tons and in 1915 to 10,800 tons. This production consisted mostly of ferro-silicon made in electric furnaces at Welland by the Electro-Metals Limited, and in 1915 a little ferro-phosphorus made at Buckingham, P.Q. by the Electric Reduction Company. In 1916 the production had risen to 28,600 tons, valued at $1,777,000 and included besides ferro-silicon and ferro-phosphorus, a considerable production of the valuable product ferro-molybdenum, which is now made in electric furnaces at Belleville and Orillia. Ferro-silicon is now made at Shawinigan as well as at Welland. For the manufacture of steel there is a considerable demand for ferro-manganese and spiegel-eisen, and experiments are now in progress for making these alloys from Canadian manganese ores. The production of ferro-chrome is also under consideration.

Gold and Silver.—More than 80% of Canadian silver comes from the Cobalt region, and during the period of the war the production has been falling slowly as the mines became exhausted, and on account of the scarcity of labour and supplies. The recent rise in price of silver has however stimulated the production. The metallurgical treatment of the ores had been worked out prior to the war, but the recent development of the flotation process has improved the concentration processes at Cobalt. The production of silver in 1916 was 25,600,000 ounces, valued at $16,800,000 and in 1917 was about 23,500,000 ounces. The production of gold in Canada was varied from about 800,000 ounces to 1,000,000 ounces per annum during this period; gold is obtained from Yukon, British Columbia and Ontario. Before the war the two Western provinces had the largest outputs, but since the growth of the Porcupine district the production of Ontario is nearly equal to the sum of the other two, and the total productionhas somewhat increased.

Lead.—The need of lead for bullets caused an increase in the mining and smelting of lead ores, with the result that in 1917, when this demand was reduced, there arose a difficulty in disposing of the lead already on hand and undergoing treatment. The management of the Trail Smelter informed the miners that they would only take ores that were relatively free from zinc, so as to reduce the ore supply and at the same time obtain a better grade of ore. This ruling gave rise to a great outcry, and arrangements were made with the Munitions Board to take a larger amount of the metal. The success of the Victory Loan should enable the production to be taken up by the Board, but it may be pointed out that the normal Canadian market for lead and lead products is amply sufficient for the Canadian production, and it is only necessary to arrange for the lead being converted into marketable forms. Some advance has been made in this direction and there are two "corroding" plants in Montreal for the production of white lead, but red lead and litharge should also be manufactured. The usual production of lead is about 20,000 tons per annum, but in 1917 it amounted to 28,000 tons. The Cottrell electrical precipitation process for the collection of smoke and fume has proved of great service in the treatment of ores of lead and zinc, and has been introduced at the Trail Smelter.

Antimony.—The metal antimony was selling at about 8 cents per pound, but early in the war its price rose to 16 cents, and later to more than 40 cents a pound. Antimony is used to a large extent alloyed with lead for making shrapnel bullets, and the supply was for a time insufficient for the need, so that bullets have been made of lead encased in a steel shell, the whole being arranged to have the same size and weight as the bullets of alloy previously in use. Antimony is not produced to any extent in Canada, although a small amount is derived from the electrolytic refining of lead at Trail. In view of the high price and unusual demand for the metal a mine of low grade ore at Lake George, New Brunswick was reopened and after much patient work an efficient process was devised for extracting the metal from the ore by volatilization. An account of this process was recently presented to the Society by Mr. J. A. DeCew, M. Can. Soc. C. E. By the time that the process had been brought to a satisfactory condition, other supplies of antimony became available, the price fell, and the mine and smelter had to be closed.

The metal aluminium is made at Shawinigan Falls from bauxite which is imported from France and elsewhere as no ores of commercial value have been found in Canada. Figures for the production of this metal are not published as only one firm is engaged in the industry, but there is no doubt that the production has increased materially during the war, aluminium being needed in large amounts for the construction of aeroplanes and other military equipment. The value of metal exported from Canada in 1913 was $1,700,000; in 1914, $2,300,000; in 1915, $3,300,000, and in 1916, $5,200,000.

Aluminium and Magnesium.—The process of extracting aluminium from alumina (purified bauxite) consists, as is well known, in electrolysing fused salts to which alumina is added. Before the war this was the only operation in use in Canada, that depended on the electrolysis of fused salts, but a fresh industry of this kind has now started—the production of the metal magnesium. Before the war this metal was made in Germany from the natural deposits of carnallite, but the war stopped this source of supply. The writer succeeded in producing magnesium in the laboratory, and the process was developed into commercial operation at Shawinigan Falls. At present the crude materials are imported, but ultimately it is intended to use Canadian magnesite as the ore of this metal. Magnesium is now made in considerable quantities at this plant; it is used in the form of powder for star shells and alloyed with aluminium for the construction of aeroplanes. It is also used as a deoxidizing addition in melting certain metals and alloys.

Cobalt.—The metal cobalt, which gave its name to the largest silver producing district in Canada, has not as yet found very many uses. It can be employed like nickel for plating, but being more costly its use is limited. It has also been employed in steel making. Valuable researches in regard to its use have been made by Professor Kalmus at Queen's University. A recent development is its use in the production of "Stellite", an alloy of cobalt, chromium and tungsten which is used for cutting tools in place of high speed steel.

Brass Melting and Rolling.—In connection with the production of zinc and the refining of copper in Canada the need arises for converting these and other metals into marketable forms. The Dominion Copper

Products plant at Lachine has been built during the war for melting and rolling brass and copper for use in shell and cartridge cases. As the need for such products decreases, it is expected that this plant and Brown's Copper and Brass Rolling Mills in Toronto will be employed on a large scale for making brass and other non-ferrous alloys and for rolling these and the metals copper and zinc into forms suitable for use by Canadian manufacturing industries, so that a market will be found in Canada for Canadian copper, zinc, nickel, lead and other metals.

TESTS OF THE CHAIN FENDERS IN THE LOCKS OF THE PANAMA CANAL

By HENRY GOLDMARK, M.E.I.C.

(Read before meeting of the Montreal Branch, February 28, 1918.)

GENERAL

When the alternative plans for the Panama Canal were under discussion, advocates of the sea-level type laid great stress on the dangers to navigation inherent in a lock canal. Such dangers undoubtedly exist, although experience has shown that the risk of serious accident is very small in locks that are properly designed and carefully operated. Even at the Soo where the traffic, for many years, has been extremely heavy, only one serious accident is on record since the first lock was opened in 1855.

In comparing the two types in the case of Panama, it should be borne in mind, that the broad and deep channels provided by Lake Gatun, possess elements of safety which would have been absent in the smaller cross-sections of a sea-level canal.

On the other hand, the accidental destruction of certain of the lockgates, would not only involve the risk of injury to vessels but might also set free the water impounded in Lake Gatun, and lower its level so much as to stop navigation for a long period of time.

In working out the detailed plans of the locks, it was thought wise to take all possible precautions against injury to the gates and to provide special safeguards against further damage in case, after all, one or more gates were accidentally destroyed.

The safeguards adopted with these ends in view are the following:—

(1) Electric Locomotives for towing all vessels through the locks. These travel on a rack railroad close to the edge of the lock walls and have, so far, proved entirely satisfactory in controlling vessels and keeping them centered in the lock chambers.

(2) Chain Fenders for protecting the most important gates.

(3) Duplicate Gates in certain parts of the locks. There are the usual "guard gates" at both ends of each lock flight and besides these a second pair of lower operating gates is provided in Pedro Miguel lock and the upper chamber at Gatun and Miraflores.

(4) Emergency Dams of the drawbridge type at the upper end of each lock for shutting off the flow of water in case of serious injury to the gates.

The first of these devices forms a part of the machinery used in normal locking. As long as it functions properly no further safety mechanism comes into operation.

The second device, the chain fender, protects the gates, when, for any reason, a vessel is not under the control of the towing system.

The third safeguard, the duplicate gate, in its turn, does not come into play, until the fender protecting it has failed to fulfill its proper function and finally:

The emergency dam is needed only after all the preceding safety appliances have failed so that it becomes necessary to check the current of water flowing through the locks.

A full description of the various safety appliances is given in a series of papers on the Panama Canal written by the members of the engineering staff responsible for the different parts of the work and presented to the International Engineering Congress at San Francisco in 1915. *

They are also described in a concise, but readable and comprehensive article, published in "Engineering and Contracting" Jan. 7, 1914, which is the best general account of the Panama Canal known to the writer. Reference should also be made to an excellent paper **, which records the experiences obtained in the actual operation of the locks, since the opening of the canal.

The chain fenders, the second of the safeguards mentioned above, were adopted at the suggestion of the writer who was in immediate charge of their design and construction. While similar fenders have been used in English locks for a number of years, they are believed to be inferior in strength and reliability to the Panama design.

The present paper is intended to supplement previous articles on the Chain Fenders and more especially to put on record, in a systematic form, some unique tests which were made during the construction of the fenders, as well as since the canal has been opened for use.

In view of the many novel features and the fact, that, in actual service the fenders proved entirely satisfactory in bringing vessels to rest, it was thought that an account of these tests might be of interest to the Society.

* Transactions of the International Engineering Congress 1915. The Panama Canal Vol II. Also published separately by the McGraw Hill Publishing Co., N.Y. 1916.

** First Year's Operation of the Locks of the Panama Canal; F. C. Clark and R. H. Whitehead. Journal of the Western Society of Engineers Vol. XXI No. 4. April 1916.

The fenders were placed in the upper and lower approaches to the lock flights, thus protecting the upper and lower guard gates, and also just above the intermediate and lower gates in the Pedro Miguel lock and the upper chamber at Gatun and Miraflores.

DESCRIPTION OF FENDER MACHINERY.

It is deemed unnecessary to give a detailed description of the fender machinery, although a brief account is requisite for a proper comprehension of the tests. The fenders consist of heavy chains, which normally span the lock chambers, near the top, being lowered to the lock floor, when a vessel is about to pass. Each gate and its protecting fender are interlocked electrically, so that the chain cannot be lowered, until the gate is opened, and hence is no longer in danger from collision.

The chain is arranged to pay out under stress, when it is struck by a vessel, so that the energy of the vessel is absorbed and it is brought to rest without damage. The machinery must, therefore, not only make provision for lowering and raising the chain, in daily operation, but must also include some reliable means of putting the chain under stress when it is stopping a vessel. Evidently the success of the entire fender depends upon the mechanism for producing a suitable resistance to the travel of the chain in its emergency action.

In the English fenders, mentioned above, the friction of the chain about a horizontal cast iron cylinder placed on one of the lock walls, is depended upon, to give the necessary resistance. A small hoisting engine on the other wall raises and lowers the chain.

The writer examined one of these fenders at Avonmouth near Bristol in 1908, and discussed their details with the designers and builders, Messrs. Brown, Lenox & Co. of Pontyprid, Wales. They are simple in construction, but the frictional resistance is likely to be variable in amount. It is also believed that lowering the chain from one end only is undesirable, as it often forms a loop at the bottom, which may foul vessels in the lock. As far as could be learned, no tests in actually stopping vessels have ever been made with these fenders.

It is proper to add that the Panama designs were well in hand before the writer had heard of the English fenders, although their inspection proved of much interest. He would also like to record here, his indebtedness to his friend, Mr. E. H. McHenry, M. Can. Soc. C. E., for most valuable suggestions in connection with the first inception of the Panama chains.

The adopted design was the result of an extended investigation. Frictional resistances of different kinds were studied, also the use of heavy weights, for stopping the vessels, but the hydraulic apparatus finally selected was considered to have advantages over all other forms.

With some minor variations, the design shown on Plate (1) was used in all the twenty-four fenders built at Panama. There are three cylinders of the plunger type, the upper of which is suspended from beams spanning the machine pit while the bottom plunger rests directly on the concrete. The intermediate cylinder is movable, and slides on the inner surface of the upper and the outer surface of the lower cylinder. The chain passes through a hawse-pipe casting of steel, secured to a heavy anchorage, and is connected to the moving cylinder by a system of grooved sheaves. The pull of the chain when stopping a vessel is transferred to the anchors embedded in the concrete.

The lowering and raising of the chain is brought about by pumping water under pressure into the bottom and top cylinders respectively. The maximum stroke is 21′ 3″ and the multiplication given by the sheaves is four fold, so that the chain pays out 85 ft. from each wall, a length which is sufficient for the deepest lock and also provides ample stopping power in emergency operation.

The chains were made from wrought iron bars 3″ in diameter and have links 10″ wide and 17″ long. The sections spanning the lock chamber have standard Navy stud links, while open links are used for the part that passes around the sheaves. Considerable difficulty was met with in obtaining chains of proper strength, especially the open links which have rarely been made of so large a size. The specified breaking strength was 500,000 lbs. for the studded and 450,000 lbs. for the open links, but all shots of chain were subjected to proof tests of 300,000 lbs. and 250,000 lbs. respectively.

The operation of lowering and raising of the chain will be readily understood from the plans, especially the small diagram on Plate 1 which shows the arrangement of the piping and valves.

In order to start the cylinder, on either the upward or downward stroke, it is necessary to start the centrifugal pump and also to reverse the position of the operating valve which controls the direction of the flow. The latter is of the double piston type and operated by a small electric motor. Both the pump and valve motors are normally started from the central control house, from which all the gate and valve machines in the lock flight are controlled, but local control is also provided for.

The cylinder is brought to rest at each end of the stroke by a limit switch which stops the pump automatically, and it also starts the same whenever leakage has caused the cylinder to move up or down a predetermined distance from its end position.

The maximum pressure in the cylinders is from 100 to 150 lbs. per sq. in., the higher pressure being required in lowering the chain, as the heavy intermediate cylinder has to be lifted in this case.

Typical indicator diagrams taken on the first fender erected (at Gatun locks) are given in Figs. 1 and 2. The high pressure prevails in the upper cylinder when raising and the lower cylinder when lowering the chain.

The pump has two stages, the first being of the volute, the second of the turbine type, a somewhat novel arrangement, which has proved entirely satisfactory. The pump has a 6″ suction and 5″ discharge pipe and is operated at 460 r.p.m. by a 70 H.P. 250 volt 25 cycle induction motor. The lowering or raising of the chain is done in about one minute in a perfectly satisfactory manner, the chain dropping into a pit in the floor so as to offer no obstruction to the passage of vessels.

EMERGENCY OPERATION.

As the sole function of the fender is the checking of vessels, the device for maintaining a heavy tension on the chain, as it pays out, after being struck, is the most vital part of the entire apparatus.

It consists of a pair of resistance or relief valves placed as shown in the Diagram of Operating Machinery on Plate 1. When the chain is struck by a vessel, there is a tendency for the moving cylinder to rise, so that the water pressure in the piping increases rapidly. The resistance valves must permit the water to escape, as soon as the pressure reaches a point corresponding to a suitable working tension in the chain links and then keep the pressure as nearly constant as possible. As a rule it will be necessary, in order to accomplish this result, for the opening in the valves to vary slightly as the chain pays out. Their movement must, of course, be reliable and they must close promptly when the strain on the chain is entirely relieved.

It should be noted that the travel of the chain is resisted not only by the hydraulic resistance to the motion of the cylinder but also by the weight of the cylinder and other moving parts, by the friction of the chain at the hawse-pipe casting, as well as by frictional resistance in the machinery itself. As will be seen later, it proved entirely feasible to measure these supplemental forces accurately. They proved about equal in amount to the internal hydraulic resistance, making it necessary to set the valves which control the pressure in the cylinder, for a much lower pressure than originally contemplated.

It may be seen by reference to the piping diagram, that, with the chain in its normal operating condition across the top of the lock, all gate and check valves are closed, so that the resistance valves provide the only means by which the pressure can be relieved.

In view of the importance of the subject, various types of valves that seemed suitable for the purpose were carefully studied, and three different designs were finally selected for detailed tests. The first of these was a differential piston valve of special design which originated in the writer's office and is referred to in the sequel as the I. C. C. Valve. It did not prove entirely satisfactory in the tests. The **second** valve was made by the H. Mueller Manufacturing Co. of Decatur, Ill., and differed from their standard design mainly in the use of bronze for the valve body in order to withstand the high pressures. This valve is quite simple. There is a disk $4\frac{1}{2}''$ in diameter with a conical seat. The stem is directly controlled by a heavy helical spring, which keeps it from rising until the pressure reaches the predetermined amount for which the valve has been set. The **third** valve was built by the Ross Valve Manufacturing Co. of Troy, N.Y. from their own designs. It is practically identical with the valve used with much success in maintaining a uniform pressure in the high pressure fire mains of New York City. The main valve has a movable stem with two pistons $6''$ in diameter, arranged so as to be very nearly balanced. There is an auxiliary valve of safety spring diaphragm type, which opens at a definite pressure, for which it may be set, and also a small needle valve permitting the escape of water. The pressure at which the main valve opens depends upon the setting of the auxiliary valve and the needle valve. The pressure actually maintained at the main valve is usually somewhat higher than that for which the auxiliary valve is set.

The tests were very carefully made, with delicate apparatus, so that they may be called laboratory experiments on a large scale, and proved of sufficient interest to warrant their publication in some detail. There were three series of tests:—

(1) Preliminary tests on the three valves at a large pumping plant in the United States, which provided water under high pressure.

(2) Tests made on the first fender machine erected in Gatun Lock, the chain being put into tension by a large winding engine.

(3) Actual working tests of one of the Gatun fenders in stopping large vessels.

FIRST SERIES OF TESTS (AT NEWARK, N.J.)

(1) The first set of tests was made in May 1912 in the power plant of the Prudential Life Insurance Co's. building at Newark, N.J. The public spirit of the Company in permitting the use of its plant, was much appreciated, as the tests proved of great value in giving greater confidence in this type of fender.

The arrangement of the apparatus is shown in Fig. 3. Water under pressure was supplied by three high-pressure pumps and regulated by three accumulators on the discharge lines, the pressure at the accumulators being about 750 lbs. per sq. in. while the discharge was as much as 3,400 gal. per minute. A pipe 8″ in diameter conveyed the water from the accumulators to the resistance valve. Beyond the valve an increaser was placed for connection to the 12″ Venturi tube. On the high pressure side of the resistance valve was placed an 8″ diameter gate valve, with by-pass, which was generally kept entirely open during the tests. Next to this was placed an 8″ diameter quick-opening valve also with a by-pass. The flow of water was regulated by this last valve. It is believed that the throttling of the water in passing through the latter valve was, to a great extent, the cause of the reduction of pressure during heavy flow, shown in the tests. Beyond the resistance valve the discharge pipe lead to a 12″ Venturi Meter for measuring the rate of flow.

By means of three gas-engine indicators, the drums of which were arranged to be revolved uniformly by a small electric motor, the pressures on the high pressure side of the resistance valve, at a point just above the Venturi meter, and at its throat, were continuously recorded. From the indicator cards thus obtained, the pressures at the resistance valve, and the rate of flow at any given instant of time, can be readily found. The Venturi Meter had been calibrated very accurately by its makers, the Builders Iron Foundry of Providence, R.I. The use of a Venturi where the discharge varies so rapidly as in this case, was somewhat novel. It is believed, however, that the results obtained were sufficiently accurate for the purpose.

I. C. C. VALVE:—

Seventeen tests were made in all with this valve, a summary of which is given in Table I. It will be seen that the valve worked satisfactorily for discharges as high as 750 gal. per minute. For greater rates of flow, as shown in Tests Nos. 2, 7 and 17, the results were unsatisfactory. The plunger moved up and down violently, causing severe vibrations in the piping system.

The unsatisfactory results, under heavy flow, were ascribed to the large momentum of the plunger at the time of opening, resulting in oscillations and causing water-hammer.

This type of valve was abandoned as the result of the above tests.

MUELLER VALVE:—

A total of twenty tests was made with this valve, the results of which are given in Table II. Tests Nos. 1 to 11 gave a rate of flow of 2,000 gal. per minute, or less. Tests Nos. 12 to 20 gave higher rates of flow. The accumulators ran down during test No. 14, showing that the capacity of the plant had been reached. In these tests the valve gave results that were satisfactory in every respect, proving itself capable of reducing pressures from 550 lbs. to about zero under a flow of 3,000 gal. per minute.

TABLE I

RESULTS OF HYDRO DYNAMIC TESTS WITH I. C. C. RESISTANCE VALVE

Tests 1 to 8 were made May 19, 1912, and Tests 9 to 17 on May 20, 1912.

General Arrangement as Indicated in Fig. 3.

Test No.	Water Pressure at Indicator A Lbs. per sq. in.	Venturi Pressures Inlet lbs. pr sq. in. or (lbs. pr □″)	Throat lbs. pr sq. in. or (lbs. pr □″)	Difference lbs. pr sq. in. or (lbs. pr □″)	Flow Gallons per min.	Duration of Test	Max. Lift. of Plunger	Remarks
1	About 700	0.4	0.1	0.3	600		No Record	
2	Variable Maximum 960	Variable +4.5 to +5.5 −3.5 to −3	Variable	Variable	Variable		" "	Vibrations
3	670	0.35	0.5	0.2	500	28 sec.	4–15/16″	
4	680	0.35	0.1	0.25	550	47 "	4–15/16″	
5	660	0.4	0.1	0.3	600	33 "	4–31/32″	
6	680	0.5	0.15	0.35	650	32 "	5″	

7	Variable Maximum 890	Variable			Inter-mittent	$2\frac{1}{2}$ sec.	Maximum 5-5/16″	Vibrations
8	680				Very small	29 "	5″	
9	No Record	0.5	0.2	0.3	600	16 "	4-31/32″	
10	700	Very Small			Very Small	19 "	4-31/32″	
11	660	Very Small			"	26 "	5″	
12	670	0.45	0.0	0.45	700	19 "	5″	
13	670	0.45	0.0	0.45	700	20 "	5″	
14	680	0.55	0.10	0.45	700	25 "	5″	
15	680	Very Small				24 "	5″	
16	680			0.5	750	23 "	5″	Flow,Var'bl
17	Variable Maximum 1250	Variable				$2\frac{1}{4}$ "	6″	Vibrations

TABLE II.

RESULTS OF HYDRO DYNAMIC TESTS WITH MUELLER VALVE

All Tests were made May 28, 1912.

General Arrangement as shown in Fig. 3.

Test No.	Water Pressure at Indicator A lbs. per sq. in.	Venturi Pressures			Rate of Flow		Duration of Test	Max. lift of Piston
		Inlet lbs. pr sq. in. or (lbs. pr□″)	Throat lbs. pr sq. in. or (lbs. pr□″)	Difference lbs. pr sq. in. or (lbs. pr□″)	Gallons per min.	During Time		
1	730		Negligible					none
2	730		"				21 sec.	"
3	730		Very Small				19 "	1/32″
4	700 to 730		" "				11 "	1/16″
5	700 to 720		" "				15 "	
6	650 to 670		Small				14 "	1/8″
7	650 to 680			0.5	800	8 sec.	14 "	1/8″
8	630 to 680		Small		Small		12 "	1/8″

9	600 to 650	1	–2	3	1800	5 sec.	12 sec.	3/16″+
10	600 to 650	1	–2.5	3.5	2000	5 "	12 "	7/32″
11	600 to 650	1	–2.5	3.5	2000	5 "	10 "	9/32″—
12	640	1	–3	4	2100	5 "	10 "	5/16″—
13	530 to 600	1.5	–4.5	6	2600	5 "	9 "	3/8″+
*14	500 to 570	1.5 1.5 1.5	–5.5 –6.5 –7.5	7 8 9	2800 3000 3200	5 " 3 " 1.5"	12 "	1/2″
15	500 to 550	1.5 2	–5 –6	6.5 8	2700 3000	3 " 2 "	8 "	13/32″
16	550	3 3	–6.5 –7	9.5 10	3300 3400	4 " 3 "	10 "	7/16″
17	550	2.5 2.5	–6.5 –7	9 9.5	3200 3300	4 " 3 "	8 "	7/16″ —
18	550	2 2	–6 –7	8 9	3000 3200	4 " 3 "	8 "	13/32″+
19	550	2 2	–6.5 –7	8.5 9	3100 3200	3 " 2 "	8 "	13/32″+
20	550	2 2 2	–4.5 –6.5 –7	6.5 8.5 9	2700 3100 3200	5 " 4 " 3 "	8 "	13/32″+

* Accumulators ran down.

TABLE III.

RESULTS OF HYDRO DYNAMIC TESTS WITH ROSS VALVE.

Tests 1 to 14 were made May 22, 1912, and Tests 15 to 27 on May 23, 1912.
General Arrangements as shown in Fig. 3.

Test No.	Pressure at Indicator A Lbs. per sq. in.	Venturi Pressures			Rate of Flow		Duration of Test	Remarks
		Inlet lbs. pr sq. in. or(lbs. pr □″)	Throat lbs. pr sq. in. or(lbs. pr □″)	Difference lbs. pr sq. in. or(lbs. pr □″)	Gallons per min.	During Time		
1	660				Very Small			
2	660				"			
3	670				"			
4	670	Variable		0.5	750			
5	670 to 680	"			Small			
6	670 to 680	"			"			
7	670 to 680	"			"			
8	670 to 680	"			"			
9	670 to 680	"		0.5	750			
10	670 to 680	"		0.7	900			
11	680 to 700	"			Small			
12	680 to 700	"		1.0	1050			
13	680 to 700	"		1.0	1050			
14	660	"		3.0	1800	2 sec.		
15	670	"		2.5	1700	3 "		

16	670	0.5	–2.0	2.5	1700	4 sec.		
17	620 to 680	0.5	–2.0	2.5	1700	3 "	8 sec.	
18	600 to 650	1.0	–2.5	3.5	2000	3 "	9 "	
19	600 to 650	1.0 1.0	–4.5 –2.5	5.5 3.5	2500 2000	2.5" 3.5"	10 "	
20	590 to 630	1.5 1.5	–5.5 –2.5	7.0 4.0	2800 2100	2 " 4 "	9 "	
21	550 to 620	1.5 1.5	–5.5 –3.0	7.0 4.5	2800 2200	2 " 3 "	8 "	
22	550 to 620	2.0 1.5	–5.0 –3.0	7.0 4.5	2800 2200	2 " 3 "	6 "	
23	550 to 620	2.0 1.5 1.0	–5.0 –3.5 Max.–7	7.0 5.0 Max. 8	2800 2400 3000	2 " 3 "	8 "	
24	550 to 620	1.5 1.5 1.5	–4.0 –5.5 Max.–6	5.5 7.0 7.5	2500 2800 2900	3 " 2 " 1.5"	7 "	
25	520 to 620	1.5 1.5	–4.0 –6.0	5.5 7.5	2500 2900	3 " 2 "	7 "	
26	550 to 620	1.5 1.5	–5.5 –6.5	7.0 8.0	2800 3000	3 " 2 "	7 "	
27	550 to 620	2.0 1.5	–6.0 –7.0	8.0 8.5	3000 3100	3 " 2 "	7 "	

Typical indicator diagrams from Test No. 16 have been reproduced in Figs. 4, 5 and 6.

ROSS VALVE:

A total of twenty-seven tests was made with this valve, the results of which are given in Table III. Tests Nos. 1 to 18 gave rates of flow of 2,000 gal. per minute, or less. Tests Nos. 19 to 27 gave higher rates of flow.

Indicator diagrams from Test No. 23, are reproduced in Fig. 7, 8 and 9 as typical of these tests.

The action of this valve was also satisfactory in every respect.

It will be noted that in this, as in the other tests, there was a decided drop in the pressures maintained, in the larger as compared with the smaller discharges.

As the results of these tests made at Newark, four valves, two, of the Ross and two of the Mueller type were purchased for further trial on the Isthmus.

SECOND SERIES OF TESTS (MADE IN GATUN LOCK)

These tests were made in the winter of 1913 on the first fender erected on the Isthmus, which was in the upper approach to the Gatun locks. One Ross and one Mueller valve had been attached to each machine. Only one valve was used at a time, the other being entirely shut off from the piping. At this time there was no water in the locks. It was intended that these tests should be made, as nearly as possible, under the conditions that would prevail with the fenders in actual operation. The chain had been connected to the machines on both the walls and was stretched across the lock at the top. It was proposed to fasten a cable to the chain at a point half way across the chamber wall, and attach the other end to a large winding engine on one of the walls. In this way, a pull along the axis of the lock would have been exerted, the effect being similar to the thrust of a vessel striking the chain. It proved necessary to modify this programme slightly. The middle section of the chain was detached and the cable connected directly to the end of the chain where it emerged from the hawse-pipe on one of the walls—. The arrangement is shown in Figs. 10, 12 and 14. In the first series of tests (Fig. 10) the cable made an angle of $12\frac{1}{3}^0$ with the wall so that there was friction between the chain and the hawse-pipe casting. In the second Series (Fig. 12) a snatch block was fastened to the opposite wall so that the cable made an angle of nearly 90° with the wall and did not touch the hawse-pipe. In the third arrangement (Fig. 14), a second block was added in order to increase the pull on the chain.

For these tests a Lidgerwood Unloader, consisting essentially of a 60 ton winding engine mounted on a flat car, and supplied with steam from a locomotive, was placed on the wall opposite the fender machine to be tested, and about 600 ft. from the same; the tension in the chain was produced by winding in on the unloader, thus causing the moving cylinder of the fender machine to tend to rise, producing a pressure in the upper cylinder. The amount of pressure depended on the setting of the resistance valves. The cylinder pressures were shown by gauges and continuously recorded by the indicators which were used in the tests made in the United States. The valves were set, by trial, for gradually increasing pressures.

A number of runs were made with the arrangement shown in Fig. 10, the pressure varying from 170 to 350 lbs. per sq. in. When the pressure exceeded 350 lbs. the unloader was apparently unable to overcome the hydraulic resistance and the hawse-pipe friction and internal resistances in the machinery.

A typical run is shown in Fig. 11.

After the valves had been adjusted, the pressure curves were very uniform for both types of valve, with practically constant pressures throughout the stroke, except for the small oscillations due to chain friction on the hawse-pipe. The plunger speed varied from 6 to 25 ft. per minute (equivalent to flows of 350 to 1470 gal. per minute), being limited by the capacity of the unloader. The low speeds correspond to the highest pressures.

Four runs were made with the second arrangement, when the cable parted owing to imperfections. The pressures ranged from 310 to 370 lbs. per sq. in., as shown on the typical card, Fig. 13. The pressures were perfectly constant and steady, without any of the small variations due to hawse-pipe friction.

For the third set of tests there was a three fold multiplication of the rope (Fig. 14), the pull being again nearly normal to the walls. A series of runs was made, using the two valves alternately, with pressures running up to 550 lbs. per sq. in. at plunger speeds as high as 8.4 ft. per minute. In the final test a maximum pressure of 630 lbs. was reached when the chain broke in a flaw. Fig. 15 shows a typical card.

The pressure curves obtained with both valves, in the last series of tests, were also entirely uniform and satisfactory. In these tests, as in those previously made in the United States, both types of valve gave equally good results. The choice became a difficult one. The simplicity of the Mueller Valve was in its favor. The Ross valve appeared, however, to have two advantages. In the first place, it is more readily set for the desired pressure, as the auxiliary valve is controlled by very small springs. In the second place, it was possible to connect the auxiliary valve by a small pipe directly with the large machine cylinder, and thus control directly the pressure in the cylinder itself, and not the pressure in the piping close to the valve. In this way, the variable drop in pressure between the cylinder and the point in the piping at which the valve is attached, was entirely eliminated. For these reasons, the Ross Valve was finally selected for use in all the fenders.

THIRD SERIES OF TESTS (MADE IN GATUN LOCK) TO DETERMINE FRICTIONAL RESISTANCES

In order to determine the most suitable pressure for setting the valves, a further set of experiments was made in February 1914. Their purpose was to measure the friction of the chain on the hawse-pipe and the frictional and other resistances in the machinery. The arrangement of the apparatus is shown in Fig. 16. It differed from that previously used by the addition of an hydraulic dynamometer, for measuring the strain in the wire rope, close to the winding engine. This dynamometer was compared with a standard spring dynamometer and found to be practically frictionless and to give correct results. Indicator cards were taken simultaneously at this dynamometer and at the upper cylinder of the fender machine. From these the effect of friction, etc. was readily determined.

Two series of tests were made. In Series I, the chain pull was at 90° with the lockwall, so that there was no friction at the hawse-pipe. In Series II, the pull was at $25\frac{1}{2}^\circ$ with the wall so that the total resistance included hawse-pipe friction.

The observations are plotted in Figs. 17 and 18 in which the abscissas and ordinates are respectively the pressure (P) in the machine cylinder and (p) at the dynamometer, both in lbs. per sq. in.

There is a four-fold multiplication between the movement of the machinery cylinder and the travel of the chain, and a further three-fold multiplication in the wire cable, while the cross sections of the machine and dynamometer cylinders are respectively 1134 and 380 sq. in. Hence, if friction and the weight of the moving cylinder be entirely disregarded, the corresponding values of p and P whould be given by the equation:

$$p = \frac{1134}{3 \times 4 \times 380} \times P = 0.25\ P$$

In Fig. 17 the simultaneous readings in the two cylinders, when there was no hawse-pipe friction, were plotted. It was found that a straight line could readily be drawn, which fairly represented the actual readings, and was at the same time parallel to the line of no friction $p=0.25$ P. This showed that the sum of the internal machine friction, and the weight of the moving parts is constant for all pressures. It is represented by the difference between the ordinates of the two lines and found to be equal to 30 lbs. per sq. in. in the dynamometer, corresponding to a pull of $3 \times 380 \times 30=34{,}200$ lbs. in the chain.

The second series of measurements is plotted in Fig. 18. The observations cannot be as closely represented by a linear equation as in the previous case, though the line drawn in the figure gives a fair average. The difference between the ordinates of this line and the upper line in Fig. 17 represents the effect of the hawse-pipe friction. Its effect on the pressure at the dynamometer is $p=0.20$ P.

Taking hawse-pipe friction, and all other resistances, into account, we have:

$$p=0.25\ P+30+0.20\ P=0.45\ P+30.$$

If now T_1 and T_2 represent the pull on the chain inside and outside the hawse-pipe respectively, we readily obtain from the previous equations:

$$T_1=\frac{1134}{4}P+34{,}200=283.5\ P+34{,}200.$$

and

$$T_2=3 \times 380\ p=1140\ p=513\ P+34{,}200$$

while their difference—

$T_2-T_1=229.5$ P represents the effect of hawse-pipe friction.

As noted above, series II gives rather irregular results for the value of p. This was doubtless due in part to the wear of the chain and hawse-pipe during the tests. Flat places were worn on the side of the chain links as wide as $2\frac{3}{4}''$ and as long as $7''$. and on the hawse-pipe, $7''$ wide and $40''$ long, the maximum depth in both cases being about $\frac{1}{4}''$. This wear of the chain and hawse-pipe evidently increased the friction as is shown on plotting the values of p, which were larger for the later observations.

It becomes still more apparent on computing the coefficient of friction. This can be done by the formula:

$$f=\log\left\{\frac{T_2}{T_1}\right\}\div\frac{64.5}{360}\times 2.73=\log.\frac{T_2}{T_1}\div 0.489$$

in which f = coeff. of friction.

64.5 = arc of contact of chain around hawse-pipe, while the log. is a common or Briggsian logarithm (See Unwin Machine Design, New Edition, Part I p. 447). The coefficients are plotted in Fig. 19. The higher values of f in the later experiments become quite clear on inspection.

The principal purpose of the foregoing tests was to determine the proper pressure for the setting of the resistance valves.

For a given cylinder pressure, as shown above, the maximum pull on the chain; i.e., the pull outside of the hawse-pipe, is

$$T_2 = 513\ P + 34{,}200$$

The proper value to be used for T_2 depends on the strength of the chain; as shown by test.

The minimum breaking strength, permitted for any link, was 400,000 lbs. and all chains had also to stand a proof test of 245,000 lbs. without sign of failure.

As there will also be some secondary stresses due to bending of the chain around the sheaves and to other causes, it seemed best to limit the working stress to 220,000 lbs. For this value of T_2 we have—

$$P = \frac{T_2 - 34200}{513} = \frac{220{,}000 - 34200}{513} = 362$$

It was therefore decided to fix the cylinder pressure at 360 lbs. per sq. in. At this pressure, of the total chain pull of 220,000 lbs., $\left(\frac{1134}{4}\right.$ x 360) or 104,120 lbs. is due to the cylinder pressure i.e., only about one-half the total stress, the rest being the result of the weight of the moving parts, the internal resistances of the machinery and the friction at the hawse-pipe.

FOURTH SERIES OF TESTS (MADE IN GATUN LOCK)

TESTS IN STOPPING VESSELS

The foregoing tests gave a reasonable assurance that the fenders would function properly in stopping vessels. Twenty-two of the fenders were therefore built, practically identical in plan, while two others (in the lower approach to Miraflores Lock) differ only in having two chains stretched across the lock at different levels, to provide for the great difference (22 ft.) between high and low tides in the Pacific. Their machinery is absolutely identical with that in the other fenders, the high and low level chains being alternately connected and detached, as the tide changes.

It was, of course, desirable, to make an actual test of the fenders in checking a vessel in the lock.

In October and November 1915, after the writer had left the Isthmus, a number of such tests were therefore made by a Board appointed by the Governor of the Canal. They proved of great interest and value especially as the vessels were of considerable size. Two ships were used, the "Alliança" having at the time a displacement of 4,221 tons, and moving at speeds varying from 1.23 to 3.38 miles per hour, and the "Cristobal" with a displacement of 18,000 tons and speeds as high as 2.45 miles per hour.

The resistance valves were set to open at 360 lbs. per sq. in. in most of the tests, and the propellers of the vessels were stopped in every case before the chain was struck. Indicators were connected to the piping system in both the machinery rooms (Nos. 810 and 811—Gatun lock), and the pressures and also the travel of the moving cylinder of the machines were automatically recorded.

A rope mat was woven around the central portion of the chain and a similar protection given to the stem of the ship. The Photograph Pl. 3 shows the "Cristobal" approaching the fender. No damage to the ships occurred as a result of the tests, and the chain was marred only very slightly. Twelve runs were made with the "Alliança" and ten with the "Cristobal", and the vessel was brought to a stop in every case before the chain had paid out to its extreme limit.

The tests with the "Alliança" were not entirely satisfactory, as the resistance valves had not been cleaned for a long time, and there was a slight sticking of the valves which prevented them from closing promptly when the pressure was reduced. All the valves were, therefore, thoroughly cleaned, a new leather placed in one valve, and the other leathers softened up.

The tests with the "Cristobal" were made after these changes were made and proved entirely satisfactory. The indicator cards taken with this vessel are shown in Figs. 20, 21, 22 and 23, which also give other information in connection with these tests.

The pressure curves have a decided peak at the beginning, which is in every case decidedly above the setting of the resistance valve. Beyond this point, and throughout the greater part of the stroke the pressure remains remarkably uniform, with very few oscillations. The vessel was brought to rest from 51.5 to 62.0 ft. beyond the center of the fender, its speed when striking the chain being from 2.06 to 2.45 miles per hour.

There was little difference between the pressures in the machines on the two lockwalls or in the length of chain paid out from each side. The travel of the cylinders was hardly over 6 ft. out of a total possible stroke of 21.5 ft.

The distance travelled by the ship before being stopped was less than the shortest distance from any of the fenders to the gate it is intended to safeguard, so that there seems to be every assurance that the fenders, if ever called upon, would fulfill their purpose, even in the case of a ship as large as the "Cristobal" and moving at a speed of over 2 miles per hour. As this vessel is about 500 ft. long, and 58 ft. wide, few larger ships are likely to use the Canal, nor is the speed of two miles likely to be exceeded in the approaches or the locks.

It is a matter of considerable interest to note that the distance in which the "Cristobal" was stopped agrees very closely with the theoretical curves which were computed before the designs were completed, but after the working stress of 220,000 lbs. had been adopted for the chain in stopping vessels. These curves are shown on Pl. 2, which is copied from the Annual Report of the Isthmian Canal Commission for 1911. This close agreement with theory is, of course, very satisfactory.

Accounts of the various tests, though in less detail, are given in the Annual Reports for 1913, 1914 and 1916.

DISCUSSION

Mr. French

Mr. R. DeL. French, Assoc. Mem., Can. Soc. C.E.—The most interesting part of this paper, to the writer, is the account of the tests on the resistance valves carried out by Mr. Goldmark at Newark.

As he remarks, the use of a Venturi meter for measuring such rapidly fluctuating flow is somewhat novel. The Builders Iron Foundry, in their bulletins on the use of the Venturi, state that the meter *should* be preceded by a straight pipe at least six times as long as the diameter of the meter tube. This condition does not seem to have been complied with in Mr. Goldmark's experiments. However, the makers are prepared to issue special instructions for cases where this requirement cannot be met, and it may be that it was taken into account in the "accurate calibration" referred to.

From the description of the "resistance valves," they seem to be practically identical with the reducing valves used in many water works systems. Little data is available regarding the loss of head through these valves under different conditions of flow, and it would be of great interest to water works engineers in general if Mr. Goldmark were able to furnish some information on this point. The apparatus shown in Fig. 3 is well fitted for loss-of-head experiments, requiring only the addition of two pressure gauges, one on each side, close to the valve under test. Possibly Mr. Goldmark made some observations of this sort, and has purposely omitted them from his paper as not relevent to the subject.

Mr. Goldmark is to be congratulated on the close agreement between the expected and the actual results, as shown by the tests with the "Allianca" and "Cristobal," particularly as there was no precedent to help in the solution of the problem.

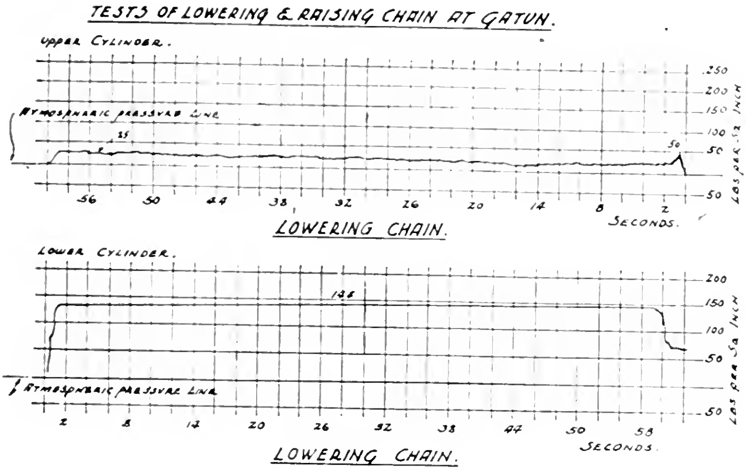

TESTS OF LOWERING & RAISING CHAIN AT GATUN.
UPPER CYLINDER.
ATMOSPHERIC PRESSURE LINE
LOWERING CHAIN.
SECONDS.
LBS. PER SQ. INCH
LOWER CYLINDER.
ATMOSPHERIC PRESSURE LINE
LOWERING CHAIN.
SECONDS.
LBS. PER SQ. INCH

FIG. 1

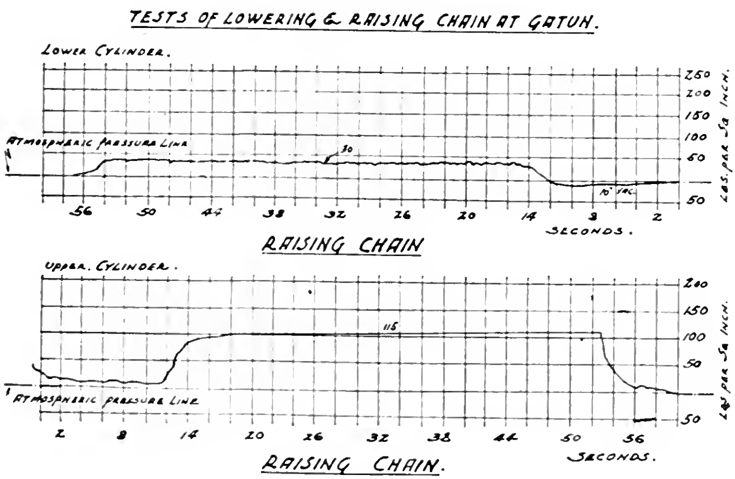

TESTS OF LOWERING & RAISING CHAIN AT GATUN.
LOWER CYLINDER.
ATMOSPHERIC PRESSURE LINE
RAISING CHAIN
SECONDS.
LBS. PER SQ. INCH.
UPPER. CYLINDER.
ATMOSPHERIC PRESSURE LINE
RAISING CHAIN.
SECONDS.
LBS. PER SQ. INCH.

FIG. 2

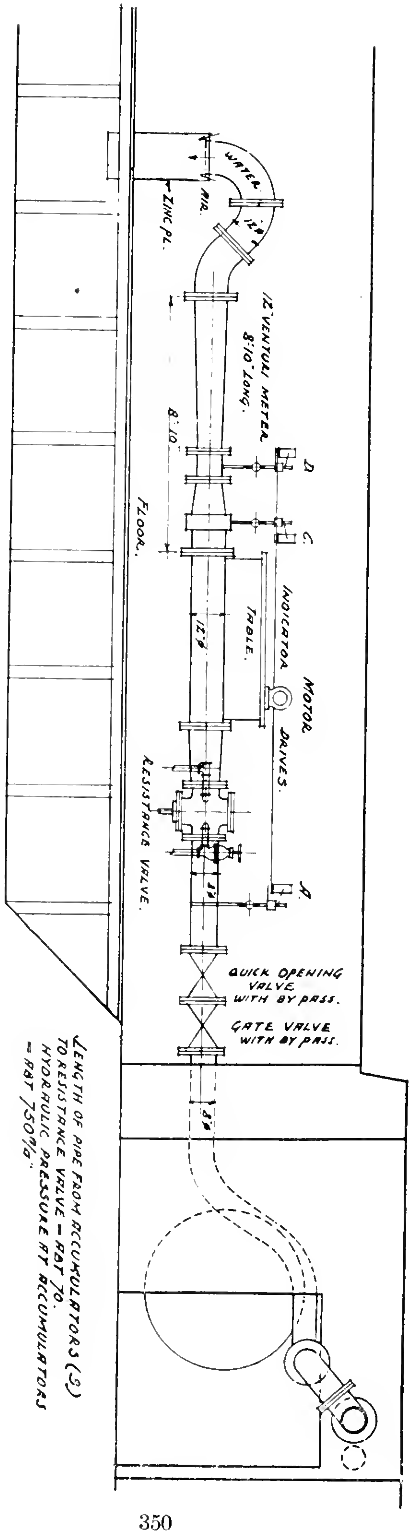
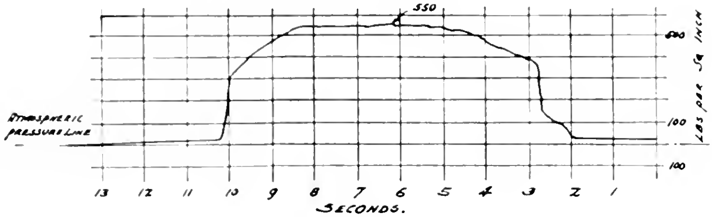


FIG. 3.

ARRANGEMENT OF APPARATUS. TEST MADE AT NEWARK, N.J.

ABOVE RESISTANCE VALVE.



TESTS OF MUELLER VALVE AT NEWARK, N.J.

FIG 4

VENTURI INLET.

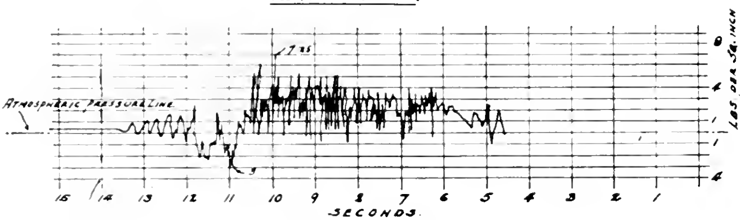


FIG. 5

VENTURI THROAT.

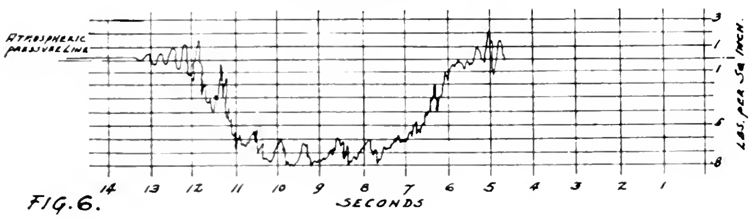


FIG. 6.

TESTS OF ROSS VALVE AT NEWARK, N.J.

ABOVE RESISTANCE VALVE.

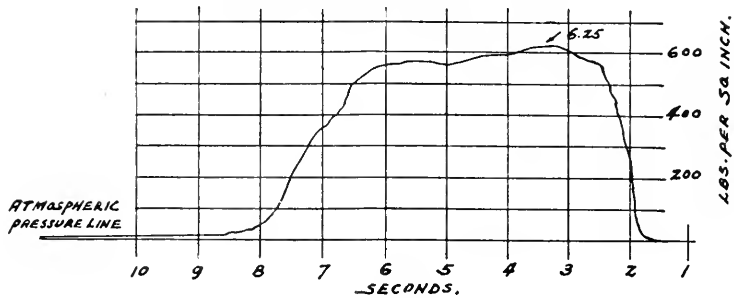


FIG. 7.

VENTURI INLET.

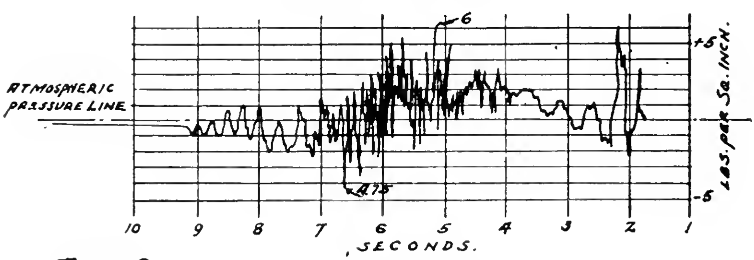


FIG. 8.

VENTURI THROAT.

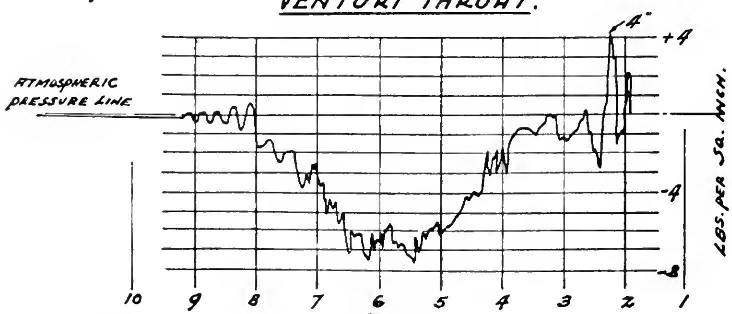


FIG. 9.

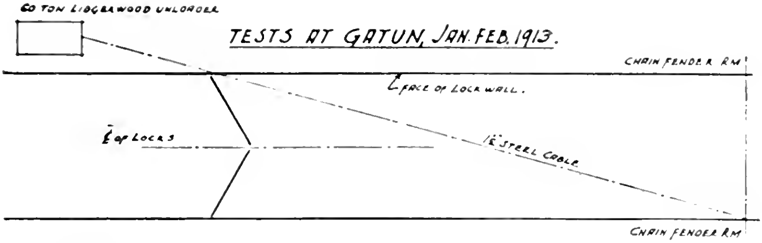


FIG. 10.

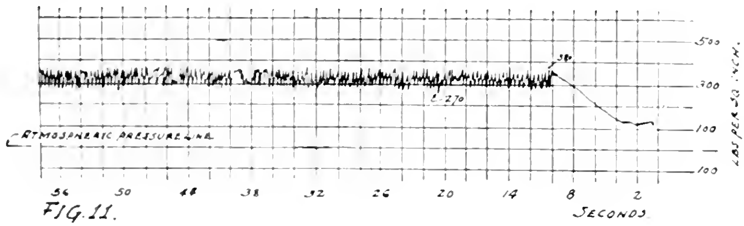


FIG. 11.

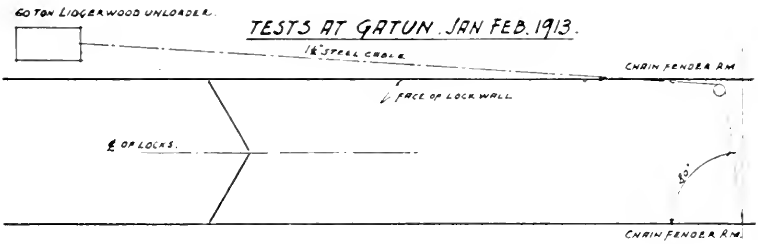


FIG. 12.

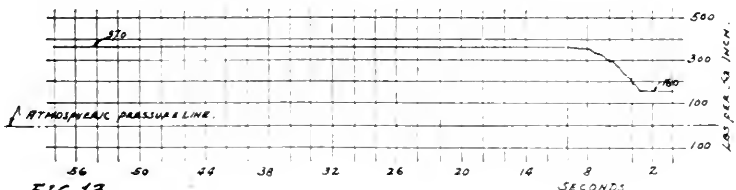


FIG. 13.

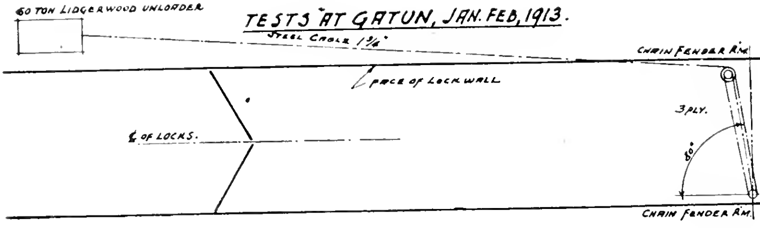


FIG. 14.

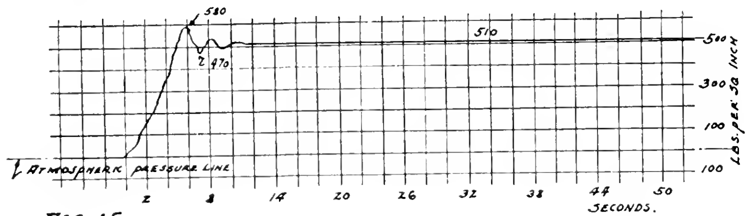


FIG. 15.

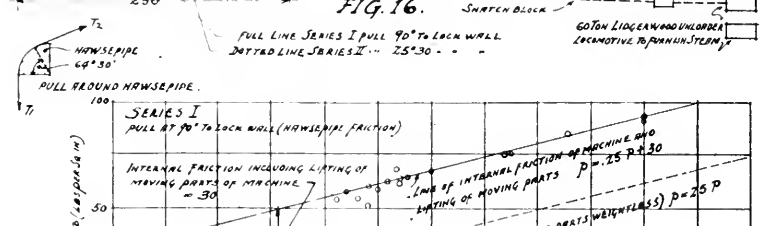


FIG. 16.

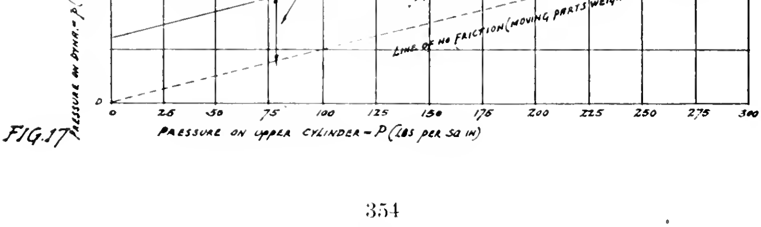


FIG. 17

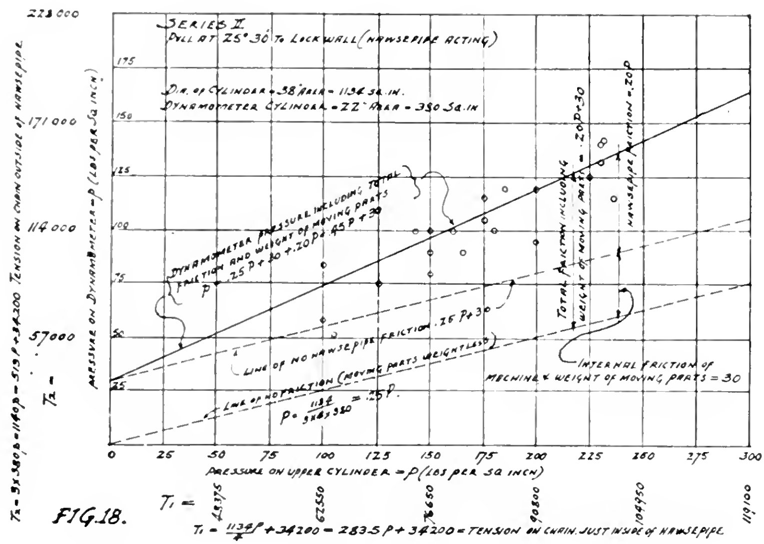
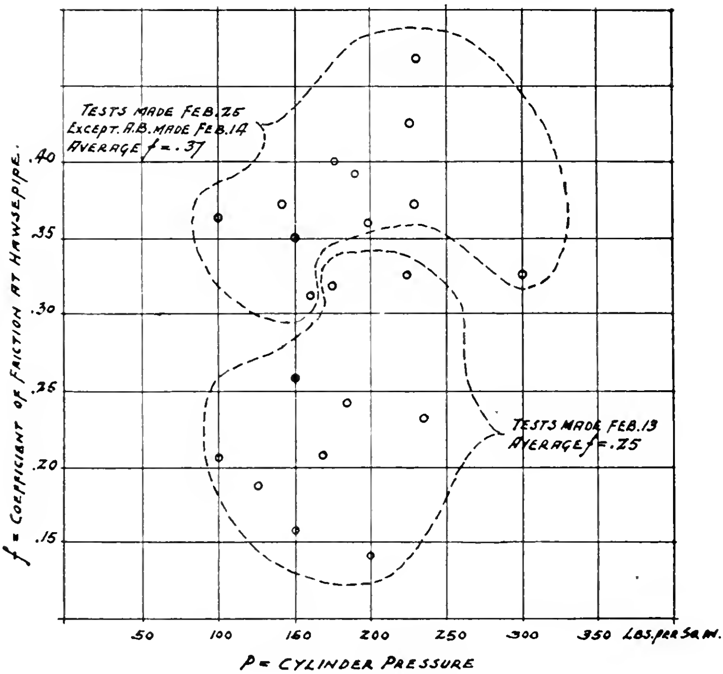


FIG. 18.

TESTS AT GATUN, FEB. 1914.



ARC OF CONTACT AT HAWSEPIPE = $64°-30'$

COEFF. OF FRICTION $f = \log.\frac{T_2}{T_1} \div \frac{64.5}{360}\ 2.73 = \log.\frac{T_2}{T_1} \div .489$

T_2 = CHAIN PULL OUTSIDE OF HAWSEPIPE = $1140p$.

T_1 = " " INSIDE OF " " = $283.5p + 34,200$

p = DYNAMOMETER PRESSURE AS REGISTERED.

P = CYLINDER " " "

FIG. 19.

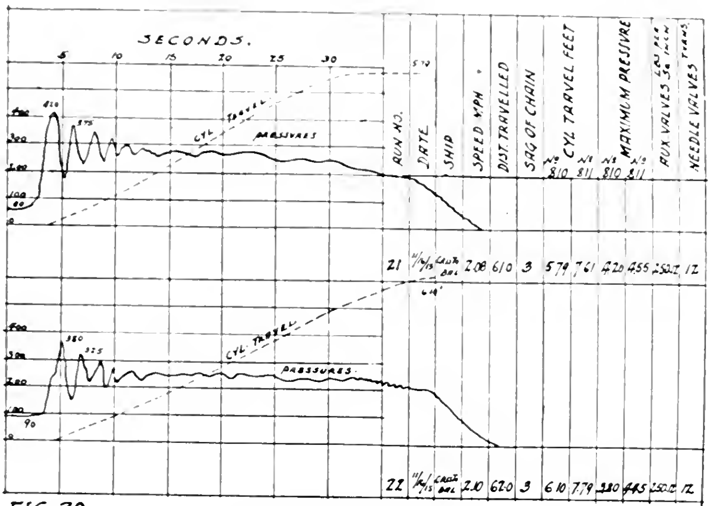


FIG. 20. TESTS AT GATUN, NOV. 1915.

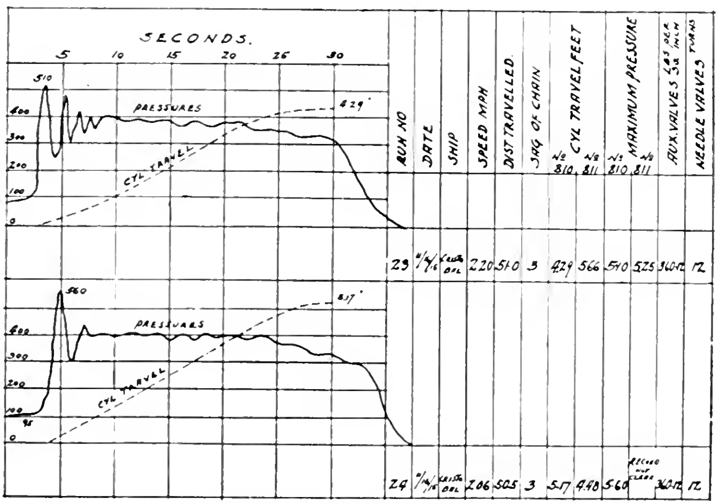
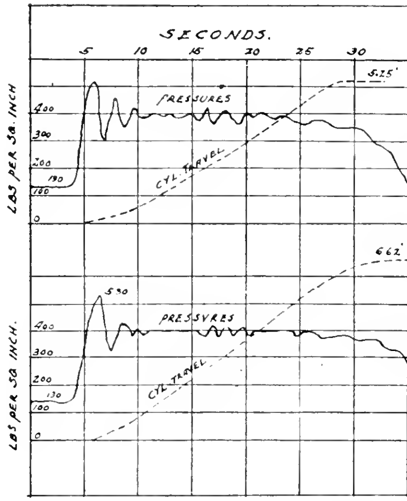
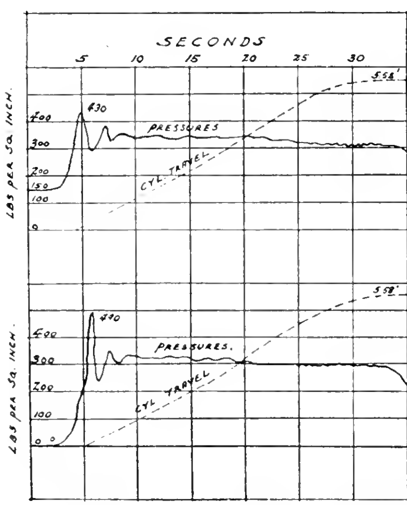


FIG. 21. TESTS AT GATUN, NOV. 1915.



RUN NO	DATE	SHIP	SPEED MPH	DIST TRAVELLED	SAG OF CHAIN	CYL. TRAVEL FEET No 810	CYL. TRAVEL FEET No 811	MAXIMUM PRESSURE No 810	MAXIMUM PRESSURE No 811	AUX. VALVES LBS PER SQ. INCH	NEEDLE VALVES TURNS
25	11/16/15	CRISTOBAL	2.14	515	3	5.25	4.62	5.20	5.25	360.12	12
26	11/16/15	CRISTOBAL	2.45	575	3	6.62	5.58	5.30	5.45	360.12	12

FIG. 22. TESTS AT GATUN, NOV. 1915.



RUN NO.	DATE	SHIP	SPEED MPH	DIST TRAVELLED	SAG OF CHAIN	CYL. TRAVEL FEET No 810	CYL. TRAVEL FEET No 811	MAXIMUM PRESSURE No 810	MAXIMUM PRESSURE No 811	AUX. VALVES LBS PER SQ. INCH.	NEEDLE VALVES TURNS
27	11/16/15	CRISTOBAL	2.10	565	3	5.58	6.19	4.30	4.50	360.12	12
28	11/16/15	CRISTOBAL	2.10	560	3	5.58	6.81	4.90	5.10	360.12	12

FIG. 23. TESTS AT GATUN. NOV. 1915.

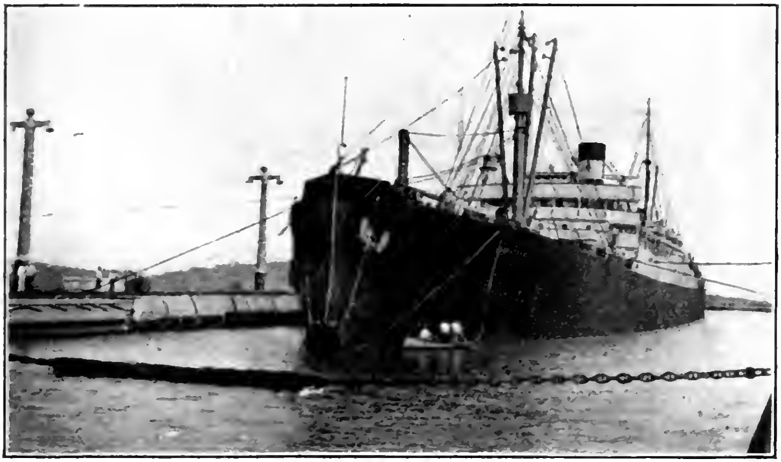
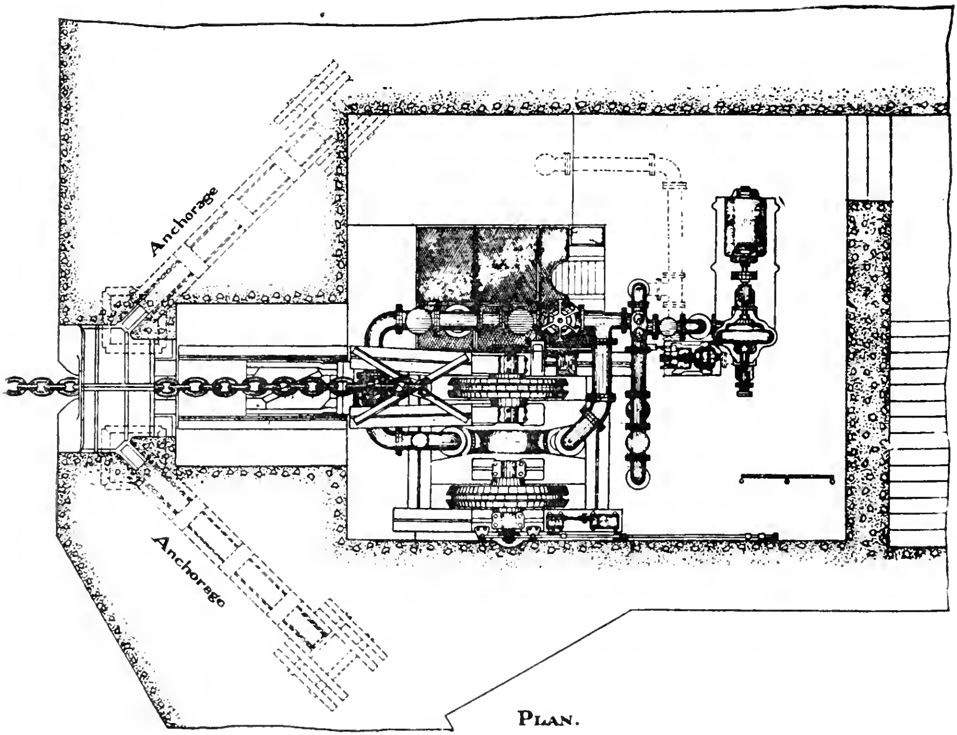
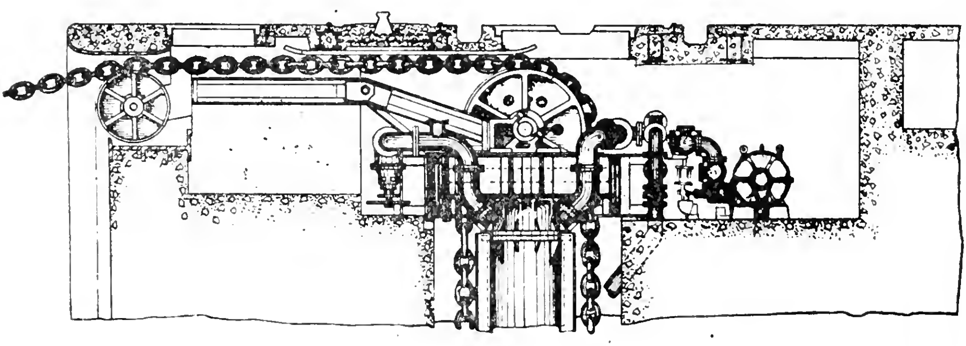


PLATE 3.

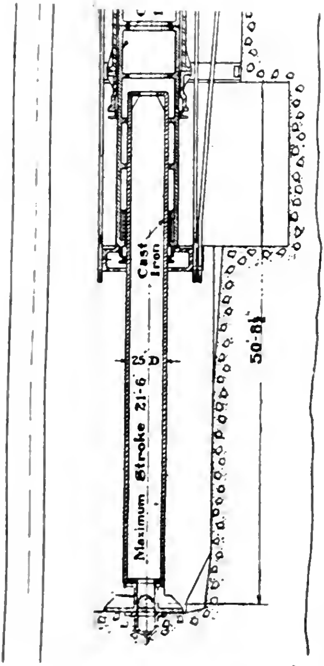


PLAN.

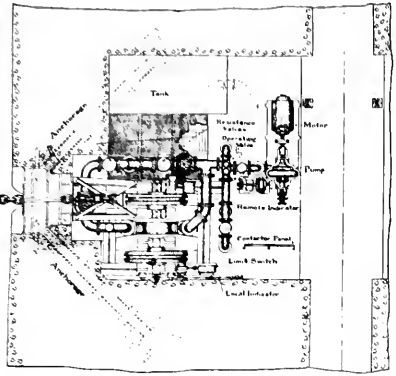
Typical Pit for Fenders at Approaches



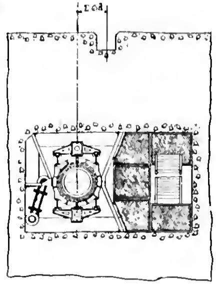
ELEVATION.



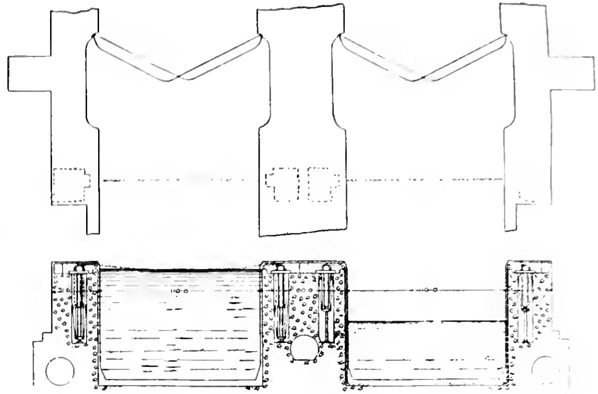
ION THRU CYLINDER.



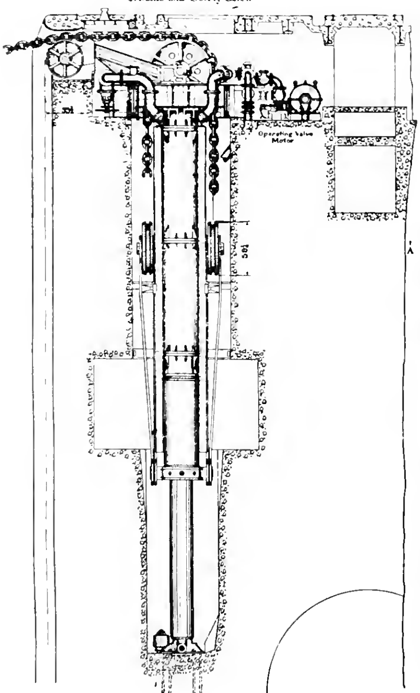
PLAN
Typical Pit for Fenders above ¢ Middle and Safety Gates



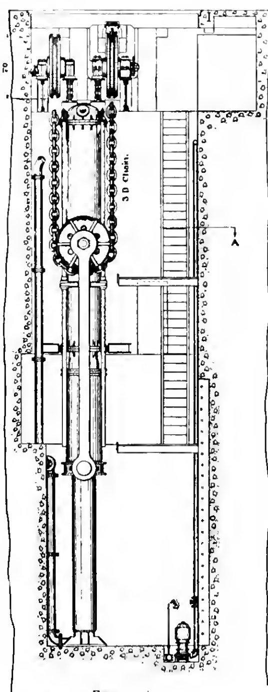
SECTION AA.



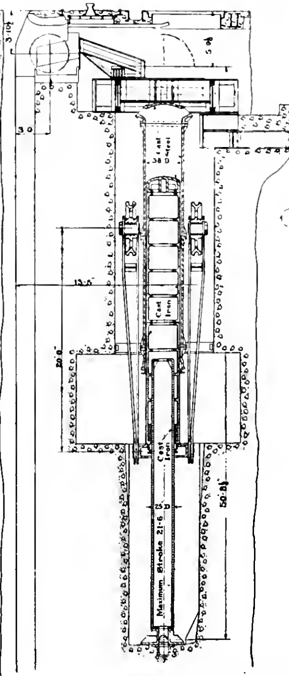
GENERAL ARRANGEMENT - Scale



ELEVATION.



ELEVATION'.



SECTION THRU CYLINDER.

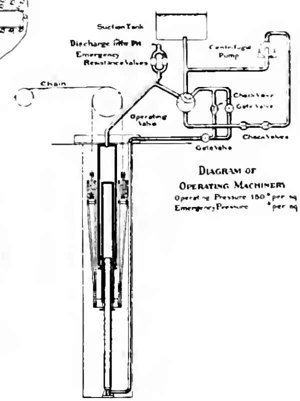
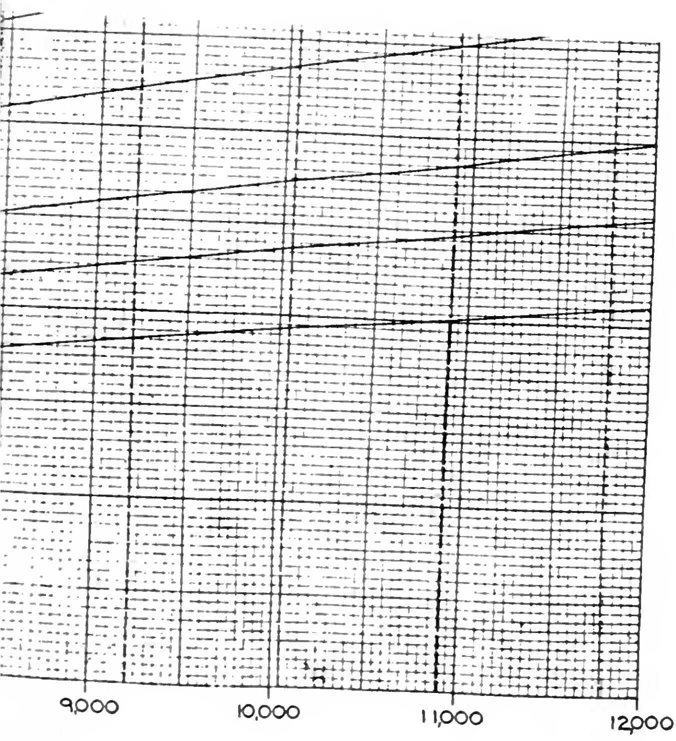


DIAGRAM OF OPERATING MACHINERY
Operating Pressure 150 # per sq in
Emergency Pressure # per sq in

PLATE I



9,000
10,000
11,000
12,000

Note – The calculations are based on the following assumptions –

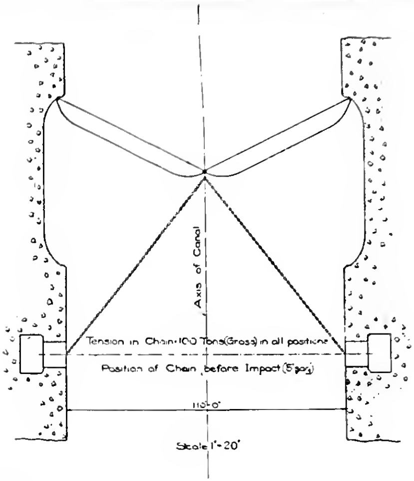
The chain is stretched across the lock and, when struck by a vessel, pays out under a constant tension of 100 gross tons. Its resistance is the only force tending to change the speed of the vessel after it strikes the chain.

The curves give the kinetic energy of various vessels for initial speeds of from 1 to 6 knots.

The abscissas of the points 20', 25' etc give the total energy absorbed by the chain, paying out equally from each wall, after the vessel has traveled 20', 25' etc.

The ordinates to the curves for the points 20', 25' etc give the speeds from which the vessels will be stopped after traveling 20, 25 etc.

Thus a vessel of 5,000 tons will be stopped in 70' if its initial speed was 5.5 knots.



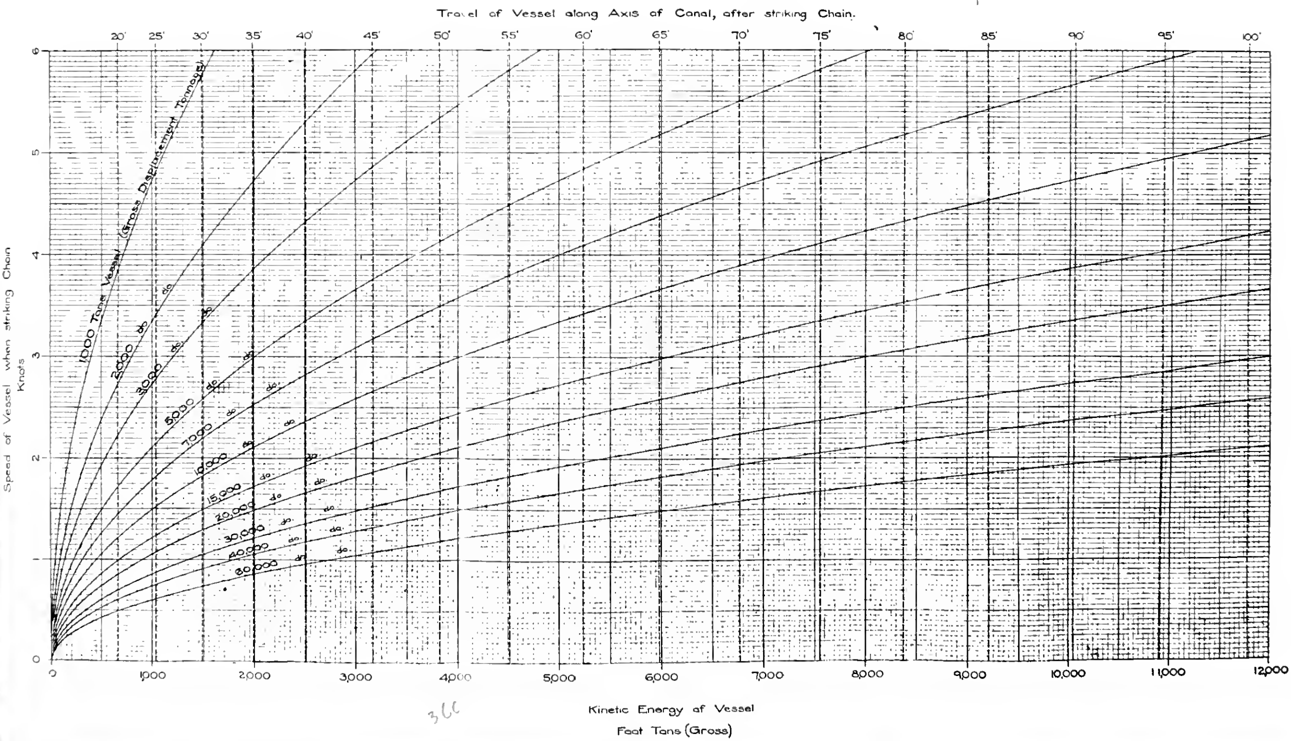


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PLATE II

NICKEL-COPPER STEEL

By R. W. LEONARD, M.E.I.C.

(Read at a Meeting of the Institute on March 28th, 1918)
Revised to date—August 31st, 1919.

In the early eighties, during the construction of the Canadian Pacific Railway through what is now known as the Sudbury District, some copper ores were discovered, and subsequently a Company was formed to develop the ore bodies. This Company—The Canadian Copper Company—sent its ore or matte to the Orford Copper Company's refinery at Constable Hook, N.Y., for treatment, which plant was established for the purpose of treating the copper ores mined at Orford Mountain, P.Q.

When it was realized that these copper ores contained substantial quantities of nickel—for which metal there was very little demand at that time except for the purpose of making German Silver and for nickel-plating—the Canadian Copper Company and the Orford Copper Company were merged into the International Nickel Company. This Company developed very large properties at Sudbury and greatly stimulated the demand for nickel, especially for the purpose of alloying with steel to be used for the many purposes so well known to all engineers of the present day.

During the past three years the International Nickel Company has constructed a refinery at Port Colborne, Ont., for the purpose of completing the process of separation of the nickel from the copper and of refining these products in Canada for the Canadian and foreign trade.

The Mond Nickel Company, of England, also acquired properties in the Sudbury District, and ships its partially manufactured product to England in the form of matte.

Latterly, the British-America Nickel Corporation has acquired large mineral claims which it is developing, and is erecting in the Sudbury District metallurgical works to treat its ores, and at Deschesnes, P.Q., an extensive refining plant.

Although the different Companies are pursuing somewhat different processes, in general the operation consists in mining and sorting the ore, then roasting it (generally in open heaps or in stalls) to largely eliminate the sulphur, and thereafter smelting in the ordinary type of copper-smelting furnaces to a matte, consisting of sulphide of iron, nickel and copper, which matte is further bessemerized for the purpose of eliminating as much of the iron as possible and producing a matte much richer in nickel and copper content, and containing therein some small proportion of the precious metals.

Until the Port Colborne plant started operations a few months ago these mattes were exported either to England or to the United States for refining or separation of the nickel from the copper, and in one or two cases for the recovery of the accompanying precious metals.

The above-outlined process results in the waste of all the sulphur content of the ore, amounting to many hundreds of tons of elemental sulphur per day, to the serious damage of all plant life in the immediate neighborhood. It also results in the waste of some thousands of tons per day of iron in the slags, which until recently was considered a necessary waste preparatory to separating the nickel from the copper.

The Sudbury nickel deposits, as developed by the mining companies and as worked out by geologists, consist of an oval saucershaped basin about 36 miles in length by 18 miles in width, around the South rim of which are located most of the properties of the Canadian Copper Company and the Mond Nickel Company, and around the northerly and easterly rim of which are principally located the properties of the British-America Nickel Corporation. The Towns of Sudbury and Copper Cliff are on the south rim. The Canadian Northern Ontario Railway passes in a Northerly direction through the Eastern portion of the basin, and the Canadian Pacific Railway passes almost through the centre in a north-westerly direction.

An excellent geological map of the deposits accompanies the Monograph on the Sudbury Nickel Region, by Dr. A. P. Coleman, 1913. This basin is generally conceded to be one of the greatest mineral deposits of the world, containing nickel-copper pyrrhotite of unequalled quantity and richness, which can be mined and the nickel and copper extracted and refined at a cost defying competition.

It is now the principal source of the world's nickel, and is also the source of a considerable amount of platinum and palladium, which are recovered as by-products.

The magnificent Report of the Royal Ontario Nickel Commission, 1917, which bears on the nickel production of the world, is probably one of the most complete and valuable reports on any mineral industry extant.

A number of men have experimented with alloys of nickel, iron and copper, commencing with Alexander Parks in England in 1844, who patented a "useful alloy of nickel, iron and copper." Hybinette and Shuler, of Sudbury, made experiments about 1902 in the manufacture of nickel pig from Sudbury ores, and E. A. Sjostedt carried out some experiments in the direct manufacture of nickel pig and nickel steel at Sault Ste. Marie, Ont., for the Lake Superior Corporation about the same time. In 1905, Dr. E. Haanel, Director of the Mines Branch, Geological Survey, Ottawa, made, experimentally, some nickel-copper-iron pig from roasted pyrrhotite in an electric furnace at Sault Ste. Marie, under Canadian Government auspices. W. S. Horrey experimented in the smelting of nickel-copper-iron ores at Sault Ste. Marie in 1898. "Metallurgical and Chemical Engineering" of February, 1913, gives a description of the manufacture of nickel steel from nickel pig in an electric furnace at Trondjën, Norway.

In all these experiments an endeavor was made to select ores in which the copper bore the smallest possible proportion to the nickel content, it being believed or feared that the copper was an injurious constituent, except in the case of Shuler, who claimed that the presence of the copper in certain proportions was not objectionable.

Mr. G. H. Clamer, of Philadelphia, has used Monel Metal (a natural alloy of nickel and copper as obtained from the Sudbury ores after elimination of the iron and sulphur) in the manufacture of a nickel-copper steel which is in commercial use and is said to have been successfully employed even in the manufacture of armour piercing shells for the United States Government, and the results are reported to have been very satisfactory. This nickel-copper steel is also being manufactured into commercial sheets.

Mr. George M. Colvocoresses, at one time in the employ of the Orford Copper Company and who has had a valuable experience in the mining and metallurgy of nickel and copper in Canada and New Caledonia, made laboratory experiments in the production of nickel-copper steel direct from the Sudbury ores, and has taken out patents on his process. In these patents he claims the direct smelting of Sudbury nickel-copper ores or the slags wasted in the present process, either by electricity or with fuel, into a nickel-copper pig, and ultimately refining resultant pig to Nicu steel, claiming that in this direct smelting process the copper, up to a certain proportion, may be considered as taking the place of an equal amount of nickel, and that not only is the presence of copper not detrimental, but that, on the contrary, it may be advantageous in that it produces,—owing to certain qualities of the copper,—a superior product which can be manufactured at much less cost than nickel steel made by the ordinary practice of alloying refined nickel with steel in certain definite proportions.

During the past year Mr. Colvocoresses, with some associates, has experimented in the manufacture of nickel-copper steel direct from the Sudbury ores, for which purpose about 200 tons of ore and 40 tons of slag were obtained from the Sudbury District and experiments were carried on at the plant of the Canada Cement Company, at East Montreal.

This ore was roasted in a hand-rabbled furnace and smelted to pig in an electric furnace of the Heroult type, and some of it was afterwards refined into steel in the same type of furnace, and the balance in an open-hearth furnace using producer gas.

The experiments at Montreal were under the direct supervision of Mr. H. A. Morin, who had previously been associated with Mr. Colvocoresses in the smelting of Sudbury ores, and I think I cannot do better than quote substantially and at some length from Mr. Morin's report on the result of these experiments.

In his report, dated December 7th, 1917, Mr. Morin explains that the experiments consisted in desulphurizing iron-nickel-copper sulphide ores mined in the Sudbury-Ontario District for the purpose of smelting and reducing these ores, with suitable fluxes, and producing an iron-nickel-copper pig of homogenous composition which, preferably in its molten state, could be refined to a nickel-copper steel, with or without foreign ferrous addition, according to the grade of Nicu Steel desired.

Another experiment was made in the smelting of blast furnace slag, which slags are produced in large quantities in the smelting of the Sudbury ores (partially roasted) in a blast matting furnace. While these slags will produce a pig low in nickel and copper, it is a simple matter to increase the nickel-copper content by the proper addition of roasted nickel-copper ore.

The following is an average analysis of such ores and slags:

	Nickel-Copper Ore	Blast Furnace Slag
Iron	40–50%	40–45 %
Nickel	3–4 %	35–5 %
Copper	1–1½%	25–35 %
Sulphur	25–30%	1¼–2½%
Silica	12–20%	20–30 %
Alumina	3–4 %	6–7 %
Lime	2–3 %	2–3 %
Magnesia	1–2 %	1–2 %

Balance Oxygen.

These experiments were carried on in accordance with the description given in the Patent papers issued and granted, both in Canada and the United States, to Mr. G. M. Colvocoresses.

ORE SUPPLY

According to the Royal Ontario Nickel Commission Report, published in April 1917, a total of 75,000,000 tons of ore had been developed by the Operating Companies up to that time, and the Report further states that out of 110 miles of nickel-bearing formation, only about ten miles have been developed, partially, by diamond drill, and that consequently it is fair to assume that this ten mils of partly developed formation is capable of further extending the ore bodies within this area.

The ore secured for these experiments, amounting to about 200 tons, was obtained from the Algoma Steel Corporation and was of rather low grade. It theoretically should have produced a 3 to 3½% nickel-copper steel, but in actual operation the nickel-copper pig was considerably diluted, by reason of the fact that the electric furnaces used had built-up banks and bottoms of iron which had formed during the previous operation of the furnaces in the production of pig from scrap.

The ore obtained for treatment was mined from one of the properties of the Algoma Steel Corporation about 14 years ago and, having been exposed to the air and weather during all that time, was decomposed and the nickel-copper-sulphur content was considerably leached out. The following is a close approximation of the composition of the ore when it was first mined and as it is to-day:

	Freshly Mined Ore	Ore Received
Iron	45%	46%
Nickel	2.9%	1.35%
Copper	75%	25%
Silica	17%	19%
Sulphur	30%	8%

SMELTING COPPER CLIFF SLAG TO PIG

The slag was smelted in the same manner as the roasted ore, having a very similar composition and therefore requiring but a slight variation in the proportion of the fluxes. It is particularly interesting to note that the slags used in this experiment contained 2.2% sulphur, and after smelting this slag in an electric furnace the resultant pig contained .065% sulphur.

Mr. Morin reports that the conversion of the pig to steel in the open-hearth furnace proved entirely satisfactory, the operation being identical with the production of steel from ordinary pig iron. About 70 tons of 2½% Nicu steel were made.

Since the experiments at Montreal the Nicu Steel Corporation has produced an additional 60 to 70 tons of Nicu pig and steel containing approximately 3½% combined nickel and copper, at the plant of Electric Steel & Engineering Ltd., Welland, Ont., considerable of which has been brought into actual use in various industries, the desired changes in the composition of the steel having been made for the various purposes for which it was required.

The ore was smelted in electric furnaces, after a preliminary roasting, and the physical properties in every case have proven very satisfactory when compared with the properties of nickel steel, as shewn by the following report furnished by Mr. Morin:—

Engineering Standard Committee E.S.C. 3% Nickel-Chrome Steel.		NICU STEEL Independent test by Dept. of Mines, Ottawa.
Carbon	0.30%	0.28%
Silicon	0.30%	0.014%
Manganese	0.60%	0.58%
Sulphur	0.04%	0.038%
Phosphorus	0.04%	0.005%
Nickel	2.75 to 3.50%	2.16 to 2.62%
Chromium	0.45 to 0.75%	Copper..........0.46%

Physical properties of above steel when oil hardened from 820°C and tempered at 600°C.		Physical properties of above steel when heated to 800°C, quenched in water and drawn back at 400°C.	
Tensile strength	100,800 lbs.	Tensile strength	101,200 lbs.
Yield ratio	75,600 "	Yield ratio	78,900 "
Elongation	15%	Elongation	21.5%
Reduction	50%	Reduction	58.8%

Chemical and Physical properties of Nicu Steel as compared with Nickel Steel

American Society for Testing Material, O.H. Nickel Steel Specification.		Nicu Steel, Independent test by Royal Ontario Nickel Comm., Page 415 of Report.
Carbon	0.46%	0.43%
Silicon	0.066	0.30
Manganese	0.70%	0.47%
Phosphorus	0.021%	0.05
Sulphur	0.034	0.05
Nickel	3.36	Nickel..........2.10 } —3.30%
		Copper..........1.20 }

Physical properties of above steel, rolled natural.		Physical properties of above steel, rolled natural.	
Tensile strength	122,000	Tensile strength	110,400
Elastic Limit	74,625	Elastic limit	82,600
Elongation	16%	Elongation	22%
Reduction	34%	Reduction	48%

" Nicu Steel users comprise the automobile trade, makers of different kinds of mining machinery in different forms, such as pistons for pneumatic tools, rock drills, etc. A considerable tonnage will, no doubt, enter the market as castings, crusher parts, ball mill liners, balls, and many other uses of high-grade steel, where strength, toughness and durability are required, and in general competition with nickel steel and other simila alloys."

Analysis and Tests of 'Nicu' Steel Specimens:

	'Nicu' Steel A	'Nicu' Steel B
Carbon	0.32%	0.36%
Silicon	0.028%	0.029%
Sulphur	0.045%	0.035%
Phosphorus	0.079%	0.003%
Manganese	0.83%	0.73%
Nickel	1.79%	1.73%
Copper	0.27%	0.31%

Soft test of above steel after forging:

Elastic Limit, sq. in	66,800 lb.	68,320 lb.
Tensile Strength, sq. in	90,720 lb.	91,840 lb.
Elongation on 2 inches	29%	29%
Reduction of Area	0.190	0.190
Description of Fracture	Cupped Silky	Cupped Silky

Hard test after the following heat treatment:

Heated to 1800 deg. F. Quenched in Oil. Re-heated to 700 deg. F., and cooled slowly:

Elastic Limit, sq. in	92,960 lb.	89,600 lb.
Tensile Strength	113,120 lb.	110,880 lb.
Elongation on 2 in	22%	22%
Reduction of Area	0.150	0.155
Description of Fracture	½ Cupped Silky	½ Cupped Silky
Cold Bending Test	Showing no crack	Showing no crack

The above Cold Bending Tests were made on Specimen 4.5 in. 0.75 in. x 0.375 in. Bent cold, through 180 deg. with a Presser of 1.5 in. diameter.

Physical Tests and Analyses of Nicu Steel Heat No. 6, made by Dr. Alfred Stansfield, M.E.I.C., McGill University; and comparison with Nickel Steel as per Specification for Plates and Shapes, Ontario Nickel Commission Report, Page 365.

	Natural Nicu Steel	Natural Nickel Steel
Carbon	0.37%	0.45%
Manganese	0.88%	0.70%
Phosphorus	0.064%	0.04%
Sulphur	0.047%	0.04%
Nickel	1.89% } 2.26%	Nickel 3.25%
Copper	0.37% }	
Tensile Stress lbs. per sq. in.	96,300	85,000 to 100,000
Yield Point lbs. per sq. in	56,350	50,000
Elongation on 9 in. %	18.7%	16.2%
Reduction of Area%	36.3%	25.0%

Nickel Steel Specification in connection with the fabrication of the large bridge to span the Mississippi River at Memphis:

Tensile Strength..................		85,000 to 100,000
Elastic limit not less than...........		50,000
Elongation in 8 in. not less than....	$\frac{1,600,000}{T.S.}$	Average 17%
Reduction of Area not less than.....		30%

D

Nicu Steel produced commercially at the Canada Cement Company's Steel plant, East Montreal, and tested by Dr. Alfred Stansfield, M.E.I.C., —in comparison with Nickel Steel of similar composition, as given in tabulated form on Page 387 Marked (C) and Page 416 marked (1) Royal Ontario Nickel Commission Report.

	Nicu Steel Heat No. 6	Nickel Steel (C) Page 387	Nickel Steel (1) Page 416
	%	%	%
Carbon.................	0.37	0.47	0.47
Manganese..............	0.88	0.86	....
Nickel..................	1.89 } 2.26	2.15	2.92
Copper.................	0.37 }	....	.....
	lbs.	lbs.	lbs.
Yield Point.............	52,800	52,000	56,000
Tensile Stress...........	96,500	93,000	95,400
Elongation on 2 in.......	24.3%	24.5%	22%
Reduction of area........	50.8%	51.8%	44.6%
Bending Tests 180°......	Shewing no crack	Shewing no crack	....

E

Results of the Royal Ontario Nickel Commission, Page 415, Table 3, obtained with Nicu Steel and Nickel Steel produced under exactly the same conditions during their investigation of the Colvocoresses process.

	Nicu Steel Heat No. 2	Nickel Steel Heat No. 4	Nicu Steel Heat No. 6
Carbon.................	0.43%	0.53%	0.53%
Nickel..................	2.10% } 3.30%	3.43%	2.45% } 3.25%
Copper.................	1.20% }		0.80% }
	lbs.	lbs.	lbs.
Elastic Limit............	82,600	77,400	80,000
Tensile Strength.........	110,400	115,400	111,600
Elongation % on 2 in....	22%	20%	19.1%
Reduction of area%......	48%	38.3%	38.3%

These steels were produced under the direct supervision of Geo. A. Guess, Professor of Metallurgy, University of Toronto, in conjunction with the Royal Ontario Nickel Commission.

Extracts from Prof. Guess's report:

"It is evident from the results shown in Table 3 that these laboratory "made steels are of good quality."

"The value of this process of producing Nickel Copper steel is based "on the belief that Copper may replace a very considerable amount "of the Nickel in a 3.5% Nickel steel without producing an inferior article, "which belief is, I think, well founded."

F

Royal Ontario Nickel Commission Report, Page 421, Comparison of Nicu Steel with Nickel Steel.

	Nicu Steel		Nickel Steel	
Carbon	0.44%		0.46%	
Silicon	0.034%		0.066%	
Manganese	0.50%		0.70%	
Phosphorus	0.013%		0.021%	
Sulphur	0.013%		0.034%	
Nickel	3.62%	4.10%	3.36%	3.46%
Copper	0.48%		0.10%	

The Physical Tests of the rolled natural steels showed:

Elastic Limit	72,400	74,626
Ultimate strength	115,000	122,000
Elongation in 2 in.	22%	16%
Reduction of Area	51%	34%

In the annealed condition the results were:

Elastic Limit	63,750	64,750
Ultimate strength	107,300	119,000
Elongation in 2 in.	25%	17%
Reduction of Area	48%	37.5%

Mr. Colvocoresses, from his experiments, has arrived at the conclusion that the best results are obtained when the ratio of copper to nickel is as 1 to 3 or 1 to 4 and that the total copper content should not exceed 1% if the copper is to be considered as replacing an equal percentage of nickel and the steel produced is to be put to the ordinary uses for which nickel steel is employed. The greater part of the nickel steel produced to-day contains about 3% of nickel, and it has been found that nickel-copper steel containing 2½ of nickel and .75% of copper is similar and equal to a straight 3% nickel steel.

Nickel-copper steel possesses qualities, however, which give it the advantage over straight nickel steel, namely, a greater uniformity in composition and a decreased liability to corrosion owing to the presence of the copper. This latter quality, when fully demonstrated, should give Nicu steel the preference over ordinary nickel steel for ship plates and machinery parts where such are subject to the action of acids, salt water and other corrosive agencies. Experiments made by Clamer to discover the effects of these destructive agencies on nickel-copper steel were highly favorable, and it has been known for a long time that a small addition of copper to ordinary steel is used by steel manufacturers in making special kinds of non-corrosive steels for use in locations where there is danger from corrosion.

The following extract is quoted from an article on "Corrosion-Proof Steel," by F. Rowlinson, which appeared in the "Scientific American," of August 16th, 1919:—

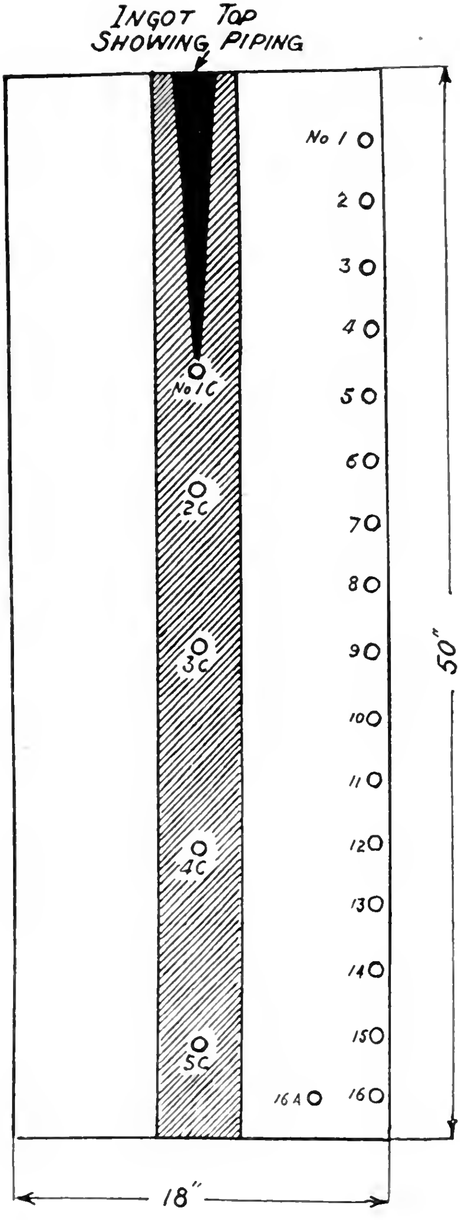
NICKEL-COPPER STEELS

Since it was found that both nickel and copper gave immunity from corrosion when added to steel, it was expected that a steel containing both would also be comparatively free from corrosion. This was found to be the case. But an additional feature was found to be present in the more complex steel so made. Providing the proportion of nickel to copper was never less than five to two, the copper does not segregate as it tends to do in carbon steel. In this way the copper content, and, therefore, the powers of resistance to corrosion may be very considerably increased. Moreover, as regards mechanical tests, the copper seems to replace perfectly an equal percentage of nickel, so that a nickel-copper steel with, say 2½ per cent nickel and 1 per cent copper has the mechanical properties of a 3½ per cent nickel steel, with increased immunity from corrosion.

Of course, it is too soon to say yet how Nicu steel will stand up under the variety of uses to which nickel steel is put, such as in the manufacture of armour plate, steel rails, etc., but the tests and experiments which have already been carried out give every promise of a highly satisfactory product for these purposes, and it is confidently expected that when the many advantages of nickel-copper steel are fully realized, and the existing prejudice against the presence of a small percentage of copper in steel has been removed, a large demand will develop for this product.

The question of costs is an important one, and as Nicu steel has not, as yet been manufactured on a commercial scale, estimates of cost must be approximate; but Mr. Colvocoresses estimates—and his figures would appear to make due allowance for the several operations from the mining of the ore to the production of the finished steel—that Nicu Steel can be produced from ore at a cost of about $20.00 per ton over the cost of ordinary carbon steel.

Some fears have been expressed that segregation of the nickel or copper might take place, thus destroying the uniformity of the product; but the accompanying sketch shewing one half of an 18″ octagonal ingot, which was supplied for the purpose of testing and thereafter drilled and analysed, indicates clearly that these fears are groundless.


INGOT TOP
SHOWING PIPING
No 1 O
2 O
3 O
4 O
5 O
6 O
7 O
8 O
9 O
10 O
11 O
12 O
13 O
14 O
15 O
16 O
16 A O
No 1 C
2 C
3 C
4 C
5 C
50"
18"

The following partial report of T. W. Hardy and John Blizard, with diagrams and photomicrographs, is the result of an investigation by the Mines Branch of the Dep't of Mines, Ottawa, at the request of the Imperial Munitions Resources Commission, of the properties of certain samples of Nicu Steel made by the Nicu Steel Corporation:

"The advantages obtained by the addition of nickel to steel have been well known for many years and nickel steel is one of the most common of the alloy steels in use to-day. Nickel dissolves in iron in all proportions, but the steels of the greatest commercial importance contain less than 5% nickel and not more than 1% carbon. Steels coming within these limits are, like plain carbon steel, pearlitic on slow cooling. Notwithstanding this structural similarity, however, the physical properties of pearlitic nickel steels are considerably superior to those of the corresponding carbon steels. The addition of nickel to plain carbon steel in the production of pearlitic nickel steel results in a considerable increase in strength, particularly in elastic limit, while the ductility is decreased but little, if at all. To develop fully the high physical properties which they are capable of possessing, nickel steels must be heat-treated, and in this condition their superiority over similarly treated carbon steels is more apparent than when neither the nickel nor the carbon steels are heat-treated. Heat-treated nickel steel is not only stronger, but is tougher and has greater resistance to wear and shocks than a similarly treated carbon steel."

"In the majority of the nickel steels manufactured to-day, the nickel content does not exceed 3½% and the carbon content is generally less than 0.50%. Steels of this class find a wide application in structures where a high combination of strength and ductility combined with a saving in weight are essential factors. Among the more common applications of these nickel steels may be mentioned rails and bridge members, for which they are used without heat treatment; other perhaps more common applications are automobile and engine parts, machine parts and gun forgings, for which purposes the steel is practically always heat-treated."

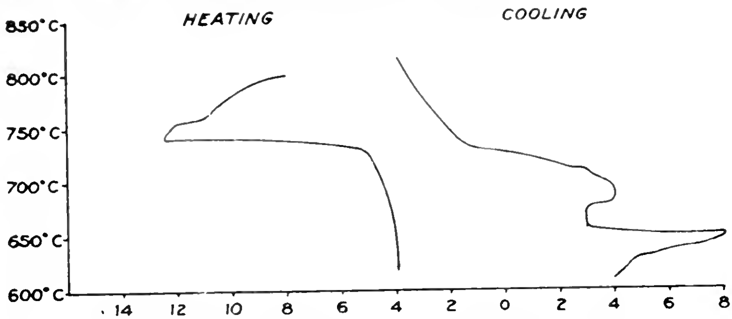
" THE INFLUENCE OF COPPER ON THE PROPERTIES OF STEEL "

"The effect of copper on the properties of steel has been the subject of a great deal of discussion. For a long time copper, even in very small amounts, was supposed to cause red-shortness, and this view is still held by some. Of later years, the researches of several able investigators have proved that copper, within certain limits, is not only harmless, but really improves the properties of the steel. As a result of these researches it may be conservatively stated that in proportions not exceeding 0.75% copper does not make steel red-short provided the sulphur content is not high. There seems to be no doubt that an amount of sulphur, say 0.08% to 0.1%, that would not cause serious trouble in steel free from copper, would, in conjuction with a few tenths of one per cent of copper, produce serious red-shortness."

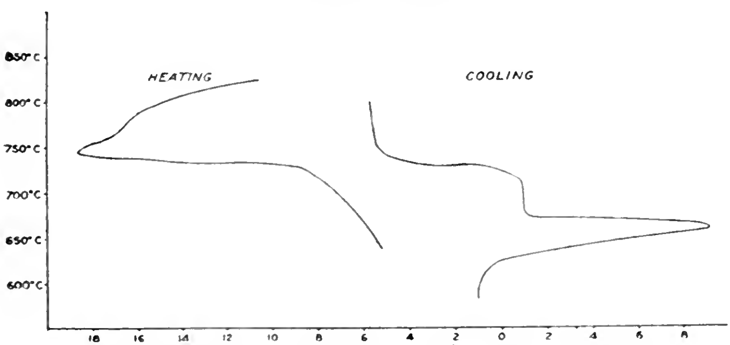
"The results of these investigations also show that the presence of "small amounts of copper makes steel more resistant to corrosion. They "also show that the general effect of copper, in small amounts, on the tensile properties of the steel, is to increase the tensile strength and yield point, "and to decrease the elongation and reduction of area."

It seems probable that the red-shortness and segregation that sometimes accompany copper when present in steel in an amount exceeding 1%, have their ultimate explanation in the fact that copper is but slightly soluble in iron. Since nickel unites readily with both copper and iron, it is reasonable to assume that the presence of nickel will permit larger amounts of copper to be used without introducing the above defects. This effect of nickel in increasing the solubility of copper in steel, together with the fact that copper resembles nickel in its effect on the cold properties of the steel, lends weight to the probability that copper may replace a considerable proportion of the nickel in a nickel steel without materially altering its hot or cold properties."

Acknowledgments are due to Mr. H. S. Foote, who prepared and photographed the microsections; to Messrs H. A. Leverin and F. W. Baridon, who determined the chemical composition of the steel; and to Mr. E. S. Malloch, who assisted in the tensile tests.



TEMPERATURE DIFFERENCE - DIFFERENTIAL CURVES
HEAT NO. 4.



TEMPERATURE DIFFERENCE - DIFFERENTIAL CURVES
HEAT NO 10

NICU STEEL TESTS

HEAT NO. 4

CHEMICAL ANALYSES

Analysis by	Bar	Carbon	Phosphorus	Manganese	Sulphur	Silicon	Nickel	Copper	Cobalt
Nicu Steel Co...	..	0.28%	0.005%	0.58 %	0.038%	0.014%	2.16%	0.41 %	..
Mines Branch..	A	0.262	0.012%	0.522%	0.061%	0.009%	1.98%	0.470%	0.16%
do	B	0.267	0.006%	0.532%	0.036%	0.006%	1.96%	0.450%	..

SERIES A

PIECES from BAR A, Quenched in Water from 800°C, and Drawn to Various Temperatures

HEAT TREATMENT and PHYSICAL PROPERTIES

Test No.	Heat Treatment			Physical Properties			
	Heated to	Quenched in	Drawing Temp.	Yield Point	Ultimate Strength	Elongation (in 2″)	Reduction in Area
1	800°C	Water	350°C	77,400 lbs./sq. in.	104,400 lbs./sq.in.	23.5%	64.0%
2	"	"	400°C	78,900 "	101,200 "	21.5%	58.8%
3	"	"	450°C	76,200 "	99,400 "	25.0%	64.0%
4	"	"	500°C	74,400 "	95,800 "	27.0%	66.1%
5	"	"	550°C	72,100 "	91,000 "	29.0%	66.2%
6	"	"	600°C	67,500 "	86,400 "	29.5%	70.8%
7	"	Cooled Freely in Air.		56,000 "	79,000 "	30.0%	59.0%
8	"	Cooled Slowly in Furnace.		51,000 "	72,000 "	31.0%	53.0%
9	As Rolled			47,400 "	76,700 "	33.0%	59.0%

SERIES B

PIECES from BAR B, Quenched in Oil from 800°C, and Drawn to Various Temperatures

HEAT TREATMENT and PHYSICAL PROPERTIES

Test No.	Heat Treatment			Physical Properties			
	Heated to	Quenched in	Drawing Temp.	Yield Point	Ultimate Strength	Elongation (in 2″)	Reduction in Area
1	800°C	Oil	350°C	68,000 lbs./sq. in.	91,000 lbs./sq. in.	20.0%	66.3%
2	"	"	400°C	67,700 "	90,300 "	29.5%	66.3%
3	"	"	450°C	66,200 "	88,700 "	31.5%	68.5%
4	"	"	500°C	65,000 "	87,100 "	31.5%	66.3%
5	"	"	550°C	62,600 "	85,900 "	32.0%	66.2%
6	"	"	600°C	61,900 "	83,000 "	35.0%	70.5%
7	"	Colled Freely in Air.		57,500 "	77,900 "	34.0%	61.5%
8	"	Cooled Slowly in Furnace.		54,000 "	73,100 "	31.0%	59.0%
9	As Rolled			47,800 "	73,300 "	34.0%	58.8%

NOTES:

1. All heat treatments made on the full section (1″ Round), and testpieces (A.S.T.M. Standard 2 inch with threaded ends) machined from the heat treated stock.
2. This heat was made from Sudbury ore.

NICU STEEL TESTS

HEAT No. 10

CHEMICAL ANALYSES

Analysis by	Bar	Carbon	Phosphorus	Manganese	Sulphur	Silicon	Nickel	Copper	Cobalt
Nicu Steel Co..	..	0.34 %	0.017%	0.63 %	0.041%	0.019%	1.33%	0.46 %	..
Mines Branch..	A	0.355%	0.025%	0.547%	0.043%	0.017%	1.31%	0.570%	0.13%
do	B	0.347%	0.024%	0.554%	0.050%	0.023%	1.30%	0.572%	..

SERIES A

PIECES from BAR A, Quenched in Water from 800°C, and Drawn to Various Temperatures

HEAT TREATMENT and PHYSICAL PROPERTIES

Piece No.	Heat Treatment			Physical Properties			
	Heated to	Quenched in	Drawing Temp.	Yield Point	Ultimate Strength	Elongation (in 2″)	Reduction in Area
1	800°C	Water	350°C	87,600 lbs./sq. in.	123,000 lbs./sq.in.	15.5%	51.0%
2	"	"	400°C	83,400 "	118,000 "	18.5%	53.8%
3	"	"	460°C	83,600 "	115,000 "	19.5%	56.5%
4	"	"	500°C	84,000 "	112,500 "	22.0%	61.6%
5	"	"	550°C	79,000 "	106,000 "	23.0%	61.6%
6	"	"	600°C	74,400 "	102,800 "	25.0%	64.0%
7	"	Cooled Freely in Air.		52,000 "	81,000 "	30.5%	59.1%
8	"	Cooled Slowly in Furnace.		48,400 "	76,400 "	32.0%	53.8%
9	As Rolled			53,500 "	84,600 "	32.0%	59.1%

SERIES B

PIECES from BAR B, Quenched in Oil from 800°C, and Drawn to Various Temperatures

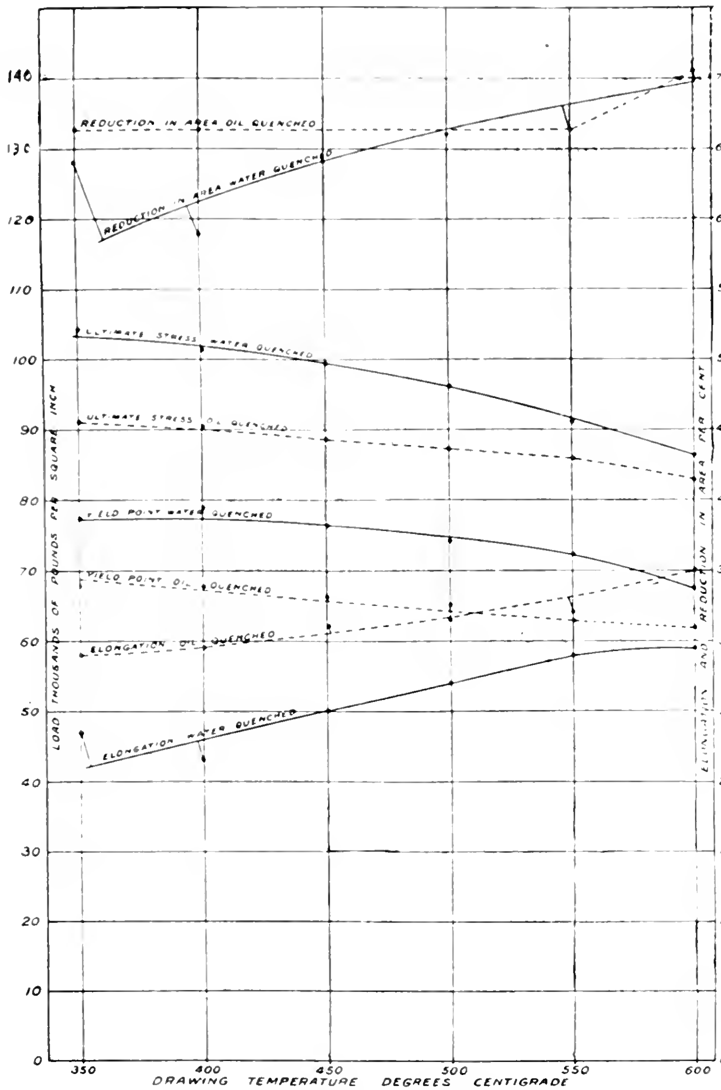
HEAT TREATMENT AND PHYSICAL PROPERTIES

Piece No.	Heat Treatment			Physical Properties			
	Heated to	Quenched in	Drawing Temp.	Yield Point	Ultimate Strength	Elongation (in 2″)	Reduction in Area
1	800°C	Oil	350°C	62,200 lbs./sq. in.	98,300 lbs./sq. in.	25.5%	61.6%
2	"	"	400°C	65,200 "	99,400 "	24.0%	61.6%
3	"	"	460°C	64,700 "	99,400 "	26.0%	61.6%
4	"	"	500°C	72,900 "	99,800 "	25.5%	61.6%
5	"	"	550°C	65,700 "	94,200 "	27.0%	64.0%
6	"	"	600°C	66,200 "	94,200 "	29.0%	64.0%
7	"	Cooled Freely in Air.		53,000 "	81,500 "	29.0%	59.1%
8	"	Cooled slowly in Furnace		40,700 "	76,400 "	31.5%	59.1%
9	As Rolled.			53,500 "	82,000 "	29.5%	56.5%

NOTES:—

1. All heat treatments made on the full section (1″ Rounds). Testpieces (A.S.T.M. Standard with threaded ends), were then machined from the heat treated stock.

2. This heat was made from Sudbury slag.



NICU STEEL
HEAT No. 4
TENSILE TESTS
ON
A WATER QUENCHED FROM 800°C
B OIL " " "

$$\frac{dMm}{dx} = w\ (m-x\ -852+E_1$$

5. For Figure No. 1

   Refer to Fig. No. 8,

$$Mm = w\ \frac{m^2}{2} + Ee + Ex - w\ \frac{e^2}{2} + w\ e\ x$$

$$= w\ \frac{m^2}{2} + Ee + (E - w\ e)\ x - w\ \frac{e^2}{2}$$

$$= w\ \frac{m^2}{2} + 282\ x + 16602$$

$$\frac{dMm}{dx} = 282$$

6. For Figure No. 1

   Refer to Figure No. 9,

$$Mm = w\ \frac{m^2}{2} - w\ \frac{x^2}{2} + E_1e_1$$

$$\frac{dMm}{dx} = E_1 - w\ x$$

7. Moment table for locomotives, page 276.

   A moment table of the wheel concentrations of the locomotives was used from which the load E_1 and the moment E_1e_1 of Figures 7 and 9 could be easily obtained.

Maximum Stress in any member. The position of the locomotive giving the maximum stress in any member was found by means of an equation obtained by differentiating the equation for the stress in that member. The first differential is equal to zero and contains only expressions representing the loads and the coefficients for the moments, thus giving an equation which was very easily solved.

As an example refer to the calculations for the stress in the member M_4F_4, which follows, and to figures 5, 8 and 9.

The first differential of the equation for the stress in this member is:—

$$.4357 \times 282 - 58.333\ (E_B - wx) + 22.688\ (E_F - wx)$$
$$-2.3090\ (E_G - wx) + 1,0627\ (E_K - wx) + .7484 + 282 = 0.$$

where E_B, E_F, E_G, and E_K represent the weight of the locomotives to the left of panel points 4, 3, 2 and 1 respectively, x the distance from the point of moments to the uniform load preceding the locomotives and w = 5000 lbs. per lineal foot of track.

To obtain the condition for the maximum stress in the member it is then only necessary to find a position of the locomotives such that the sum of the positive factors of the equation falls between the sums of the negative factors, obtained by first considering E_B as the weight of the locomotive to the left of the panel point 4 and second by considering that the weight E_B includes the weight of the wheel concentrated at that point.

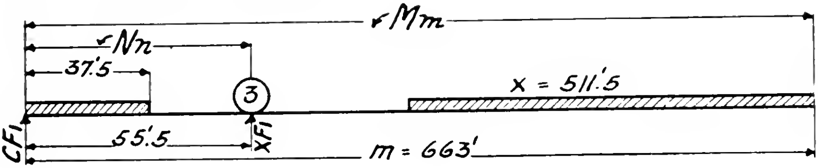
Stresses,—The full calculations for the stress in the members U_0L_0 M_2L_2, M_4F_4 and the Hangers H_F (see Fig. 1) follow and will serve to illustrate the method followed for calculating the stresses in all the members of the truss.

The clause in the specification stating that the locomotives were to be followed or preceded or followed and preceded by the uniform load made it necessary to figure all the stresses twice, first with the locomotives headed towards the main pier, as herein, and second with the locomotives headed towards the center of the span, the maximum of these two conditions being taken as the max'mum live load stress in the member. The difference in the stresses thus obtained was found to be very slight and would hardly justify the additional amount of labor involved.

STRESS IN U_0L_0

(See Figure No. 3.)

$U_0L_0 = 1.5625\ Mm - 18.666\ Nn$



CRITERION:

(See Figures Nos. 5 and 9.)

$1.5625 \times 282 = \underline{\underline{441.}}$

$$\begin{array}{ll} 90 & 150 \\ \underline{90} & \underline{90} \\ (\ 0, & 60) \times 19.06 = \underline{\underline{0,}}\ \ \underline{\underline{1140.}} \end{array}$$

STRESS

$$w\frac{m^2}{2} = 1098920$$

$$282x = 144240$$

$$\underline{18470}$$

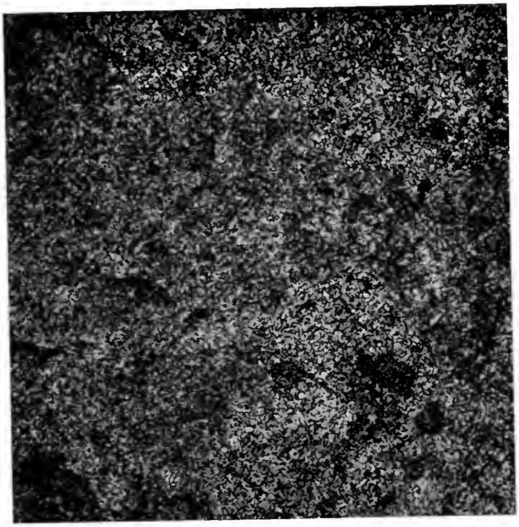
$$Mm \quad = 1261630 \times 1.5625 = 1971.2$$

$$690$$

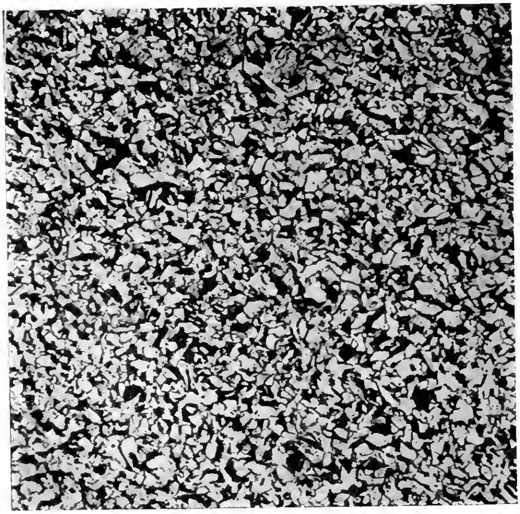
$$\underline{6890}$$

$$Nn \quad 7580 \times 18.666 = \underline{141.5}$$

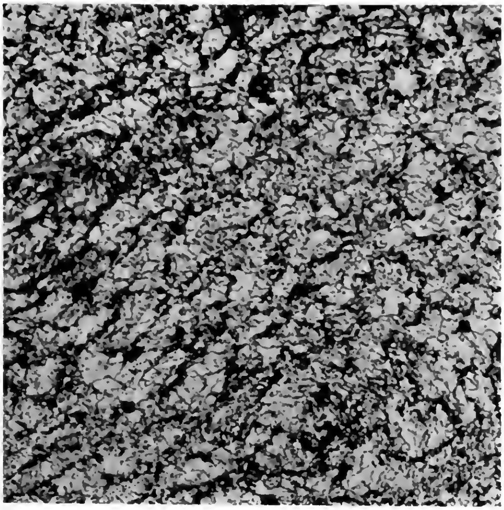
$$1829.7 = U_0L_0$$



Quenched in water from 800°C. Drawn to 600°C.



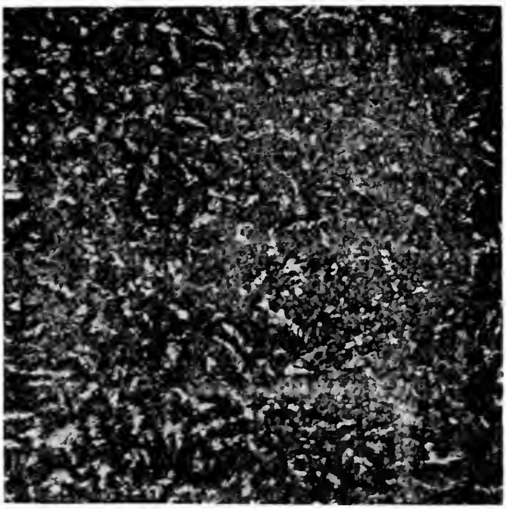
As Rolled.



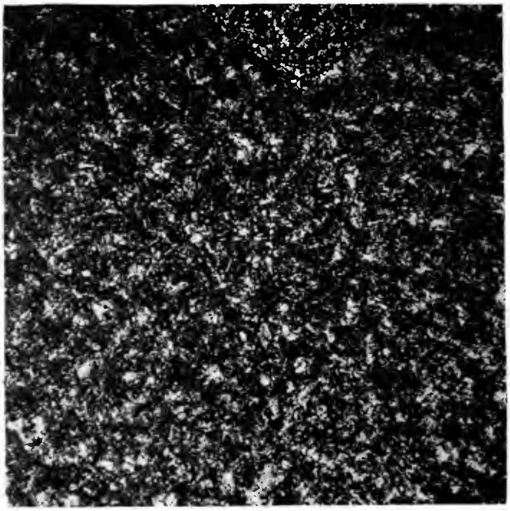
Annealed at 800°C.

PHOTOMICROGRAPHS. HEAT No. 10.

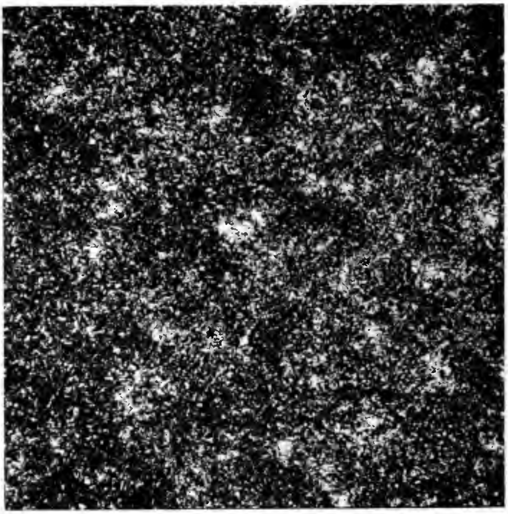
MAGNIFICATION 75 DIAMETERS



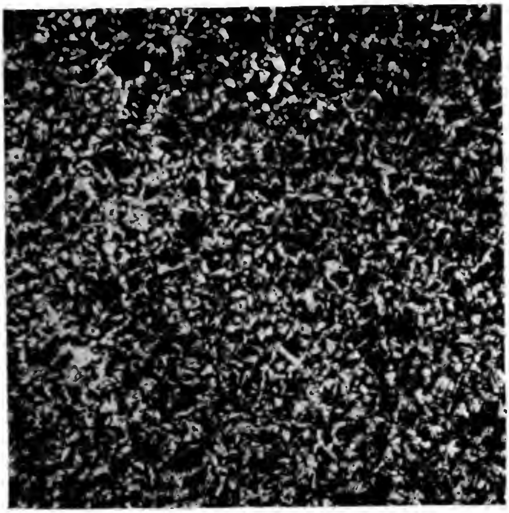
Quenched in water from 800°C. Drawn to 400°C.



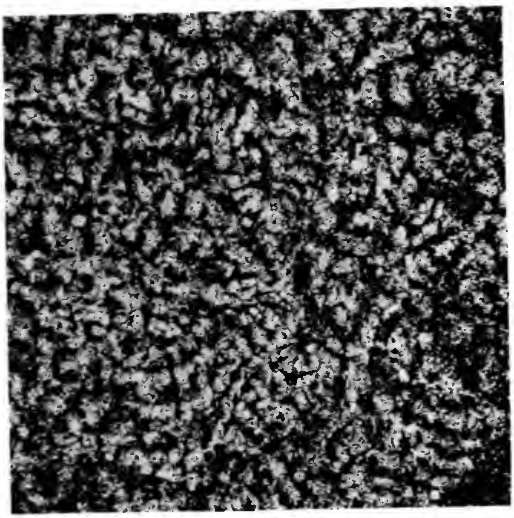
Quenched in water from 800°C. Drawn to 500°C.



Quenched in water from 800°C. Drawn to 600°C.



As Rolled



Annealed at 800°C.

Capt. Jamieson

Capt. E. A. Jamieson, A.M.E.I.C.—This paper is one that should interest every engineer at this time, and if this process be carried out on a large commercial scale, it means not only a rejuvenation of our steel industry, but means the greatest boost it has yet received.

This process means not only the reclaiming of the 8,000,000 tons of iron said to be held in the slag dumps in the Sudbury District, but means the production of a steel of excellent qualities. The presence of copper in the Nicu Steel is not too high. The copper in steel used to be considered to be very harmful, but now it is known that when it is present in small quantities, it has no serious influence on the physical properties of steel.

According to M. A. L. Colby (Stahl and Eisen, 1900, pp. 20.55), neither a propeller shaft with .565 per cent copper nor a gun tube with .553 per cent showed any defects after forging and hardening, and armour plates with .575 per cent flanged well and answered all the requirements of the American Navy. Various samples of Bessemer steel with carbon ranging from .11 to .65 per cent and with .292 to .486 per cent copper showed no red shortness on rolling.

A steel containing a high percentage of copper, although not red short in the ordinary sense, yet will not bear the same amount of heat as the same metal without copper, thus showing that the effect of the latter resembles that of carbon, rather than that of sulphur.

After making various physical tests on cuperous samples, the investigators concluded that copper does not possess the property of making steel red short.

C. F. Burgess and J. Ashton (Electrochemical and Metallurgical Industry, 1909, vol. VII, pp. 527-529), have also examined the properties of iron-copper alloys, and found that the alloys up to two per cent copper, forge well at red heats. Those from 2 to 7 per cent copper will not forge at a low temperature, and rather poorly at a white heat, the case of workability varying inversely as the percentage of copper. The same authors (Ibid, 1910, vol. VIII, pp. 23-26), on examining the magnetic and electrical properties of these alloys found that above 2 per cent copper the permeability falls off rather rapidly as the copper content is increased, but only bars of 4 per cent and more are conspicuously poor. Annealing greatly improves the quality of the bars.

From Col. Leonard's paper, on page 9, the chemical analyses show the copper constituent to be well below the figures given by Mr. Colby. As regards the nickel constituent, its percentage is somewhat lower than that required in the manufacture of gun steel but as this system appears not to take into consideration the addition of any elements other than those already contained in the ore or slags used, the increase in the per cent of nickel can be artificially carried out. The physical tests appear very satisfactory and must be very gratifying to the makers of the Nicu steel.

Last year, it was my privilege to visit a number of the large steel producing corporations of Great Britain, and everywhere the same question was asked, "What are you doing in Canada to improve the steel stiuation?" Steel is required in larger quantities and of a better quality than the world has ever demanded before, and if we as Canadian engineers, know how to increase this production of steel, it is our duty to see that it is carried out. Why should Canada not compete in the great call for armament? Previous to the War, Great Britain and Germany supplied the largest percentage of the World's armament, and what has it done for Great Britain? Huge industries can be built here in Canada, as well as over there. Capt. Jamieson

Do the engineers or general public realize what it would mean to our Empire should the Huns succeed in their efforts to bomb the large steel plants in England.

I was present during raids of the Huns and saw them make efforts to reach the steel plants in the North of England. An extremely high wind was all that saved them one night, and judging from the damage caused by one Zeppelin bomb, a steel plant would be wiped out in no time and past all hopes of repair, as several tons of molten steel let loose are not as snow, regards removal.

It would be the greatest national catastrophe that could befall us and would we in Canada, as a part of our Great Empire, not be sadly lacking in our duty as such, if we were not ready to step in to help fill the breech. The production of high grade steel in large quantities, in Canada is a national necessity at present and in the future post war days, it would enable us to compete with the world.

From the viewpoint of armament, which at this time is before us all, why should Canada not be independent and manufacture her own armament? Col. Leonard shows us that we have the raw material and that this system can produce high grade steel. If the Government of our country does not see the necessity of encouraging the increased production of steel, then we as members of the Canadian Society of Civil Engineers are not doing our duty if we do not appeal to them to see as we do and urge upon them the necessity, if necessary, of subsidizing corporations to enable Canada to take her place in the front line with the great steel producing countries together with their allied industries.

Mr. G. C. Mackenzie, (Sec., Munitions Resources Commission, Ottawa.)—I have been very much interested in Col. Leonard's paper as it reminds me of a discussion I had some years ago with Mr. R. W. Seelye, late manager of the Mines Department, of the Algoma Steel Corporation. Mr. Seelye had an idea that some use could be made of the large slag dumps at Copper Cliff and Victoria Mines, and suggested that Mr. Mackenzie

Mr. Mackenzie the iron content might be separated by means of electro magnets. This, of course, was out of the question because the iron exists in these slags in the form of a silicate, chemically combined. Subsequently, I proposed the smelting of these slags to the blast furnace superintendent who was then in charge of the Soo furnaces, but he was not at all enthusiastic over making use of this material which he claimed, quite rightly, was very similar to mill-cinder, a slag product of puddling furnaces, which is sometimes used in the manufacture of pig iron.

Mill-cinder, as every iron blast furnace man knows, is a most objectionable material to smelt because its fusion point being low it melts high up in the blast furnace stack and runs ahead of the charge in the form of unreduced iron silicate with ill affect upon the furnace lining and also upon the quality of the pig iron produced. On the other hand, oxides, such as hematite, limonite and magnetite are gradually reduced and carbonized in their descent to the tuyeres where the iron is finally brought to the fusion point and falls to the hearth.

I have myself smelted a considerable tonnage of mill-cinder and have a lively recollection of its behavior; and while I have never heard of a furnace burdened entirely with this material I would anticipate considerable difficulty in smelting in a blast furnace the Sudbury slags which are similar in physical condition to a mill-cinder.

A mixture of roasted copper-nickel ore and slag might be easier to work than the slag alone; but I would anticipate that fuel consumption would be relatively high, possibly over 3,000 pounds of coke per ton of copper-nickel pig on the supposition that a furnace operating with this material would require to be run very hot and with a very basic slag. It might be possible to carry out experiments in one of the idle blast furnace stacks at Parry Sound or Midland, Ontario. Such experiments would, of course, be costly but would I think demonstrate the practicability or otherwise of manufacturing Nicu pig from a mixture of roasted Sudbury ores and Sudbury slags.

In so far as the electric reduction of these materials is concerned I would think that electric power should be secured for not more than $10.00 per horse-power year, if commercial success is to be attained. From a purely metallurgical standpoint the production of Nicu steel in electric furnaces may be said to have been a success; but it is quite another matter to produce this steel to compete in the open market with nickel steels made by standard practise. It may be said, therefore, that Nicu steel might eventually become a commercial success, by first manufacturing Nicu pig in coke iron blast furnaces, providing the smelting of iron silicate is proved a success and then converting this pig to Nicu steel either in electric or open hearth furnaces.

COL. D. CARNEGIE, (Imperial Munitions Board).—Col. Leonard is to be congratulated on the work he has done in connection with the manufacture of copper-nickel steel from the Sudbury ores. This work is of the highest importance to Canada but I feel there is much yet to be done in an experimental way before it is proved really a commercial proposition. Col. Carnegie

The Imperial Munitions Board are now conducting experiments with regard to the quality and uses of the steel produced by this company.

MR. W. J. DICK, M.E.I.C., (Conservation Commission).—The successful and economic manufacture of Nicu steel is one of considerable importance both from the commercial and conservation standpoint. By the present practice all of the iron and sulphur content of the ore is lost, the former representing a loss of about three-quarters of a million tons of iron per year while the loss of sulphur amounted to about 100 tons per day. It is true that the markets could not consume this quantity of sulphur or sulphuric acid, but owing to the nature of the process now in use it is not possible to recover the sulphur as elemental sulphur or as sulphuric acid. Mr. Dick

Canada has always realized the importance of having an iron and steel industry.

Since 1896 a total of $16,785,827 has been paid by the Government of Canada in bounties for the production of iron and steel, the annual payments on pig-iron, puddled iron bars, steel, and manufactures of steel, being shown in the following table.—

Total Bounties on Iron and Steel Paid by the Government of Canada since 1896

Year Ended	Pig-Iron	Puddled Iron Bars	Steel	Manufactures of Steel
	$	$	$	$
June 30, 1896	104,105	5,611	59,499	....
" 1897	66,509	3,019	17,366	....
" 1898	165,654	7,706	67,454	....
" 1899	187,954	17,511	74,644	....
" 1900	238,296	10,121	64,360	....
" 1901	351,259	16,703	100,058	....
" 1902	693,108	20,550	77,431	....
" 1903	666,001	6,702	729,102	....
" 1904	533,982	11,669	347,990	15,321
" 1905	624,667	7,895	676,318	231,324
" 1906	687,632	5,875	941,000	369,832
Mar. 31, 1907 (9 months)	385,231	312	575,259	338,999
" 1908	863,817	....	1,092,201	347,135
" 1909	693,423	....	838,100	333,091
" 1910	573,969	....	695,752	538,812
" 1911	261,434	....	350,456	526,858
" 1912	....	....	....	166,750
" 1913	....	....	....	....
Total	7,097,041	113,674	6,706,990	2,868,122

These bounties have been the means of greatly stimulating the industry and, therefore, of great importance in connection with the manufacture of munitions in Canada. At the same time, the iron and steel industry is largely dependent on Newfoundland and the United States for supplies of iron ore. There are only two iron mines in Canada—the Magpie and the Helen.

The importance of Nicu steel to Canada is evident when it is considered that the Sudbury deposits consitute the greatest known reserves of iron in Canada.

Challies Mr. J. B. Challies, M.E.I.C. (Supt. Water Powers Branch, Dept. of the Interior).—We have at last heard of a real constructive conservation scheme of national moment. If the process for nickel-copper steel production so instructively described by Colonel Leonard and his colleague, Mr. Colvocoresses, proves to be the economic and commercial success which we are all so reasonably assured it will, it will mean the salvation of two of our great industries and the rejuvenation of a third. The assured production of high grade Nicu-steel will save our great steel industry and apparently without the necessity of Government bonuses. The production of large quantities of sulphur—the milk of chemical manufacture—will prove a boon to a great many important electro-chemical processes now largely dependent upon imported sulphur, and probably of equal importance will be the rejuvenation of the great industry of hydro power development in Central Canada. It is not clear yet that electric energy is an essential feature of the process necessary to produce Nicu-steel. It is probable, however, that, when all the factors involved are carefully weighed, cheap electric power will prove to be an integral part of the project. Fortunately Canada has in her vast water power resources potential possibilities which can, if necessary, be called upon. Mr. Challies

Probably the most interesting feature of the whole matter, however, is that raised by Capt. Jamieson, namely, the relationship of this process to the world production of steel. We sincerely hope that Capt. Jamieson's prediction that Canada, as a result of this process will be at least able to take the position occupied by Germany in the world production of steel previous to the war. If so, the future of our country is indeed a rosy one for the engineer.

Mr. Adam Shortt.—Colonel Leonard's paper and much of the discussion which followed it leads to the practical conclusion, so often verified in other cases, that the real line of progress in the development of national resources lies in a close study of the peculiar nature and possibilities of these resources. Yet, lacking original and independent initiative, an addition to the careful study of the world's past experience, the popular tendency is towards the mechanical reproduction of the methods and processes which have been justified elsewhere. As this mechanical reproduction of foreign methods is commonly accompanied in its early stages with considerable loss, it is sought to compensate for this by drawing on the national resources, either through direct bounties or indirect taxation via customs duties. A closer examination commonly reveals the fact, however, that the successes achieved elsewhere, and which it is frequently so difficult to reproduce under new conditions, have themselves been due to just that enterprising study of the special features of the peculiar combination of factors which every new country or area presents. Mr. Shortt

The possibilities of turning out from the iron, nickel and copper ores of the Sudbury region, under the most favourable market conditions, indefinite quantities of nicu-steel, without first separating and then recombining these metals, suggests a revolution in the production of this highly valuable alloy, comparable only to that accomplished through perfecting the methods of producing carbonized steel directly from the ores as compared with the older and enormously more costly methods of carbonizing

Mr. Shortt wrought iron. It would appear that what chiefly remains to be done is to work out the details of the most efficient and economic method of producing from the ores and slags of the Sudbury region the most effective alloy or series of alloys of iron, nickel and copper suitable for the uses to which the finished product may be applied. Here we have a series of problems of the highest national interest which at once calls for and justifies liberal assistance on the part of the government. It is just at this stage in the development of national resources that government assistance is certain to be most effective and most completely justified. Such a policy contributes to the national interest as a whole, since the results of the demonstrations which may be made through government assistance will be available for any corporate or private interests which may be able and willing to undertake as a commercial venture what has been demonstrated to be a practicable use of a great national resource. The government would be in an excellent position to attach whatever regulations may be necessary in the public interest to the private development of these resources. In this way, and at a mere fraction of the outlay incurred in the line of iron and steel bounties, the government may place an incalculably important industry on a sound financial basis attractive to either private or foreign capital.

Mr. DeCew. MR. J. A. DECEW, A.M.E.I.C.—Mr. President and Gentlemen, I am pleased to be able to express my interest and appreciation of this paper as there are several aspects of it which are of particular interest to the chemical engineer, one being the remarkable chemical inactivity of this new product. These copper steels which contain such a small quantity of copper in solution in the iron, become so resistant to corrosion that they compete directly with the almost chemically pure iron, which is now widely used for those purposes where corrosion is a great disadvantage.

The economic importance of this "Nicu Steel" to the industries of Canada is perhaps not yet fully appreciated and I sincerely trust that nothing will occur to retard its development.

KETTLE RAPIDS BRIDGE

By W. Chase Thomson, M.E.I.C.

(Read at a meeting of the Society on April 11th, 1918).

GENERAL

The Hudson Bay Railway extends from The Pas, the northern terminus of the Canadian Northern Railway in Manitoba, to Port Nelson on Hudson's Bay, a distance of 424 miles. Although primarily intended as a short route for the export of grain to Europe, the railway opens up a valuable territory, rich in minerals, fish and pulp-wood and of great agricultural possibilities. The grading has been completed throughout, and the rails have been laid to mile 332.

There are three important bridges on the line: The first crosses the Saskatchewan River at The Pas, and comprises four fixed-spans of about 150 feet each together with a swing-span of about 250 feet; the second crosses the Nelson River at Manitou Rapids, mile 242, and is a handsome structure of conventional deck cantilever type, with a channel-span of 304 feet 6 inches and anchor-spans of 108 feet 9 inches, supplemented by an 85 foot plate-girder span at the north end; the third, which is the subject of this paper, is at the second crossing of the Nelson River, or Kettle Rapids, mile 332, the present end of steel. The latitude of the bridge-site is 56°–23′–28″ N., and the longitude, 94°–34′–49″ W.

The Nelson is one of the great rivers of Canada, its drainage including the prairies of Alberta, Saskatchewan and Manitoba on the west, the Red River Valley on the south and part of Ontario on the east; but, owing to the intervention of Lake Winnipeg which serves as a huge reservoir, the flow of water in this river throughout the year is remarkably uniform. At Kettle Rapids, the lowest water-level recorded to date is 316.0, as observed December 31, 1917; and the highest water-level, with the river unobstructed by ice, 319.0, as observed November 3, 1916; a difference of only 3 feet. But, with the freezing of the river and the consequent jamming of huge quantities of drift-ice, the channel is greatly obstructed and the water rises suddenly. In the winter of 1916-17, this phenomenal rise began at Kettle Rapids on January 19th, when the water was at elevation 317.0, and the greatest height, 338.5, was reached on February 3rd. Within two days, however, the water dropped 7 feet; after which, it gradually subsided to 325.0 on May 23rd; it then went down suddenly and reached a normal elevation of 317.0 on June 6th, when the river was clear of ice. Owing to the exceptionally low temperature throughout December 1917, the ice-jam occured on the last day of that month, which was much earlier than usual. The water was then at elevation 316.0, but rose 9 feet during the next 24 hours; and, on January 6th when the last observation was made, it had reached elevation 335.0, or within 3.5 feet of the recorded maximum.

The highest ice-peaks have always been found on the islands, where piers 2 and 3 are located. In the winter of 1916-17, when the water was at its maximum height of 338.5, there were ice-peaks as high as 344.5, the same as had been observed during the winter of 1913-14; but, with the fall of the water, they settled to elevation 342.0, and remained there until melted.

The main channel at the bridge-site is only 350 feet wide, and estimated to be about 200 feet deep at the centre; the current is very swift, and there is always a certain amount of open water. Directly above and below the bridge-site, however, the river freezes all the way across, but only after the jamming of the ice and the consequent rising of the water. It is evident that there can never be any danger from ice, either to the superstructure or to the piers; for the steelwork is 15 feet clear of the highest fixed ice-peaks, and there is running-ice only when the water-level is much below its maximum elevation.

A diagram of water-levels, from June 1, 1916, to January 6, 1918, is shewn on plate I.

In locating the line, advantage was taken of two very conveniently-placed islands, allowing a central span of 400 feet, with piers and abutments on the solid rock. This rock is of pre-cambrian origin and is a tough granitoid gneiss.

The general design and dimensions of the bridge, including the substructure, are shewn on plate II. It will be noted that it is a continuous structure 1000 feet long, having a channel-span of 400 feet and two side-spans of 300 feet each. The trusses, or main girders, are of the sub-divided Warren type, 50 feet deep throughout, 24 feet apart centre to centre, having 25-foot panels. There are two lines of stringers, 8 feet apart centre to centre; and the base-of-rail is 17 feet 6 inches above the centre-line of the bottom-chords. The structure is rivetted throughout, and all bracing is rigid; it is fixed at pier 3, and provided with expansion-rollers at all other bearings. The ties are 8 x 12 inches, 14 feet long, spaced 12 inches centre to centre; they are notched ½ inch on the stringers, and every fourth tie is fastened thereto by a ¾-inch hook-bolt. The outer guard-timbers are 8 x 9 inches, spaced 10 feet 10 inches in the clear; they are notched one inch and secured to every fourth tie by a $\frac{3}{4}$-inch bolt. Steel guard-rails, weighing 60 lbs. per yard, are provided inside of the running rails, with 8 inches clearance between heads; they are brought together in a frog beyond the ends of the bridge. The main (or running) rails are of the American Society of Civil Engineers' standard section, weighing 80 lbs. per yard; at abutment 1, where the total expansion and contraction of the bridge will be about 8 inches, they are provided with specially-designed expansion-joints of the split-switch form, with points of manganese-steel. Refuge-bays, for pedestrians, are provided at intervals of 200 feet.

Three types of bridges were practicable for this location: 1st, simple spans, with temporary members over the piers for cantilever-erection of the channel-span; 2nd, the conventional cantilever bridge, with a central freely-suspended span; 3rd, a true continuous-girder bridge. The first would have been satisfactory, but un-economical, owing to the great weight of extra metal required for erection stresses only. The second was rejected partly on account of the objectionable articulated joints at the ends of the suspended span, but principally because of the expensive shop and erection work in connection therewith; for an economically-designed cantilever structure would have required a much greater depth over the piers, with considerably less depth at the abutments and for the suspended span, resulting in sloping chords and irregular webbing; besides, in order to obtain such economical proportions, it would have been necessary to locate the bottom-chords as close to the base-of-rail as possible, thus largely increasing the quantity of concrete in the substructure.

The third type, as designed and built, is the most rigid of all, and the most economical; for it required no extra metal for erection-stresses, except in the bottom-chords of the channel-span adjacent to the piers, where the increase of section was slight; the simplicity and uniformity of the framing reduced the cost of fabrication to a minimum; and the continuous horizontal chords, without adjustable joints, greatly facilitated the work of erection. It is admitted that continuous-girder bridges have been regarded somewhat unfavourably in the past; for it has been claimed that the usual theory for computing the stresses therein, which assumes a constant moment of inertia, is inexact; that the computation of the stresses is too difficult and tedious; finally, that the least settlement of any support would radically alter the stresses and thus endanger the structure. No doubt, in the old days of pin-connected bridges, continuous girders were undesirable in many respects; although a notable example of such a structure, which has received much praise and which gave excellent service for many years, was the old Canadian Pacific Railway bridge at Lachine. Now that pin-connections have been almost entirely superseded by rivetted joints, continuous girders are growing in favour; and the most remarkable example of such construction is to be found in the recently-constructed Sciotoville Bridge over the Ohio River, comprising two continuous spans of 775 feet each.

Regarding the objection, that the computed stresses are inexact, it may be stated that, in the present instance, the reactions were first computed for panel-concentrations by formulae as given in Merriman and Jacoby's "Roofs and Bridges", part IV, pp. 12 and 13, and afterwards checked by the elastic method. The difference in the results obtained by these two methods was less than ½ of 1%, which should satisfy the most exacting. This close agreement was undoubtedly due to the parallel chords and nearly constant moment of inertia; but, in the most extreme case, the error due to the use of the formulae would probably not exceed 6%.

The objection, that the labour of computing the stresses for continuous girders is too difficult and tedious, is unworthy of notice, especially where a large and important structure is concerned.

Finally, the claim with reference to results that would be produced by a possible settlement of one or more of the supports, is more rational, but much exaggerated; for continuous girders are very elastic structures, and can accommodate themselves to small settlements of supports without developing serious alterations of stress in their members. In this case, the ends of the trusses, if unsupported, would deflect 25 inches below the horizontal line from dead-load; and the alteration in the dead-load reactions at the abutments due to raising or lowering the end supports a whole inch would only be 9500 lbs., or 4%, whilst the reactions and stresses for the live-load would not be affected at all. Moreover, it is inconceivable that any settlement of the foundations can take place, as they are all on the solid rock; otherwise, this design would not have been adopted. Furthermore, in order to provide for possible small inaccuracies in fabrication or erection, the ends were made adjustable by allowing 1⅝ inches for shims between the shoe-castings and the bottom-chords; and hydraulic-jacks, with gauges attached, were used to set the ends at the height necessary to obtain the computed dead-load reaction. In this connection, and referring again to the Sciotoville Bridge, the following quotation from an article by Clyde B. Pyle, published in Engineering News-Record, January 31, 1918, will be of interest:

"One of the most striking features of the entire erection was the curve for the last few inches of the jacking, after the steel towers were both free. The computed increment for each inch of lift was 7.5 tons; and the actual increase in load was too small be read on the gauges.

"It is quite evident from this condition that, for long-span continuous trusses, it is not as vital a point as was formerly thought to be the case to have the supports at exactly correct elevations. In this case an error in setting one of the end supports, say as much as 3 inches, would have changed the end reaction 22.5 tons, or the stress in the end-post 32 tons, which would be less than the probable error in computing the actual stress in that member. The worst condition of shop-work, erection and setting of shoes could not possibly total more than one inch; so that the certainty of stresses and therefore the safety of such a bridge is left without question.

"The fact that complications enter into the design and erection cannot bar the use of such bridges as long as they are economical. The reasons usually given, that the stresses are not statically determinate and that uncertainties of stresses result from slight errors in elevation of the supports, are no longer valid."

DETAILS OF DESIGN

The structure has been designed in accordance with the General Specification for Steel Bridges, issued by the Department of Railways and Canals in 1908, except for a slight modification in the impact formula, affecting alternating stresses only, and a change in the allowable unit-stresses for compression-members.

In the matter of impact, when dealing with alternate live-load stresses, the Department's specification requires the impact to be computed by squaring the arithmetical sum of the tension and compression stresses due to the live-load (or the range of stress), and dividing by the maximum algebraic sum of co-existing dead-load and live-load stresses, or $I = \frac{range^2}{max.}$ Now, taking an extreme case in which a member is subject to alternating live-load stresses of equal amounts, but no dead-load stress, the impact would be $\frac{(L + L)^2}{L} = 4L$, or four times the live-load stress of either kind, which is certainly excessive. If, however, we take for the *range* the live-load stress of one kind and add to it 4/10 of that of the other kind, we have $\frac{(L + 0.4L)^2}{L} = 2L$, approximately, or an impact equal to twice the live-load stress of either kind, which would seem to be ample. In conformity to the above argument, impact has been computed by the formula, $I = \frac{range^2}{max.}$, with the arbitrary stipulation that the *range* shall be taken as the arithmetical sum of the live-load stress of the greater kind and 4/10 of that of the lesser. When the live-load stress is of one kind only, the formula reduces to $I = \frac{L^2}{L + D}$, in which L = live-load stress and D = dead-load stress.

Concerning the unit-stresses for compression-members, the Department's specification calls for 16000 lbs. per square-inch reduced by Gordon's formula, using in the denominator the factor 18000 for square-ends, 12000 for one square and one pin-end, and 9000 for pin-ends. It is now quite generally recognised, however, that 16000 lbs. per square-inch is entirely too high for short columns; and the Joint Committee on Columns and Struts in the United States, which has recently submitted its final report, recommends a maximum working unit-stress of 12000 lbs. per square-inch, which is therein shewn to provide a factor-of-safety of 2, or the same as for tension-members when designed for a unit-stress of 16000 lbs. per square-inch on the net section. In the General Specification for Steel Highway Bridges, recently adopted by the Society, the formula for compression-members is $12000 - 0.3\,(l/r)^2$, which becomes zero when $l/r = 200$. In this bridge, the compression-members have been designed in accordance with the formula, $12000/1 + \frac{(l\,r)^2}{36000}$, which agrees closely with that adopted by the Society for values of l/r up to 70, but gives somewhat higher unit-stresses for greater working ratios.

The stresses, with data for same and the make-up of the members of the structure, are shewn on plate III. The live-load indicated is "Class Heavy" of the Department's specification, above noted; and the dead-load concentrations at panel-points represent the actual weights of steelwork and floor, carefully computed from the detail drawings. The bottom laterals have been proportioned on the assumption that the whole of the specified wind-loads, both during erection and afterwards, would be resisted thereby; and the wind-load stresses in the bottom-chords include the vertical effect of the wind-loads, equal to their moment about the bottom-chords divided by the horizontal distance centre to centre of chords. The design includes provision for cantilever-erection from piers 2 and 3 to the centre of the channel-span. Wind and erection-stresses are shewn only where they affect the sectional area of members.

Provision for traction and braking forces has been made by horizontal trussing attached to the top-flange of the stringers and to the floorbeams at points $M0$, $M4$, $M8$, $M12$, etc., or 100 feet apart, as shewn on plates II and III; which forces are transmitted to the main girders through the inclined struts $M0$–$M1$, $M3$–$M4$, $M4$–$M5$, $M7$–$M8$, etc.

The end floorbeams are provided with stiffeners and bearing-plates at points 16 feet apart, for jacking-up the bridge; and the floorbeams at $M12$ have been specially designed for lifting the bridge, with unit-stresses increased by 50% and having stiffeners and bearing-plates at points 14 feet apart.

Latticing of main members has been avoided as far as practicable; but the open sides of compression chord-members are double-latticed with 5 x ⅝-inch flats, having two rivets at ends and at intersections; tension chord-members are similarly latticed with 5 x ½-inch flats.

All of the principal web-members are provided with substantial longitudinal diaphragms, which are considered as part of the effective section thereof; and the heavy compression diagonals, $U6$–$L8$, $U10$–$L12$, $L12$–$U14$ and $L16$–$U18$, are further stiffened by tie-plates on their out-standing flanges. All joints and splices are fully rivetted throughout.

Rocker-bearings are provided throughout, having 8-inch bearing-pins at the piers and 6-inch bearing-pins at the abutments; and the shoes are steel castings. The bridge is fixed at pier 3 and movable at pier 2 as well as at the abutments. At pier 2, the expansion-rollers are 8 inches in diameter, and each set is provided with four 12-tooth cut pinions to prevent skewing. Substantial curtain-plates are supplied for the protection of the gears and to keep out the dust; but they are removable for inspection and cleaning of the bearings. At the abutments, the expansion-rollers are 6 inches in diameter and similarly provided with alignment gears and curtain-plates. These expansion-bearings are shewn on plate IV. The fixed-bearings at pier 3 are similar to the expansion-bearings at pier 2, except for rollers and bed-plates. The bridge-seats are tool-dressed perfectly level and to the exact elevations called for on the drawings; and sheet-lead, ⅛ inch thick, is provided to equalise any minor irregularities of the surfaces.

Owing to the small deflection of this bridge, which is only 3 inches at the centre of the channel-span, for dead-load combined with the maximum effect of the live-load, it was considered unnecessary to provide for a perfectly straight bottom-chord under any particular condition of loading; so the trusses have been cambered, in accordance with the more usual method employed for simple spans of moderate length, by increasing the length of the top-chord panels, as shewn in the upper diagram on plate V. Members $U10$–$U12$, and $U12$–$U14$ have been correspondingly shortened; and ¾ inch has been added to the verticals $U12$–$L12$, to obviate a slight kink at panel-points $U12$. At panel-points $L12$, the ends of the abutting chord-members have been bevelled to accommodate the form of the trusses when fully loaded. This method of cambering has greatly simplified the shop-work; and the results are entirely satisfactory.

The total estimated weight of steel in the structure (including floor-bolts), computed from the writer's detail drawings before the contract had been awarded, was 4,424,000 lbs.; and the actual shipping-weight, as determined by the scales, was 4,415,000 lbs.

ERECTION

Erection was started on June 6th, the earliest date possible; for, before the falsework for the southern anchor-span could be placed, it was necessary to blow up with dynamite huge masses of ice. This anchor-span was then erected in the usual manner by a 75-ton derrick-car, having trucks 35 feet centre to centre and a single 50-foot boom.

At $L0$, bottom-chords were set 10 inches low, by omitting the upper shoe-castings and using flatted pins for bearings. This was to provide for the deflection of the channel-span during erection, and to insure that the connections at $L20$ could be effected before the chords at $U20$ would meet. After the anchor-span had been fully rivetted, the southern half of the channel-span was then erected as a cantilever by the same derrick-car, the rivetting following closely behind the work of erection. Panel-point $L14$ was supported temporarily by wire cables from panel-point $U12$, until the connection had been made at $U14$; likewise, panel-point $L18$ was supported from panel-point $U16$, until the connection had been made at $U18$. By August 18th, or eleven weeks from the date of beginning, the first half of the bridge was fully erected; and the rivetting on this portion of the structure was completed one week later.

The next, and perhaps the most difficult piece of work in connection with the entire erection, was the construction of a double cableway for transporting to the opposite side of the river the materials for the northern half of the bridge. The cables were supported on a rocker-bent 40 feet high, standing on the top-chords of the southern cantilever at panel-point $U18$, and on a timber tower 120 feet high from the ground-surface, located behind abutment 4, with centre-line 60 feet from panel-point $L0$. The span of the cableway was 611 feet; the sag, under maximum load, 36 feet; and the horizontal distance of the anchorages, at both ends, from adjacent supports, 400 feet. A triangular equalising-girder, suspended at its ends from the cables and having a lifting-hook at the centre, was provided for loading the cables equally. The cableway was designed for a live-load of 14 tons, the weight of the heaviest piece to be transported. Its general construction and method of operation are clearly indicated in the reproductions of photographs, herewith. In addition to its principal function of transporting materials, in which service it gave entire satisfaction, the cableway was of great assistance in the erection of steelwork.

The falsework for the northern anchor-span was then constructed, with extension-bents reaching to the floor-level, for the accommodation of the special traveller provided for the erection of the northern half of the bridge. As this traveller was to be used on the top-chords as well as at the floor-level, its four trucks (of two 24-inch double-flanged wheels each) were spaced 24 feet centre to centre transversely, the same as the trusses, and 50 feet centre to centre longitudinally, to coincide with the panel-points; it was fitted with two 62-foot booms and a hoisting-engine; and its entire weight, including counterweight, was 60 tons, equally distributed on the four trucks. To provide for the weight of the traveller only, when moving, the top-chord members were supported at their middle-point by temporary timber posts, resting on special seats at panel-points $M3$, $M5$, $M7$, $M9$, etc.

After the delay incident to the construction of the cableway, and the falsework for the northern anchor-span, erection of steel for the northern half of the bridge commenced on September 17th. Beginning at pier 3 (with the traveller at the floor-level and working towards abutment 4) the floor-system, bottom-laterals and lower members of the trusses were placed, and the falsework-extensions removed. The traveller was then blocked up to the height of the top-chords, requiring a week for this operation; and the upper part of the steelwork for this anchor-span was erected, working from abutment 4 towards pier 3. When the traveller had passed panel-point $U6$, an additional rocker-bent, 40 feet high, was set up there as an intermediate support for the cables, thereby reducing the span to 400 feet, the maximum sag to 22 feet, and providing ample working clearance above the top of the steelwork. The cantilever-erection of the northern half of the channel-span was accomplished in the same manner as for the southern half, except that the members were placed principally by the special traveller, which required temporary timber supports at the centre of the top-chord panels, already mentioned.

From the time of placing the traveller on the top-chords to within two days of the end of November, when the weather suddenly turned severely cold, rapid progress had been made; and it was confidently expected that the bridge would be entirely completed before the end of the first week in December. Accordingly, the writer was instructed to proceed to Kettle Rapids for the purpose of making a final inspection of the bridge and to supervise the adjustment of the end reactions. On his arrival at the bridge-site, December 8th, the work was still held up; but it was found that the centre connections at $L20$ had been made, and without the least difficulty; for, on meeting, the trusses had been in perfect alignment and the deflections of the cantilevered arms, exactly equal; thus it had only been necessary to jack forward on its rollers the southern half of the structure, which had purposely been set back 5 inches to facilitate the erection of the closing members. On the southern half of the bridge, the timber floor was practically complete: and the ties had been roughly distributed over the northern half, except at one panel adjacent to the centre of the channel-span, where the stringers had not at that date been placed; thus the structure was practically supporting its full dead-load.

Under these conditions, careful levels were taken to ascertain the exact deflections of the trusses or main girders, with results as shewn in the middle diagram on plate V. It will be noted that the curves of the bottom-chords are remarkably uniform; and it was a great source of gratification to the writer to find that the centre ordinate was exactly the same as had been computed. At panel-point $U20$, the distance between the ends of adjacent chord-members was one inch.

The weather having moderated slightly, although still very cold, work on the bridge was resumed December 16th; for it was hoped that it might yet be possible to complete it this winter. On the 22nd, the ends were lifted 3½ inches, which was just sufficient to bring the ends of the top-chords at $U20$ to a firm bearing. Owing to frequent stoppages due to weather conditions, it was not until the last day of the month that the rivetting of the main members was completed, and jacking of the ends was resumed. This operation was again interrupted by New Year's day (which was too cold to work, in any case); but the ends had been raised sufficiently by January 2nd to permit of placing the upper shoe-castings, without shims. Although the ends were thus 1⅝ inches below their normal position, the load at each of the four corners, as indicated by the gauges on the hydraulic jacks, was exactly 118½ tons, the amount of the computed, dead-load reaction. The bridge at the time, however, was covered with many tons of ice and snow; thus it was impossible to determine very accurately the reactions for the normal dead-load.

It had by this time been decided to give up the attempt to complete the bridge during the winter of 1917-18; for the men could not work to advantage; a satisfactory job could not be made of the track-work; the painting could not be done until the advent of mild weather; and the bridge was perfectly safe. A final adjustment will therefore be made under more favourable conditions, when it is expected that the ends will require to be raised about another inch.

With the ends 1⅝ inches low, levels were again taken on the bridge, with the satisfactory results indicated in the lower diagram on plate V; for the camber at the centre of the channel-span was found to be 1⅞ inches, whereas the maximum computed deflection due to the specified live-load is 1¾ inches.

The closing panel of stringers was placed January 4th, which ended the work for the season. The remaining work comprises a small amount of rivetting for secondary parts; some minor adjustments; the completion of the timber deck, including the laying of the rails; and painting.

On January 6th, the Superintendent of Erection pulled out, with his men and such of his equipment as will not be needed to finish the bridge, leaving a watchman in charge.

SUBSTRUCTURE

The substructure is of concrete throughout, composed of pit-gravel and cement, in such porportions as were found by trial to give the best results. It had been intended to construct at least the abutment and pier on the southern side of the river during the autumn of 1916; but the track did not reach the bridge-site until the end of October; cold weather set in shortly after, and there was barely time to construct the foundation for abutment 1. Excavation for this foundation was carried to a depth of over 10 feet, through frozen clay and silt, to the solid rock. The concrete was placed during the second week in December, and in very cold weather; but the materials had been heated, the mass was large and the result was entirely satisfactory, as found from a careful inspection the following spring. The abutment was completed during the month of April, 1917.

Operations at pier 2 were begun on April 10th, and under very adverse circumstances; for the river was then at elevation 328.0, or 10 feet above the average rock-surface at this point; and the rock was covered with a solid mass of ice, 25 feet thick. However, it was necessary to get ahead with the work as rapidly as possible; so the ice was excavated, and the rock was bared by May 5th, at which date the water had fallen to elevation 325.0. Although the ice-walls of the exacavated shaft appeared to be perfectly solid throughout, the water percolated through and stood at the same elevation as that in the open river-channel; but it was perfectly still, without current or surge. A timber caisson, conforming on the bottom to the irregularities of the rock-surface, was then constructed; and all small openings therein were sealed by sheet-piling, carefully scribed and driven so as to broom the ends thereof. Every inch of the rock-surface inside of the caisson was then picked with needle-bars, to insure that it was entirely clear of ice; and heated concrete was deposited by deep-sea buckets. The rock-surface at this pier had previously been carefully examined during low water, and found to be absolutely sound; thus every confidence may be placed in the foundation. The footing for this pier was completed on May 9th; the construction of the main shaft thereof offered no difficulties, and was effected without incident.

At pier 3, no difficulties incident to water or ice were encountered; or work at this point was not started until June 29th; but there was a horizontal fissure in the rock at about elevation 322.0, which necessitated blowing up by dynamite the overlying mass. This resulted in giving an entirely satisfactory though very irregular foundation, to which the footing for the pier was made to conform.

Excavation for abutment 4 was commenced on June 14th, and was carried through about 10 feet of frozen clay, silt and boulders to the solid rock. The footing, up to elevation 341.5, was completed July 21st.

The pit-gravel, used throughout on this work, was invariably frozen and required to be thawed by steam; thus all of the concrete was placed warm, and with most gratifying results; for, on removal of the forms, not a single bad spot was discovered.

The butterfly wing-walls of the abutments were reinforced by twisted steel rods, one inch square, placed 3 inches from the rear surface. There were horizontals, 6 inches apart, wired to verticals, 3 feet apart. In addition, two such rods were placed along the upper edge of the wings.

The total quantity of concrete in the work is about 3000 cubic-yards; and, of reinforcing steel in the wing-walls, 2300 lbs.

The laying out of the work was difficult and tedious, owing to the irregularity of the ground and to the necessity of locating pier 3 by triangulation; but the instrument-work was done with such care and precision that all important dimensions and distances were afterwards found to be practically exact. In locating the centre-line of bed-plates on pier 2, and that of the shoe-castings on pier 3, where great accuracy was desired, the piano-wire method of measurement was used, taking into account the pull on the ends of the wire and the corresponding sag, as determined on a level surface, and making the proper correction for temperature. The distance between centres of bearings on piers 2 and 3 was afterwards found to agree with the steel structure, as built, within 5/16 inch. It had been specified that the centre-line of the expansion-shoes should coincide with that of the corresponding bed-plates at a temperature of 30 degrees, Fahrenheit; and, on inspection, with the thermometer at zero, the centre-line of the roller-shoes at pier 2 was found to be exactly 1¼ inches northerly of the centre-line of the bed-plates, instead of 15/16 inch, the amount of contraction in the steelwork in 400 feet for a fall in temperature of 30 degrees. Thus the distance between the bearings was too great by 5/16 inch. If there had been any appreciable error in the setting of the bearings on these piers, it could have been rectified, as provision had been made for jacking-up the structure, if necessary; but, as the dead-load reactions here are about 500 tons each, and the shoes very awkward to move, any such adjustment after erection would have been difficult and expensive.

The entire work has been under the general supervision of Mr. W. A. Bowden, M. Can. Soc. C. E., Chief Engineer, Department of Railways and Canals, Ottawa; and of Mr. J. W. Porter, M. Can. Soc. C. E., Chief Engineer, Hudson Bay Railway, The Pas. It was designed in full detail by the writer, who has been retained throughout for consultation in connection therewith. The superstructure was fabricated and erected by The Canadian Bridge Company, Limited, Walkerville, Ont. Mr. T. B. Campbell, M. Can. Soc. C. E., Bridge Engineer, Hudson Bay Railway, was in charge at the bridge-site, under whose immediate supervision the concrete-work was constructed, and by whom all lines and elevations for the erection of the steelwork were given; Mr. I. E. Mahon was the Superintendent of Erection for the bridge company; and Mr. James Carr, Representative of the Canadian Inspection and Testing Laboratories, Limited, attended to the field-inspection. The entire work has been carried out without loss of life and without a serious accident.

Great credit is due to Messrs. McDonald Brothers, for the excellence of the concrete-work; to Mr. Campbell, for the accuracy of his lines and elevations, and for his efficient supervision; to the engineers of the bridge company, for their splendidly-conceived and carefully-prepared scheme of erection; to the shops, for the neatness and accuracy of workmanship; finally, to Mr. Mahon, for his skill and care in handling under difficulties this important and somewhat unusual piece of erection-work.

DISCUSSION

Mr. F. P. SHEARWOOD, M.E.I.C.—The continuous span design, adopted by Mr. Thomson, must have been very economical compared with either simple span or cantilever construction, but looking at the profile of the crossing, the reason for the long anchor spans is not evident, and if these spans could have been reduced by about 100 feet a considerable saving in the weight of steel and total cost of the bridge would have been effected. Mr. Shearwood

An unusual and good feature of the design is the half through type. Placing the floor near the neutral axis of the truss instead of at the upper and lower chords, has among other advantages, that of keeping the longitudinal stringers in the position where the changes in length due to stresses in the chords, are zero. It also facilitates the floor beam and truss connections by placing them clear of one another.

The adoption of the H and I shaped sections for the web members simplifies the connections, provides the best form of diaphragms to equalize the loading of the inner and outer material of the members as well as cheapening the manufacture. It is mentioned that plates are used to stiffen the outstanding flanges and it would be interesting to know how far apart these tie plates are placed, so as to compare it with the spacing adopted on the tie plated columns which were tested and condemned by the A.R.C.A. recently. The former have the tie plates merely as secondary stiffening, whereas the latter relied on them entirely to make the channels act together. The contrast in spacing may show that the columns tested by the A.R.E. should have had the tie plates placed about one third the distance apart to represent usual practice and obtain a fair result.

Mr. Shearwood The paper mentions that all the horizontal loads are carried by the lower lateral bracing. Sway bracing is provided at every vertical and the verticals are very stiff in the direction required to resist horizontal forces. It would be interesting to know whether the author considered the question of omitting them and really, taking all upper horizontal forces directly to the lower laterals, instead of merely assuming this effect. Top laterals often appear to be superfluous members, in fact it is even possible that they may be harmful.

The pier members have an ingenious method of controlling the rollers which should certainly be effective in keeping them from skewing or creeping. Past experience has shown that this is very necessary. The arrangement has the advantage of having the mechanism exposed for inspection.

The design for the cast shoes and beds is unusual in having a box section which does not seem to have the metal so advantageously distributed to resist the leading as the usual open ribbed types. It has also the disadvantage of more difficult moulding.

The erection scheme is interesting and proved very successful, but why was it necessary to carry the traveller on extended falsework to the main pier so as to lay the lower chord, and then elevate it to the upper chord height, instead of at once starting off at the upper chord height to assemble the whole truss and falsework as it travelled out from the abutment towards the main pier?

Mr. Butler Mr. M. J. Butler, C. M. G., M. E. I. C.—Mr. Thompson has contributed an interesting and valuable paper on an important bridge.

The railway in question is, however, one that, in my opinion, ought not to have been built by the Government of Canada. It was entered upon at a time, when the resources of the country were under severe strain due to the construction of the Grand Trunk Pacific and the liabilities incurred by the guarantees given to the Canadian Northern System.

Aside from the financial reason for not going on—no information of a reliable character existed as to the navigation possibilities of the Hudson Bay route. All the data available seemed to point to the fact, that a possible 2½ months might be had on the average—but, until a systematic study over a period of several years it would seem to have been wise and prudent to wait. Whether the actual period of reasonably safe navigation is two months—as I understand the information available—or four months, as I have heard estimated—the question of securing ships for such a route has not, I feel sure, been dealt with.

The information given in the paper, that there is along the line, "a valuable territory, rich in minerals, fish and pulp wood and of great agricultural possibilities," is certainly very important, if true.

I understand that the route is over a barren, sub-arctic tundra—with no land fit for settlement—that any spruce found along the line is scrub of no commercial value. The minerals,—it would be very interesting to learn what they are and where located. Mr. Butler

I am, of course, well aware, that the North West believes in the Hudson Bay route—and that the undertaking owes its existence to pressure on the Government from that part of the country. I only hope that their sanguine expectations may be realized.

I have read with interest the reasons which actuated Mr. Thompson in selecting a continuous truss and agree with him that the labour of computing the stresses is not an important matter. The design and details, both of manufacture and erection seem to me very good and reflect great credit on all concerned.

I assume that there is a general agreement that the assumptions upon which the elastic theory are based, viz. a constant modulus of elasticity and a constant moment of inertia, do not in fact exist. That certain approximate methods which have been developed check sufficiently with the "elastic theory" to re-assure the designer—that he is about as near right as is necessary to secure a safe structure. Too much refinement in bridge calculation, seems out of place. The increasing wheel loads of locomotive and rolling stock, demand that bridges be built adequate for all reasonable increase in weight.

Mr. Geo. F Porter, M.E.I.C —I wish to congratulate Mr. Thomson on the capable piece of work he has presented to the Institute this evening. His remarks in regard to the use of continuous girders where such conditions exist as in the present instance, are very well taken. Mr. Porter

There has been a very strong prejudice in the minds of American engineers against their use for fixed spans, while the usual type of draw bridge has of necessity been of this type. The fact of continuity has seldom been the cause of trouble except in cases where the foundations were poor and settlement took place.

I would like to ask Mr. Thomson, who has, no doubt, looked into the matter carefully, whether the saving of masonry due to the placing of the floor well above the bottom chord, thus making necessary the introduction of bottom struts and sway bracing beside special provision for traction strusses, really proved to be economical.

Mr. W. Chase Thomson, M.E.I.C.—Replying to Mr. Shearwood:—A preliminary design for the bridge had side spans of 200 feet, but was subject to uplift at the ends, with consequent hammering. Counterweighting, to overcome the uplift induced by the live-load, was considered, and would, no doubt, have been a satisfactory solution of the Mr. Thomson

Mr. Thomson problem, as this scheme was later adopted in connection with the new continuous-span bridge over the Allegheny River, described in Engineering News-Record, May 2, 1918. Side spans of 300 feet were finally adopted principally because it was considered advisable to provide the wider water-way, which latter will be restricted by a rock-fill embankment at the north end of the structure.

The supplementary tie-plates on the principal compression diagonals are spaced about 7 feet centres: they are about 21 inches long, with five rivets on each side.

Top laterals may be superfluous in the completed structure, although it is difficult to see how they can be harmful. In this instance, their chief function is to hold the chords in alignment, which might otherwise have a tendency to kink at the joints. In the Quebec Bridge, which has no top laterals, the upper chords are in tension throughout the anchor spans and cantilever arms.

The idea in adopting the box section for the cast shoes was to secure the best possible distribution of the load over the masonry. With the open-ribbed type, the edges of the bottom plate cannot be so well supported.

Replying to Mr. Butler:—Although the writer has no brief to defend the Hudson Bay Railway, he is of the opinion that the undertaking is less foolish than that in connection with many other lines which have been constructed during recent years; moreover, there are many who believe it will, when completed, fulfil its original function:—to help in the shipment of grain to Europe. It has been pretty well demonstrated that the Straits can be navigated not less than four months during the year, and it is hoped that this period may be extended to six months. The muskeags, which are so numerous, are generally only a few feet deep, and overlie a clay formation; when drained, they should be exceedingly fertile. Along the embankments, the writer has seen splendid specimens of oats and timothy, which had sprung up from seed accidently spilt; and several of the section-men have raised excellent vegetables. The lowest temperature thus far officially recorded, between The Pas and Port Nelson, is 49 below zero, whereas 60 below is quite common in districts growing the best wheat.

Great accuracy in the determination of stresses may not be necessary; but it is important to note that the usual formulæ developed from the elastic theory are entirely reliable, especially when applied to girders of uniform depth. As stated in the text of the paper, the reactions derived from formulæ applying to beams of constant moment-of-inertia had been checked by the more exact method of deflections, with practically identical results; moreover, the measured deflections of the structure and the scaled reactions compare very closely with the calculated amounts. As is well known, the modulus-of-elasticity for structural steel is one of its most reliable and constant properties. Considering the observed facts, Mr.

Butler's remark about "certain approximate methods which check sufficiently with the elastic theory to reassure the designer that he is about as near right as necessary to secure a safe structure" is somewhat misleading; for there can be no doubt that the calculated stresses for continuous-girder bridges are quite as reliable as those for so-called simple-span bridges. In both cases, there is far greater uncertainty regarding the actual live-load and impact. Mr. Thomson

In compliance with the expressed wish of the meeting before which the paper was read, plates VI and VII, covering typical structural details, have been added. These details have been designed with a view to economy compatible with strength. All joints are fully rivetted for the full value of the connected parts. The use of outside splice-plates where the main diagonals are attached to the gusset-plates, at upper panel-points 2, 6, 10, 14 and 18, and at lower panel-points 4, 8, 12, 16 and 20, has the double advantage in permitting the use of comparatively small gusset-plates, with short rivetted connections. At all middle panel-points, and at lower panel-points 2, 6, 10, 14 and 18, the gusset-plates are only ⅜ inch thick, but of ample strength to resist the stresses transmitted thereto by the hangers and sub-diagonals; and the remaining splice materials are just of sufficient width to conform to the dimensions (in elevation) of the main members. Owing to the large area of these gusset-plates, this arrangement results in a very considerable saving of metal. At upper panel-points 10 and 14, the gusset-plates are of sufficient size and thickness to resist the vertical and horizontal shears acting thereon. Owing to the slight bevel in the chords at lower panel-points 12 (1/64 inch in one foot), the gusset-plates extend below the chords ⅛ inch, with bottom edges planed straight to bear upon the shoe casting; furthermore, additional bearing area is provided by using tapered fillers under the bottom flange-angles of the chords, as shewn on the detail drawing of the large shoe, plate IV, the load being transmitted to these bottom flange-angles by the outside splice-plates which are ground to bear thereon, and in which additional rivets are provided, over and above the number required for their proportion of the bottom-chord stress.

The bridge was completed and the end bearings were finally adjusted on June 10, 1918. Without any shims, the deadload reaction at all four corners exceeded the computed amount (118.5 tons) by exactly 5 tons, or about 4%, which was considered entirely satisfactory. Thus the ends remain one and five-eighths inches below the normal; and the lower diagram on plate V represents the actual form of the completed structure, when unloaded. This is as might have been expected; for the cantilever erection of the channel-span naturally tended to increase the length of the top chords throughout, resulting in lowering the ends,—though but slightly.

Although the winter of 1917-18 was the most severe of any, along the line of the Hudson Bay Railway, according to official records, the maximum height of water and ice-peaks at Kettle Rapids, due to the jam, only

Mr. Thompson exceeded previous records by five-tenths of a foot. Thus there can be no doubt that the clear distance of 15 feet between the bottom chords and the previously-recorded maximum elevation of ice-peaks is more than ample to provide for all contingencies.

Replying to Mr. Porter:—The additional steelwork necessitated by the placing of the floor above the bottom chords is principally included in the bottom struts; and the entire additional weight of steel due to this arrangement is only about 75,000 lbs. which, at the contract price of 7.15c., amounts to $5362.50. This does not take into account the lower sway-bracing, which would otherwise be offset by deeper overhead sway-bracing; nor the inclined struts, in the plane of the trusses, which are necessary for stiffening the vertical members at panel-points 4, 8, 12, etc. The saving of concrete effected is about 350 cu. yds. which, at the cost of $15.00 per cu. yd., in place, is equivalent to $5250.00. Thus the cost of the extra steel is almost exactly balanced by the saving of masonry. But the plan adopted has the advantage of keeping the floorbeam connections clear of all panel-points, thus greatly simplifying the details; the application of the floorbeam concentrations to the verticals at a considerable distance from the end connections of the latter insures a much better distribution of stress among the component parts of the main truss-members; finally, the location of the track-stringers near the neutral-axis of the main girders obviates, largely, the usual racking on the stringer connections, due to the change in length of the chords under the action of the liveload. In addition to these advantages, it was desired to provide girders over the piers, strong enough for jacking up the structure in case of necessity; and the depth of the ordinary floorbeams was insufficient for this purpose. Moreover, the arrangement permitted of placing the end floorbeams underneath the stringers, thus obviating the necessity of brackets for carrying the floor-ties adjacent to the ballast walls.

In conclusion, the writer desires to express his thanks to the gentlemen who have taken part in the discussion of this paper for their general approval, so kindly expressed.

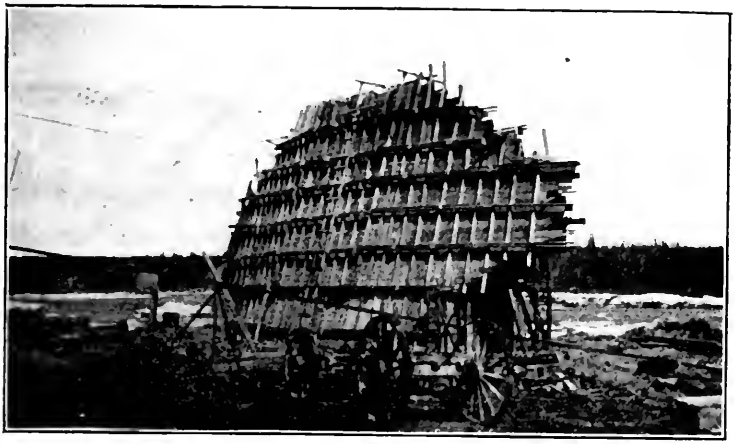


Fig. 1:—Abutment 1 under construction.

Fig. 2:—Pier 2 under construction.

Fig. 3:—Southern anchor-span erected; and beginning of cantilever-erection, shewing temporary supports for panel-points $L14$.

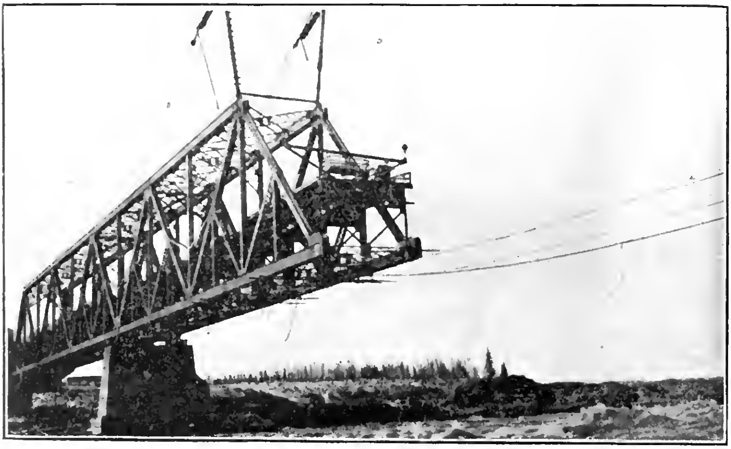


Fig. 4:—Southern half of bridge erected.

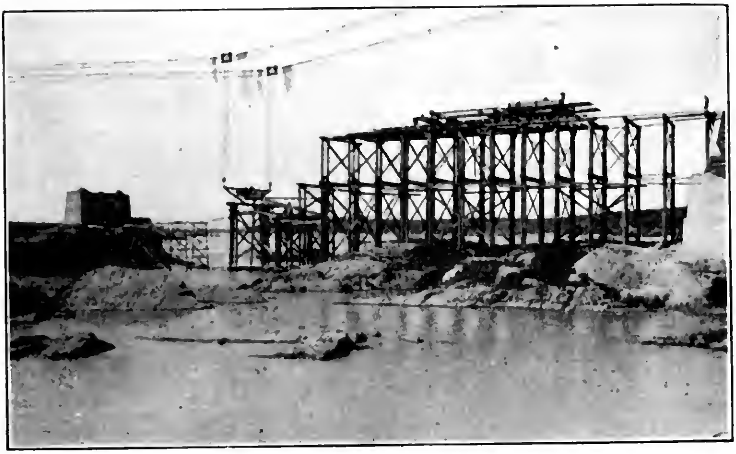


Fig. 5:—Falsework under construction for northern anchor-span.

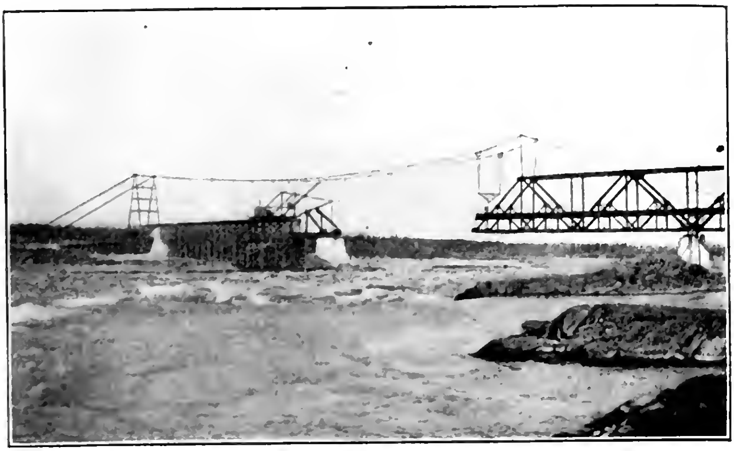


Fig. 6:—View of cableway; and beginning of erection of northern anchor-span, with traveller at floor level.

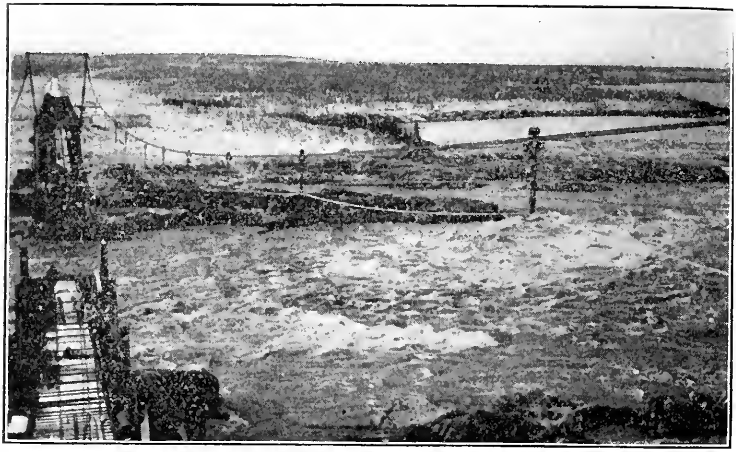


Fig. 7:—View from top of cableway-tower.

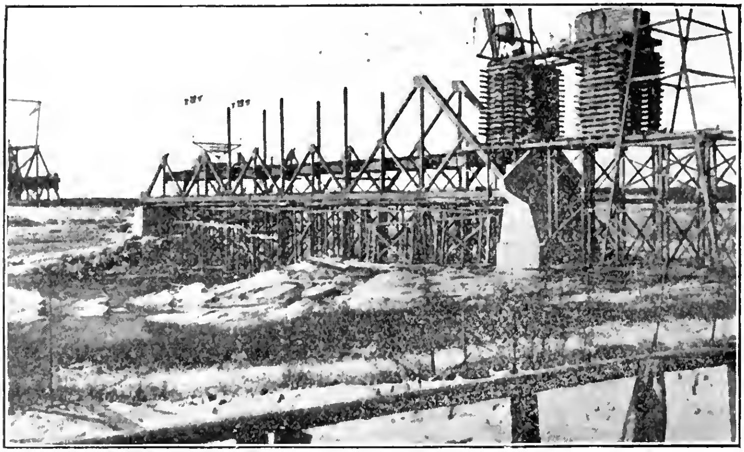


Fig. 8:—Lower steelwork for northern anchor-span erected; traveller raised on blocking, for working on top chords.

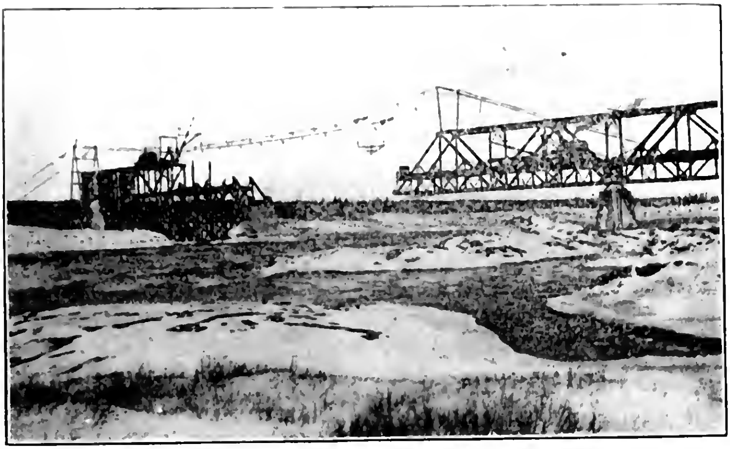


Fig. 9:—View shewing cableway with its equalizing-girder; derrick-car on southern cantilever; traveller on top-chords of northern anchor-span.

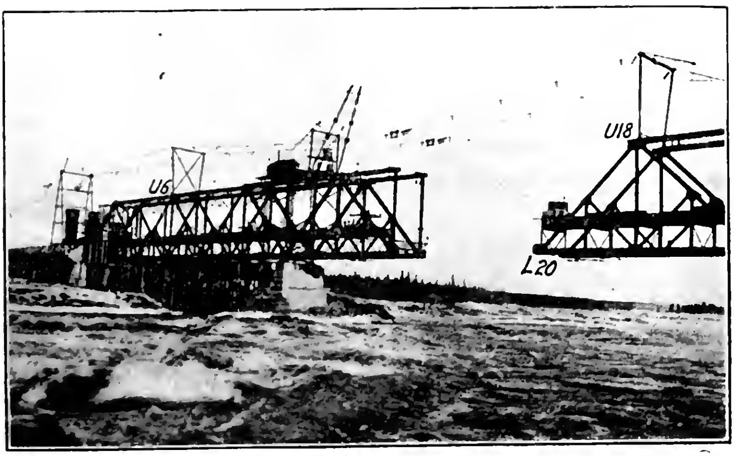


Fig. 10:—Northern anchor-span erected, also 100 feet of adjacent cantilever. Note additional rocker-bent at $U6$ for supporting cables.

Fig. 11:—View of bridge from south shore before cableway and traveller had been dismounted. Note that the lower members of the portal-struts and sway-bracing are missing: they were omitted temporarily for the accommodation of the derrick-car.

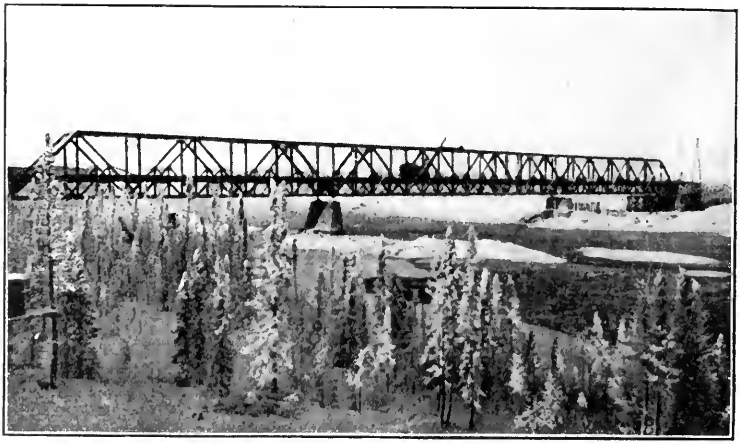


Fig. 12:—General view of bridge, taken from the south shore and looking up-stream.

CHAMPLAIN DRY DOCK FOR QUEBEC HARBOUR

By U. VALIQUET, M.E.I.C.

Superintending Engineer Department of Public Works.

(Read at Montreal and Ottawa Branches April 25th, 1918).

For a number of years the River St. Lawrence has been frequented by ocean steamers of such dimensions that they could not be accommodated in the Lorne Dry Dock, completed in 1886, at Lauzon, in the Harbour of Quebec.

In 1906 the Canadian Pacific Railway Steamship Company brought out their steamers "Empress of Britain" and "Empress of Ireland", of 65-foot beam; the Allan Line steamers "Virginian" and "Victorian" of 60-foot beam were also placed on the St. Lawrence route in that year. The "Bavarian" of somewhat narrower beam, 59 feet 3 inches, came to Quebec in 1905; thereafter the number of large ships placed on the St. Lawrence traffic increased rapidly until in 1912, there were 25 vessels that could not have been repaired in the long stretch of the St. Lawrence navigation for want of sufficient dock accommodation, the width of entrance of the present dry dock being only 62 feet. Any of these vessels that required docking had to be repaired temporarily as well as possible, while afloat, and taken either to Halifax or New York, which, in some cases, was a risky undertaking.

The case of the S. S. "Bavarian" was an unfortunate experience in this respect. On the 5th November 1905, this steamer ran aground with a full cargo from Montreal and Quebec, about 40 miles below Quebec, opposite Grosse Isle; although late in the fall she could have been raised and brought to Quebec had there been dock accommodation for her. Her beam was 59′-3″, but through the accident her sides had bulged out beyond the width of the dry dock entrance. She was raised in the following spring, although further damaged by ice during the winter, and brought on the beach a short distance below the dry dock, where she was sold as scrap. This is the worst case on record in the history of the St. Lawrence navigation. This vessel was only six years old and of a registered tonnage of 10,387 tons.

In the summer of 1898 the writer was instructed to prepare a report on the practicability of widening the entrance of the Lorne Dry Dock, which had been completed in 1886. A plan was submitted, showing the possibility of obtaining an entrance 70 feet wide, by removing part of the timber slides at the outer end of the dock; increasing the length was also suggested. The first was reported to be inadvisable as it would greatly disfigure the dock and do away with the convenience of the timber slides; the only feasible way would be to remove and rebuild in another position the eastern side wall, thus depriving the harbour of all dock accommodation for probably two seasons. A new caisson wold necessarily have to be provided; the cost would have been considerable. Further, it was considered that a new dry dock would be required in Quebec before many years.

The suggestion of lengthening the dock was adopted; the length was increased from 484 feet to 600 feet; this consisted merely in moving the circular head, stairways and timber slides 116 feet further, after excavating the rock to proper width and depth. The work was performed under contract awarded in the year 1900, for the sum of $100,000.00, and completed in 1901 without interfering with the use of the dock. The details of construction of this dry dock have already been described in a paper read before the Canadian Society of Civil Engineers some years ago, by Mr. St. George Boswell, Chief Engineer of the Quebec Harbour Commission, who was assistant engineer during the construction. This dry dock was built by the Quebec Harbour Commissioners under an Act, 38 Vict. Cap. 56–1875, by which the issue of bonds was allowed to obtain the necessary amount. The work was started in 1878 and completed in 1886 at a total cost of $921,130.

In 1888 the Canadian Government relieved the Harbour Commission of all obligations to refund the principal sum or interest expended on the dry dock and in 1890 it was placed under the control of the Department of Public Works, the writer was then placed in charge.

NEW DRY DOCK

In 1906 the Quebec Board of Commissioners urged upon the Government the necessity for a large dry dock for the harbour of Quebec. In the fall of the same year the writer was instructed to make a survey of the locality surrounding the old dry dock and report on the best location. Two sites were examined, but the position to the east of the present dock was considered the most advantageous for three principal reasons:

A larger area of land could be acquired.

A better foundation could be obtained.

The repairing plant of Messrs. G. T. Davie & Sons could have better access to both the new and old docks.

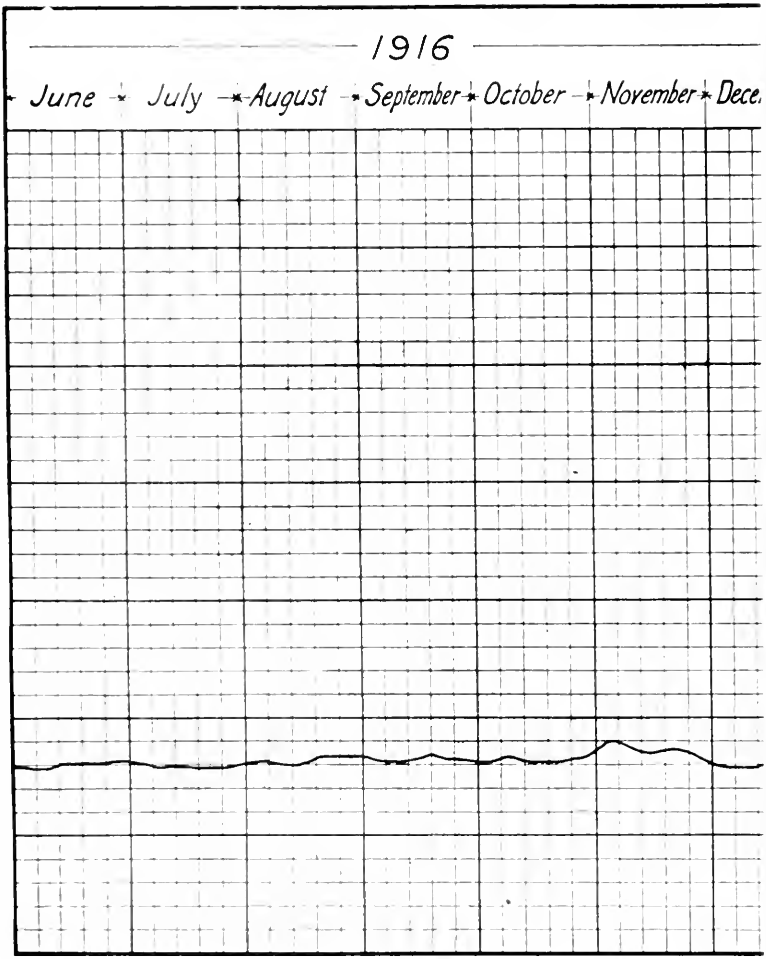
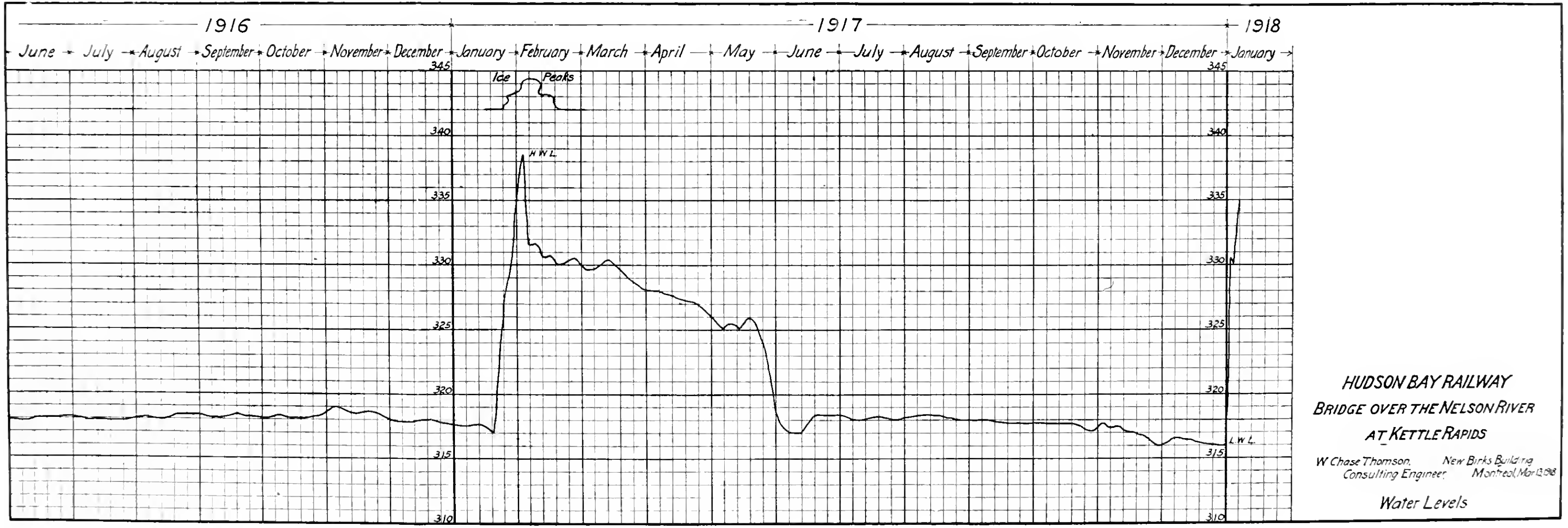


PLATE I



1916
1917
1918
June
July
August
September
October
November
December
January
February
March
April
May
June
July
August
September
October
November
December
January
345
340
335
330
325
320
315
310
Ice
Peaks
H W L
L W L
HUDSON BAY RAILWAY
BRIDGE OVER THE NELSON RIVER
AT KETTLE RAPIDS
W Chase Thomson
Consulting Engineer
New Birks Building
Montreal
Water Levels

PLATE I

4

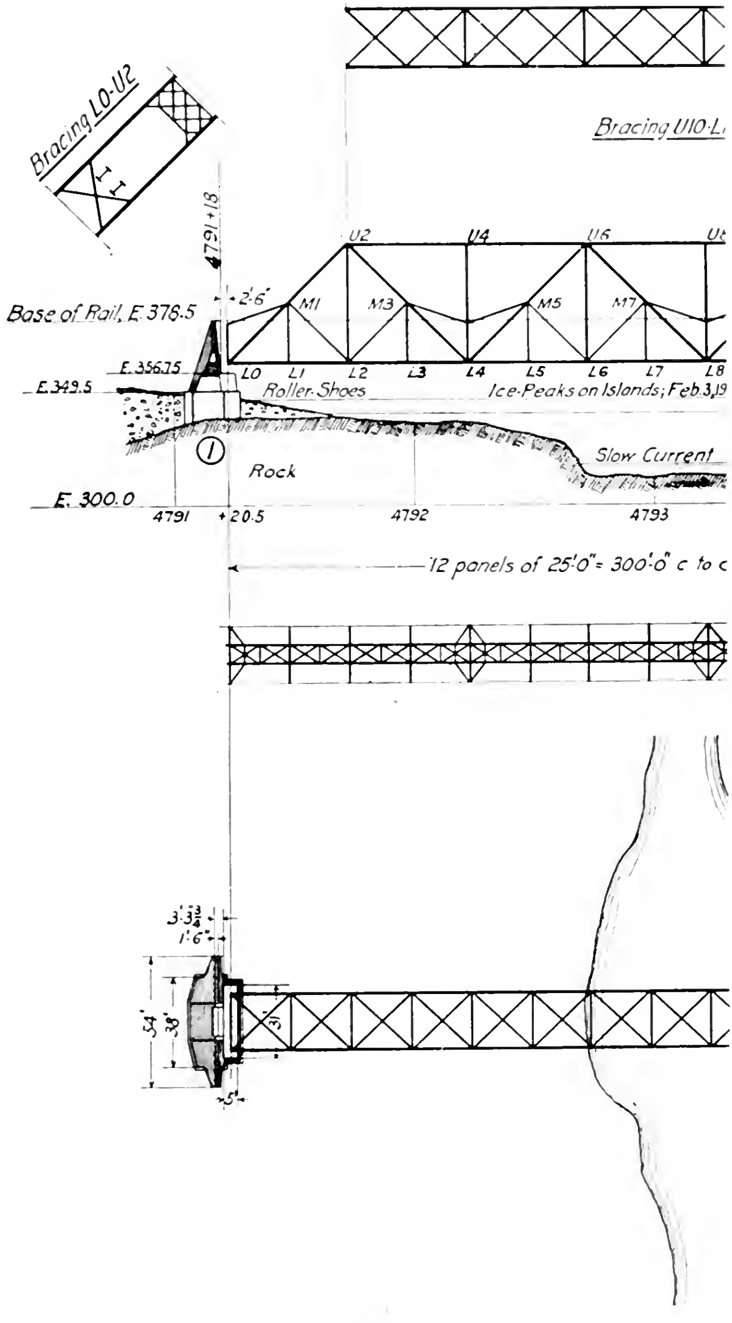

Bracing L0-U2
Bracing U10-L
4791+18
U2
U4
U6
M1
M3
M5
M7
2'6"
Base of Rail, E. 378.5
E. 356.75
E. 349.5
L0
L1
L2
L3
L4
L5
L6
L7
L8
Roller-Shoes
Ice-Peaks on Islands; Feb. 3, 19
Slow Current
Rock
E. 300.0
4791
+20.5
4792
4793
12 panels of 25'0" = 300'0" c to c
3'3¾"
1'6"
54'
38'
31'
5'

PLATE II

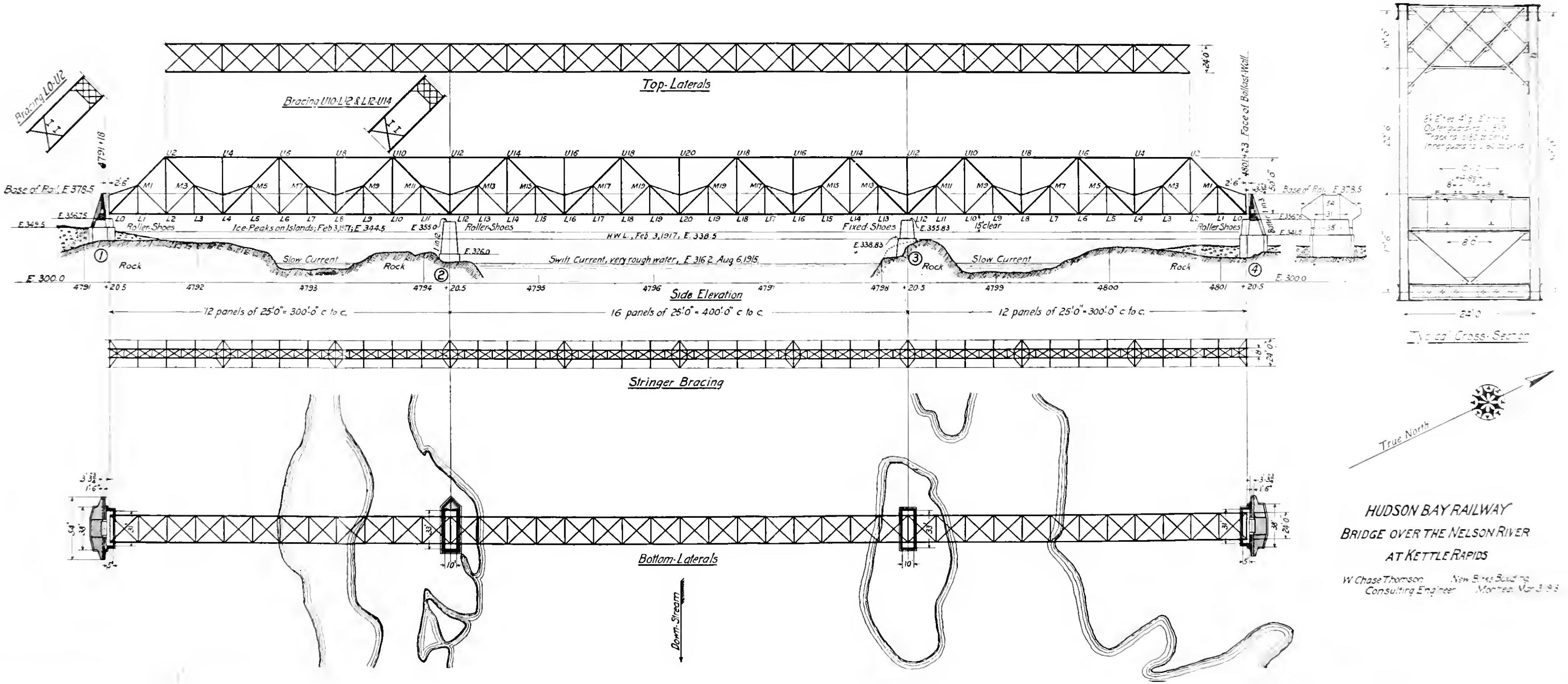

Top-Laterals
Side Elevation
12 panels of 25'-0" = 300'-0" c to c.
16 panels of 25'-0" = 400'-0" c to c.
12 panels of 25'-0" = 300'-0" c to c.
Stringer Bracing
Bottom-Laterals
Typical Cross-Section
True North
Down-Stream
HUDSON BAY RAILWAY
BRIDGE OVER THE NELSON RIVER
AT KETTLE RAPIDS
W Chase Thomson
Consulting Engineer

PLATE II

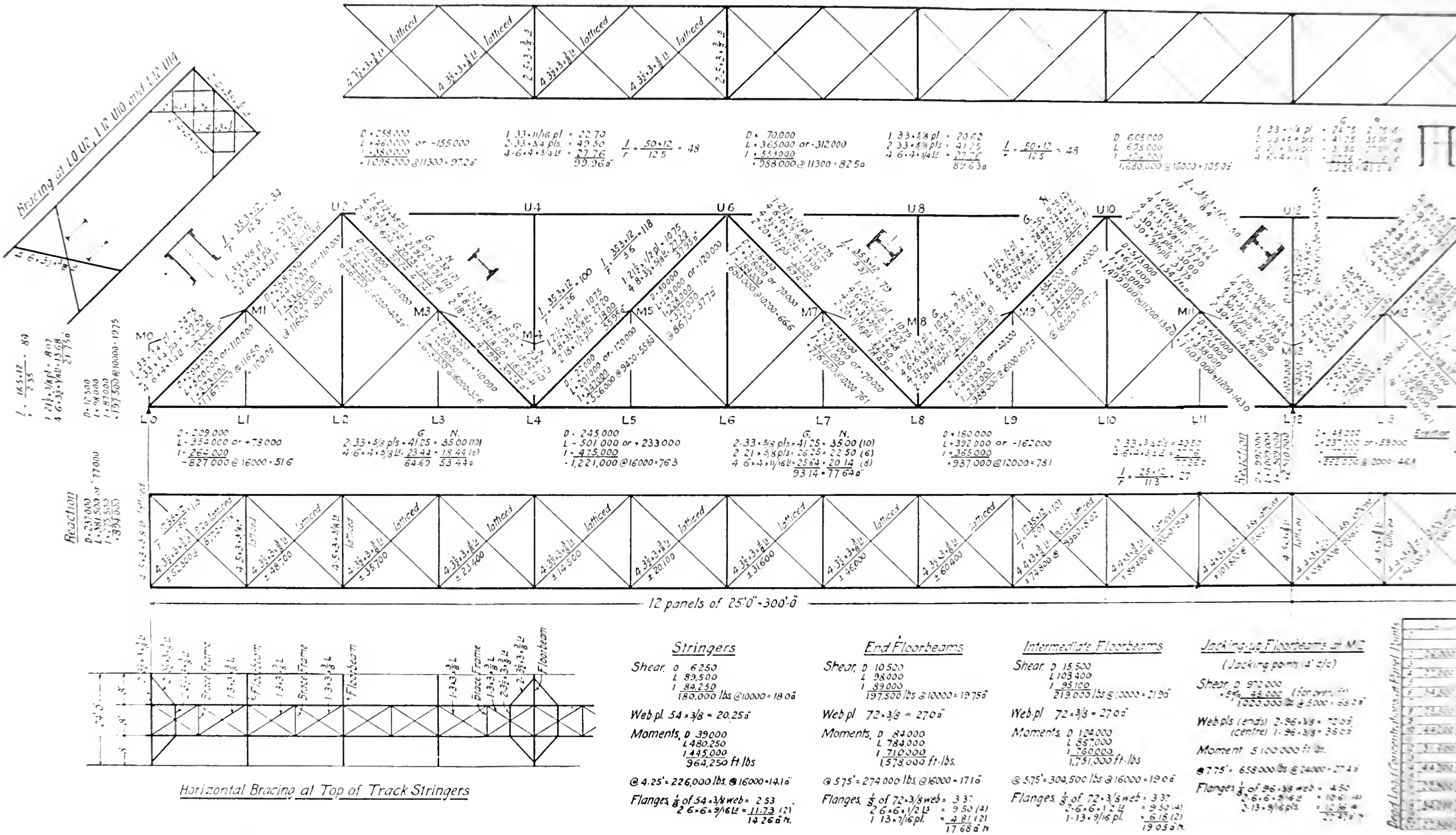
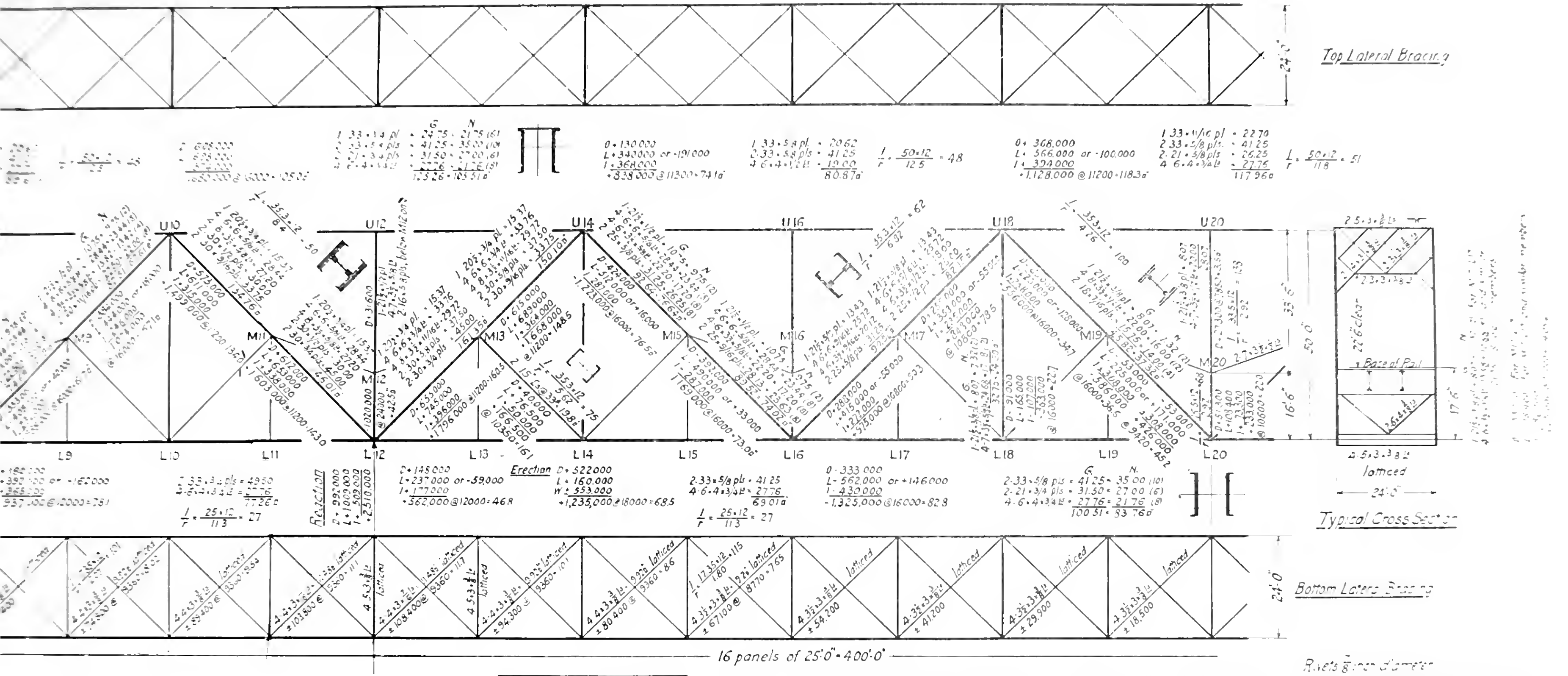

Bracing at L0 U2, L12 U10 and L12 U14
M0
M1
M3
M4
M5
M7
M8
M9
M11
M12
U2
U4
U6
U8
U10
U12
L0
L1
L2
L3
L4
L5
L6
L7
L8
L9
L10
L11
L12
Reaction
12 panels of 25'0" = 300'0"
Stringers
Shear. D 6 250
L 89 500
I 84 250
180 000 lbs @ 10000 = 18.0 □"
Web pl. 54 × 3/8 = 20.25 □"
Moments. D 39 000
L 480 250
I 445 000
964 250 ft-lbs
@ 4.25' = 226 000 lbs @ 16000 = 14.1 □"
End Floorbeams
Shear. D 10 500
L 98 000
I 89 000
197 500 lbs @ 10000 = 19.75 □"
Web pl. 72 × 3/8 = 27.0 □"
Moments. D 84 000
L 784 000
I 710 000
1 578 000 ft-lbs
@ 5.75' = 274 000 lbs @ 16000 = 17.1 □"
Intermediate Floorbeams
Shear. D 15 500
L 103 400
I 95 100
219 000 lbs @ 10000 = 21.9 □"
Web pl. 72 × 3/8 = 27.0 □"
Moments. D 124 000
L 857 000
I 760 000
1 751 000 ft-lbs
@ 5.75' = 304 500 lbs @ 16000 = 19.0 □"
Jacking-up Floorbeams at M12
(Jacking points 4' c/c)
Horizontal Bracing at Top of Track Stringers

PLATE III



Intermediate Floorbeams

Shear D 15,500
L 103,400
I 95,100
219,000 lbs @ 10000 = 21.9 □"

Web pl 72 × 3/8 = 27.0 □"

Moments D 124,000
L 857,000
I 760,000
1,751,000 ft-lbs

@ 5.75' = 304,500 lbs @ 16000 = 19.0 □"

Flanges 1/8 of 72 × 3/8 web = 3.37
2-6 × 6 × 1/2 L^s = 9.50 (4)
1-13 × 9/16 pl. = 6.18 (2)
19.05 □" N.

Jacking-up Floorbeams at M12

(Jacking points 14' c/c)

Shear D 972,000
+5% 48,000 (for overlift)
1,020,000 lbs @ 15000 = 68.0 □"

Web pls (ends) 2-96 × 3/8 = 72.0 □"
(centre) 1-96 × 3/8 = 36.0 □"

Moment 5,100,000 ft lbs.

@ 7.75' = 658,000 lbs @ 24000 = 27.4 □"

Flanges 1/8 of 96 × 3/8 web = 4.50
2-6 × 6 × 9/16 L^s = 10.61 (4)
2-13 × 9/16 pls = 12.36 (4)
27.47 □" N

Dead-Load Concentrations at Panel-Points

	U	M	L	Totals
0		12,500	17,100	29,600
1		20,000	30,400	50,400
2	26,000		36,900	62,900
3		8,700	29,900	38,600
4	27,000	24,500	19,100	70,600
5		8,200	32,400	40,600
6	34,300		41,000	75,300
7		12,300	32,300	44,600
8	25,500	24,000	24,400	73,900
9		12,700	31,400	44,100
10	44,200		40,100	84,300
11		22,800	31,400	54,200
12	31,600	30,500	35,200	97,300
13		25,000	31,400	56,400
14	44,700		40,100	84,800
15		14,200	31,400	45,600
16	22,900	24,000	26,600	73,500
17		13,800	33,700	47,500
18	34,200		42,200	77,400
19		8,200	33,800	42,000
20	27,300	24,000	22,000	73,300

Dead Load as per table of concentrations, which includes floor weighing 600 lbs per lin ft of bridge.

Live Load, 25,650 – 49,400 – 49,400 – 49,400 – 49,400 – 34,200 – 34,200 – 34,200 – 34,200 – 25,650 – 49,400 – 49,400 – 49,400 – 49,400 – 34,200 – 34,200 – 34,200 – 34,200; spacing 8' 5' 5' 5' 9' 5' 6' 5' 8' 8' 5' 5' 5' 9' 5' 6' 5' 5'; 4750 lbs per lin ft

Impact, $\frac{\text{range}^2}{\text{max.}}$, in which range = greater live-load stress + $\frac{4}{10}$ of lesser

Wind-Loads, 400 lbs. per lineal foot, applied at base of rail, considered as fixed load.
400 lbs per lineal foot, applied 8 ft. above base of rail, considered as moving load

Tension, 16,000 lbs. per sq. in.; Compression, $12{,}000 \div 1 + \frac{(\frac{l}{r})^2}{36{,}000}$ lbs. per sq. in.

Erection Conditions: Dead-Load as above; Derrick Car 150,000 lbs, trucks 35 ft c/c, with 79,000 lbs on front truck, placed at panel point 19, and 71,000 lbs. on rear truck; or Top Chord Traveller weighing 120,000 lbs., equal loads on trucks 50 ft c/c. Wind, 600 lbs per lin. ft., applied at base of rail. Unit-Stresses increased 50%.

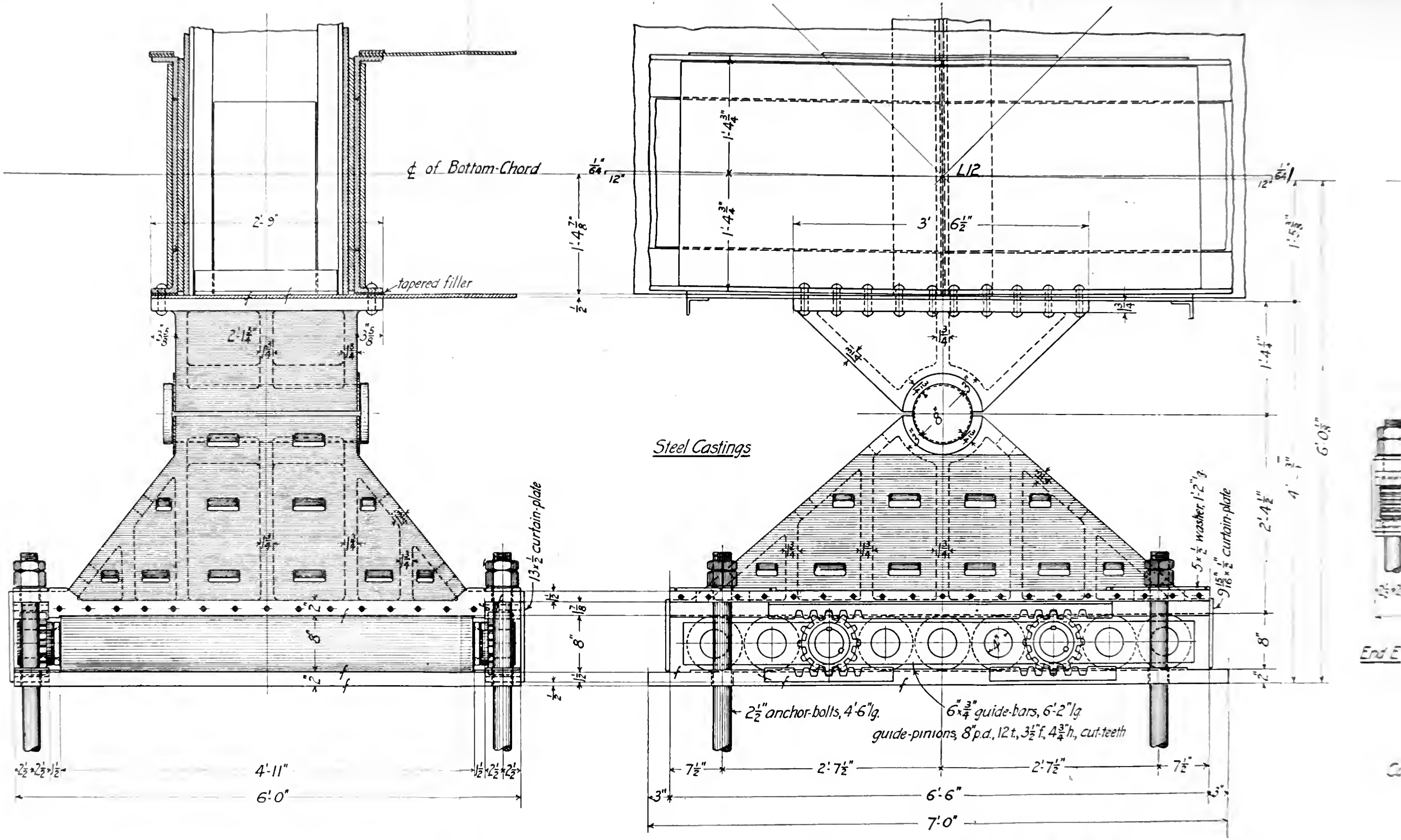
Rivets 7/8 inch diameter
Materials and Workmanship as per General Specification of the Department of Railways and Canals for 1908

HUDSON BAY RAILWAY

BRIDGE OVER THE NELSON RIVER AT KETTLE RAPIDS

Stress Sheet

W. Chase Thomson
Consulting Engineer
New Sun Life Building
Montreal May 1918

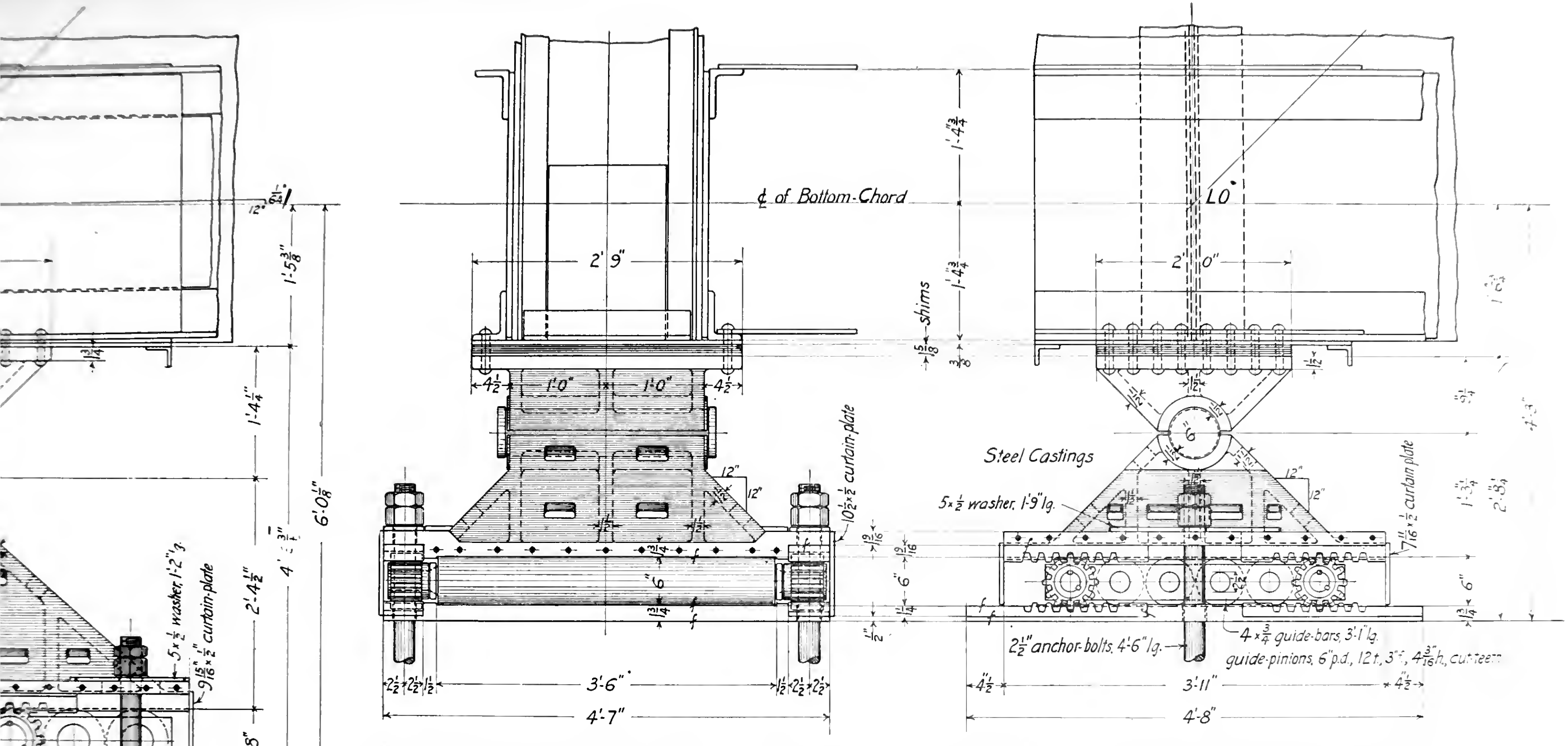


End Elevation, with End Curtain-Plate removed

Expansion-Bearings at Pier 2

Side Elevation, with Side Curtain-Plate removed

PLATE IV



End Elevation, with End Curtain-Plate removed

Expansion-Bearings at Abutments 1 & 4

Side Elevation, with Side Curtain-Plate removed

Castings filled with Concrete.

HUDSON BAY RAILWAY

BRIDGE OVER THE NELSON RIVER

AT KETTLE RAPIDS

Expansion-Bearings

W. Chase Thomson, Consulting Engineer. New Birks Building, Montreal, Mar. 13, 1916.

A plan and report were submitted in the early part of 1907; the dock then proposed was 1000 feet long with an entrance width of 100 feet.

The proposition was not immediately acted upon; the question as to whether the Government should build the dock or induce some shipbuilding firm to build it under a subsidy from the Government was unsettled. The result of the discussion was the passing of an Act of Parliament at the session of the year 1910, assented to on the 4th May, called an Act to Encourage the Construction of Dry Docks.

Under this Act the dry docks were divided into three classes. The first class included dry docks estimated to cost not more than four million dollars, and capable of receiving and repairing the largest ships of the British Navy and of the following dimensions:

(a) Clear length on bottom 900 feet; clear width of entrance 100 feet, with depth on sill at high water ordinary spring tides of 35 feet.

(b) Floating dry docks of a lifting capacity of 25,000 tons.

The second class included dry docks estimated to cost two and one half million dollars of the following dimensions:

(a) Clear length on bottom 650 feet; clear width of entrance 85 feet; depth of water on sill at ordinary high water spring tides 30 feet, if in tidal waters; or 25 feet on sill, if constructed in non-tidal waters.

(b) Floating dry docks of a lifting capacity of 15,000 tons.

The third class consisted of dry docks estimated to cost not more than one and one-half million dollars, of the following dimensions:

(a) Clear length on bottom 400 feet; clear width of entrance 65 feet; depth of water on sill at ordinary high water spring tides 22 feet, if in tidal waters; and 18 feet, if in non-tidal water.

(b) Floating dry docks of a lifting capacity of 3,500 tons.

The estimated cost in all cases includes the totally equipped repairing plant capable of effecting all sorts of repairs, including machine shops and tools, foundry, administration buildings, etc., together with the dock itself, but does not include marine slips or other installation used in the construction of ships.

According to the Act, the subsidy on dry docks of the first class is 3½ per cent per annum on the estimated cost for a period of 35 years from the time it has been reported that the dry dock is entirely completed. The subsidy on the second class is 3½ per cent per annum for 25 years from the time of completion. On the third class, the subsidy is 3 per cent for a period not exceeding 20 years from the time of completion.

In all cases the Company making the application must furnish plans with a detailed list of the plant and a complete estimate of the cost. These are revised and corrected, if found advisable; and, upon a report from the Chief Engineer of the Department of Public Works that the works intended to be built are in the public interest, the application is granted upon certain conditions of management and maintenance. The works are to be executed under the superintendence of an officer of the Department.

The above Act was amended in April 1912, by making the length of the first class dry docks 1150 feet, the entrance 110 feet and the estimated cost five and a half millions.

Another amendment was made in May 1914, by which the subsidy of four per cent on the estimated cost is allowed for first class dry docks.

The Act was further amended in 1917, by which the dimensions of the first class dry docks shall be: length on bottom 1150 feet; width of entrance 125 feet; depth on sill at high water spring tides 38 feet. A subsidy of $4\frac{1}{2}$ per cent on the estimated cost of five and a half million dollars is allowed, payable half-yearly for a period of 35 years from the time of completion. By this amendment no bonds or debentures are to be issued until one million dollars shall have been expended on the construction of the dry dock.

After the passing of the Act of 1910, shipbuilding firms were invited to build a dry dock at Lauzon, in the Harbour of Quebec, under the subsidy Act of that year. Two companies submitted plans and offered to build under contract without reference to the subsidy Act. In 1912 another company submitted plans for a dry dock to be built on the Quebec side of the harbour, just below the mouth of the St. Charles River, according to the subsidy Act, as amended in 1912. Some objection having been made to this location and with no prospect in view for any other applicant, the Department of Public Works decided that a dry dock would be built by the Government.

THE NEW DOCK

In the early part of 1913 the writer was instructed to prepare plans and specifications on which tenders could be called as soon as possible for the construction of the new dry dock, the location being to the eastward of the Davie Shipbuilding yard, so that both the old and new dry docks would be easily accessible from the shops.

Tenders for the construction of this work were advertised on the 12th May 1913, to be received on the 30th June following. The contract was awarded to the lowest tenderers, Messrs. M. P. & J. T. Davis. The contract was signed on the 7th October, 1913.

The new dock was at first intended to be built on a line parallel to the old dry dock, but this was objected to from the point of view of navigation. A commission was appointed in the fall of 1913 to investigate and find out which direction would best suit the entrance facilities, and it was decided that the centre line of the dock should form an angle of 69° with the direction of the old dry dock, or approximately 45° N. E. and it was so laid out.

Owing to the limited time available before the calling of tenders, general plans only were prepared, together with an estimate of the cost. The requirements as to details for the machinery and caissons were stated in the specification; the contractors were requested to furnish during construction all detail plans, to be submitted for approval by the Department.

The dry dock has the following general dimensions. Total length from outer caisson to head wall 1150 feet, divided into two compartments. Outer part 500 feet; Inner part 650 feet.

Width of entrance	120	feet.
Width at coping	144	"
Width on floor	105	"
Depth on sill at high water S.T	40	"
Depth on sill at low water, S.T	22	"
Spring tides rise	18	"
Coping of side wall above high water S.T	7	"
Floor at outer end below outer sill	4½	"
Slope of floor transversely	1 in 100.	
Western guide pier	400	feet.
Eastern guide pier	500	"
Depth in entrance channel at low tide	30	"

The land expropriated in connection with the construction of the dry dock has a superficial area of 25½ acres, of which 11½ acres are reclaimed beach land.

The outer entrance of the dock is closed with a rolling caisson, the top of which is provided with an automatic folding bridge; a floating caisson closes the inner entrance. This caisson can also be placed to close the outer entrance in cases when repairs are required to be made to the rolling caisson.

Three main centrifugal pumps each of 63,000 gallons per minute capacity are used to empty the dock; two pumps of 6,000 gallons per minute each are used to keep the dock dry. All pumps are run by electric power. Eight boilers of a total capacity of 3,600-horse-power furnish the steam at 200 lbs. pressure to run the three direct current turbo generators of 1,500, 750 and 300 kilowatts respectively, which furnish the current at 550 volts to run the pump and other motors.

A direct current generator of 100 kilowatts at 220 volts, driven by a steam engine, will furnish the current for the lamps around the dock and in the buildings. There are 24 lamps of 500 watts, hung from poles around the dock. The poles are made of gas pipe, with the lower end set into sockets fitted with electric connections, and made removable in case of necessity. All electric wiring for lamps and motors outside of the buildings is placed underground.

The approximate quantities of the materials in the principal items entering into the construction are:

Rock excavation above and below coping	342,000	c. yds.
Submarine rock excavation in channel	65,000	"
Dredging entrance channel	530,000	"
Concrete	100,000	"
Granite steps, altars and quoins	140,000	c. ft.
Steel beams, reinforcing bars and manhole covers	150,000	lbs.
Cast iron for roller casings and sluice valves	125	tons.
Cast steel for caisson rollers	65	"
Gun metal for caisson roller and valves	4,500	lbs.
Cast iron in keel blocks and bollards	990	tons.
Forged steel spindles for rollers	11,000	lbs.
Bricks for chimney and flues	345,000	
Fire bricks	125,000	
Cribwork in approach piers	63,300	c. yds.
Concrete in approach piers	13,300	"
Steel in rolling caisson	930	tons.
Total weight in rolling caisson and machinery	1,125	"
Steel in floating caisson	960	"

The work was started in May 1914. The concrete retaining walls on each side of the dock, specified to be built from the natural rock surface to elevation +24 and intended to prevent seepage through the filling, were completed during the season's work, as well as the cofferdam between the outer ends of these walls. Rock drilling in the prism of the dock was also carried on in the part not affected by tides. The largest part of the drilling was done by two well drillers, the holes being sunk down to grade and plugged for future blasting. The average depth of perforation for each drill was about 80 feet per day, although as much as 130 feet was done occasionally. Ten or twelve ordinary steam drills were also used on the work.

The rock consisted of hard shale, irregularly stratified, at an angle of about 45°. Considerable rock slides occurred on the west side of the cut, which necessitated a much larger quantity of concrete for the dock wall on that side, also the use of rock bolts, to prevent the sliding tendency of this wall.

Steam shovels and dump cars were used to remove the blasted rock, which was used for filling, wherever required, on the Government property.

The cofferdam was built of timber cribwork, 20 feet wide, sunk in an average depth of one foot of water, at low tide, and built to the elevation of three feet above high tide; a layer of concrete was deposited along the bottom of the outer face and this face was sheathed with plank.

The floor and walls of the dock are built of concrete, the mixture being 1–3–5. All exposed faces are finished with a fine concrete of 1–2–4 mixture for a thickness of six inches. The concrete for the walls and the floor was cast in alternate sections of approximately 30 feet, with expansion joints.

All the cement used was subjected to a laboratory test; apart from other requirements the tensile strength was required to be 600 lbs. per square inch after 27 days immersion, for neat briquettes, and 275 lbs. per square inch for 1–3 mixture.

The steps at the top of the walls are built of granite, with treads and risers of 12 inches; the altars are 2 feet 6 inches wide and consist of granite 12 inches thick tailing 9 inches into the concrete. The caisson stops of both entrances and all culvert openings are built of granite.

The floor is 5 feet thick and finished level from end to end; the sides slope down 6 inches to the side gutters. The floor is provided with three strips of granite slabs, 18 inches thick, intended to receive the cast iron keel and bilge blocks. The middle strip is 10 feet wide and level; the side strips are 9 feet wide.

In order to prevent the possibility of hydrostatic pressure under the floor and behind the side walls, a system of drains is provided, that will take the seepage water to the sumps.

There are twelve stairways from the top of the walls to the floor of the dock, two at each end of the two compartments and two half-way between the ends of each compartment. Four timber slides, built of granite slabs, 18 inches thick, are provided alongside the last set of stairways. There are also eight ladders, four on each side of the dock, that may be used to reach the floor. These are built with galvanized iron gas pipe, and set in recesses in the walls.

The coping of the walls stands at elevation + 25, or 7 feet above high tide. They are provided with the ordinary cast iron bollards, set in concrete blocks, 60 feet apart. There are nine electrically driven capstans with 15-horse-power motors, four on each side of the dock and one at the head.

The keel blocks are each built of three pieces of castings; the middle piece being wedge shaped so that it may be knocked out and the block removed from under a ship, when in the way of repair work; the upper part of the top piece of casting is provided with a piece of white oak tenoned into the casting. All rubbing faces are planed true and smooth. The keel blocks are 4 feet 4 inches long and 2 feet 3 inches high. On top of these are placed temporary hard wood timber blocks to obtain the required height above the floor. It had been intended to build bilge blocks, so arranged as to slide under the bilge of vessels. However, this was objected to by the British Admiralty, who insist on having all blocks made of the same pattern, so as to enable building a bed that will conform to the bottom of the vessel.

CAISSONS

The outer entrance is closed by a rolling caisson built of steel and operated by an electric motor of 125-horse-power; the bottom is provided with two heavy scantlings of steel, resting on flanged rollers, 3 feet in diameter, placed at 8 feet centres. These are made of cast steel and bored to receive bronze bushings. The forged steel spindles, 4 inches in diameter, are also provided with bronze sleeves. The cast iron casings, containing the rollers, are set in the concrete alters, on each side of the caisson berth and chamber. At an elevation of 15 feet 9 inches above the sill of the dock the rolling caisson is provided with 6 culverts, 42 inches in diameter, closed by sluice valves that are operated from the upper deck by a 15-horse-power electric motor, driving a longitudinal shaft provided with the necessary gearing; and, by means of clutches, any one or all of the valves may be worked. The culverts are used for flooding the dock. The caisson is divided horizontally by a water-tight deck at the elevation of 23 feet 6 inches above the bottom, forming the ballast and tidal chambers. As the tide rises the sea water comes on this deck through valves in the outer face of the caisson, which are kept constantly open during the summer to prevent the caisson from floating. A sufficient quantity of ballast is provided, so that the total weight of the structure resting on the rollers is approximately 150 tons. During the winter, when the dock is not in operation, the lower or ballast chamber of the caisson is filled with water, which is kept from freezing by a constant jet of steam. The tidal chamber is then kept dry by closing the valves. The caisson is closed and opened with heavy chains, supported on alters on each side of the caisson recess, and passing over pulleys worked by worm gears connected with the motor. The top of the caisson is provided with a folding bridge for light traffic across the dock; as soon as the caisson starts to open, the apron and railings of the bridge are automatically lowered to allow them to pass under the flooring over the caisson recess.

The middle entrance of the dock is closed by an ordinary floating or ship caisson. When in place, the deck is used as a bridge across the dock. This caisson may also be used to close the outer entrance by placing it immediately outside the rolling caisson, where the necessary stop is provided for it. This, however, will be necessary only in cases of repairs being required to the submerged parts of the rolling caisson.

These caissons were built by the Dominion Bridge Company, under a subcontract. The mode of construction and other particulars were fully described in a paper read before the Canadian Society of Civil Engineers by Mr. L. R. Thomson, A.M.Can.Soc.C.E. Volume 30, Part 1, 1916.

BOILERS AND ELECTRIC POWER

Six water tube boilers of 500-horse-power and two of 300-horse-power furnish steam at 200 lbs. pressure to produce electric current. The boilers are provided with automatic stokers, ash and coal conveyors. The coal is unloaded from cars into a coal crusher run by an electric motor, and elevated to a hopper of 500 tons capacity, over the front of the boilers. Water heaters are provided, but the steam is not superheated; one of the small boilers will be constantly under steam pressure to run the drainage pumps and the lighting dynamo.

The electric power consists of three direct current turbo-generators of 550 volts, one of 1,500 kilowatts, one of 750 and one of 300 kilowatts. The steam turbines are of the Curtis condensing type, built by the General Electric Company. In the large unit the turbine runs at 3,600 r.p.m. It is geared down to 360 revolutions for the generator; the second is geared from 5,000 to 750; the third is geared from 5,000 to 900 r.p.m. A 100-kilowatt generator driven by a high speed direct connected steam engine, furnishes the current for lighting purposes.

This power installation is more than ample for all the machinery connected with running of the dock proper. It is, however, anticipated that the whole of it will be used when large repairing and shipbuilding shops are in operation together with the pumping of the dock.

This electric installation has been criticised on the ground that the large expenditure is not justified when electric current is available from private companies in the vicinity of Quebec. When the electric installation was proposed by the writer the idea in view was that no company would be interested or willing to furnish over 3,000-h. p. at any time of the day or night for the short period of about 50 hours in the year, without interfering seriously with their general service. It had also been ascertained by personal visits to five of the principal Navy Yards of the U. S. Government that each of them has provided its own electric power for pumping their dry docks. Out of five, only one had installed alternating current machinery. It has developed since that the only electric company that could furnish the power current is not willing to entertain the proposition unless at a much greater cost to the Government than the private installation can be run, including the interest on the outlay, which is approximately $240,000.00.

PUMPS

The dock is emptied by three main pumps of the horizontal centrifugal type, each having a capacity of 63,000 gallons per minute. The bronze shafts are connected to the armature shafts of 800-horse-power motors, running at 750 revolutions per minute. The motors are built to stand an overload of 25 per cent for two hours; the total lift will very rarely be more than 33 feet. The suction and discharge pipes are 48 inches; the water is discharged into a chamber provided with non return valves, and to a culvert through the entrance wall outside of the caisson. The main pumps are guaranteed by the builders to deliver 63,000 gals. per minute against a total head of 25 feet. At the time of writing these pumps have not been tested as to efficiency.

Two auxiliary pumps each of 6,000 gallons per minute capacity, driven by electric motors of 125-horse-power will take care of leakages and seepage; these pumps will also help while the dock is being pumped. The pumps were manufactured by the Allis-Chalmers Company.

The time occupied in emptying the dock will vary according to the height of tide when the pumps are started and the size of the vessel being docked. At high water of spring tides the dock contains over 38,000,000 gallons of water. This quantity of water, however, will very rarely, if ever, exist, when pumping is started. It is estimated that the average time for pumping out the dock will be about two and a half hours.

Underground culverts 9 x 10 feet convey the water from the sumps in each compartment of the dock to the pumps; these culverts are provided with sluice gates, so as to permit of operating each compartment separately. The gates are operated from coping-level by 15-horse-power electric motors. The pressure against the gates may at times be due to a head of 50 feet of water.

From the non-return valve chamber the discharge culvert is 7 x 12 feet; it is also provided with a sluice gate. The capacity of discharge of this culvert was obtained from Chezy's formula $V = c\sqrt{rs}$, being obtained from Kutter's formula. Under a head of 4 inches the capacity will be ample to take care of the output of the pumps when discharging in open air.

The dock is filled through the six culverts in the outer caisson, each having a sectional area of nine square feet, also two culverts, one in each side wall of a sectional area of 30 feet, the valves of which are operated by electric power. These culverts are made exceptionally large due to the fact that each may only be partially opened until the water in the dock has reached the centre of the culvert opening, to prevent the heavy current that would result from a large opening from disturbing the beds prepared to receive a vessel; further, as the head between the outer and inner levels of water decreases, the valves are fully opened, thus obtaining a large flow. The time required to fill the dock may at times be as much as four hours. The middle entrance is similarly provided with filling culverts as the outer entrance.

In order to obtain sea water by gravity for the purpose of washing the floor of the dock, six-inch pipes were laid in the concrete side walls of the dock, at an elevation of two feet above low tide; each pipe has six hose-connections and valves at the face of the walls, where 50-foot lengths of 2½-inch hose may be attached for the purpose. The water is available within one hour of extreme low tide. Washing the floor is necessary owing to the sediment accumulated while the dock is flooded.

GUIDE PIERS

The western guide pier is 400 feet long and 75 feet wide; the one on the eastern side is 500 feet long, 75 feet wide at the outer and 200 feet wide at the inner end. Each is built of two lines of 12 x 12 timber cribwork substructure up to six feet above low water, spring tides; the outer face of each line of cribwork is built close, and sheathed vertically with 10-inch hardwood planks. The cribs facing on the channel were sunk in a depth of 30 feet at low water, spring tides; those on the eastern side of the east pier were sunk on the natural surface of the rock. Those on the western side of the west pier, as well as those for the landing pier, were sunk in a depth of 24 feet at low tide. From the elevation of six feet above low tide the superstructure consists of mass concrete walls, stepped at the back and filled between with excavated material. The railway spur track from the I.C.R. will be extended to the end of the western pier. These piers are intended to be used, when necessary, for unloading parts of cargoes from vessels to be docked. The entrance channel has a depth of 30 feet at low water, spring tides. The landing pier on the west side of the entrance is intended for unloading the dock supply of coal, when delivered by water.

BUILDINGS

The power-house is 120 x 100 feet, divided by a brick wall into two rooms, 120 x 50 feet, one being the boiler room and the other the generator room; the walls are solid brick, built on concrete foundation; the roof is built of reinforced concrete slabs, supported by steel I-beams, which were procured from the unused steel of the first Quebec Bridge. The building is provided with extra large windows with steel frames. Skylights and ventilators are also provided. The floor is concrete, overlaid with red tiles; and the lower part of the interior walls for the generator room is finished with a white tile wainscoting, 6 feet high. Each room is furnished with water closets and wash basins; the water is obtained from the Lauzon village aqueduct.

A special pump in case of fire and the necessary hose are provided. The generator room has an overhead travelling crane of 15 tons capacity. The lifting is done by motor; the travelling gear is worked by hand.

The pump-house is 70 x 47 feet, with foundation walls of concrete, over which solid brick walls are built. The floor is at elevation of 16 feet below low water, spring tides, or 41 feet below coping. It is finished with red tiles. The interior walls up to coping level are finished with white tiles. The pump-house is also provided with an overhead travelling crane of 10 tons capacity. The chimney is 180 feet high, built of brick, with an inner shell of fire-brick 100 feet high. There is an air space of 6 inches between the inner and outer shells; the inside diameter is 11 feet; the top consists of a cast-iron cap; four lightning-rods, well grounded, are provided to protect the chimney.

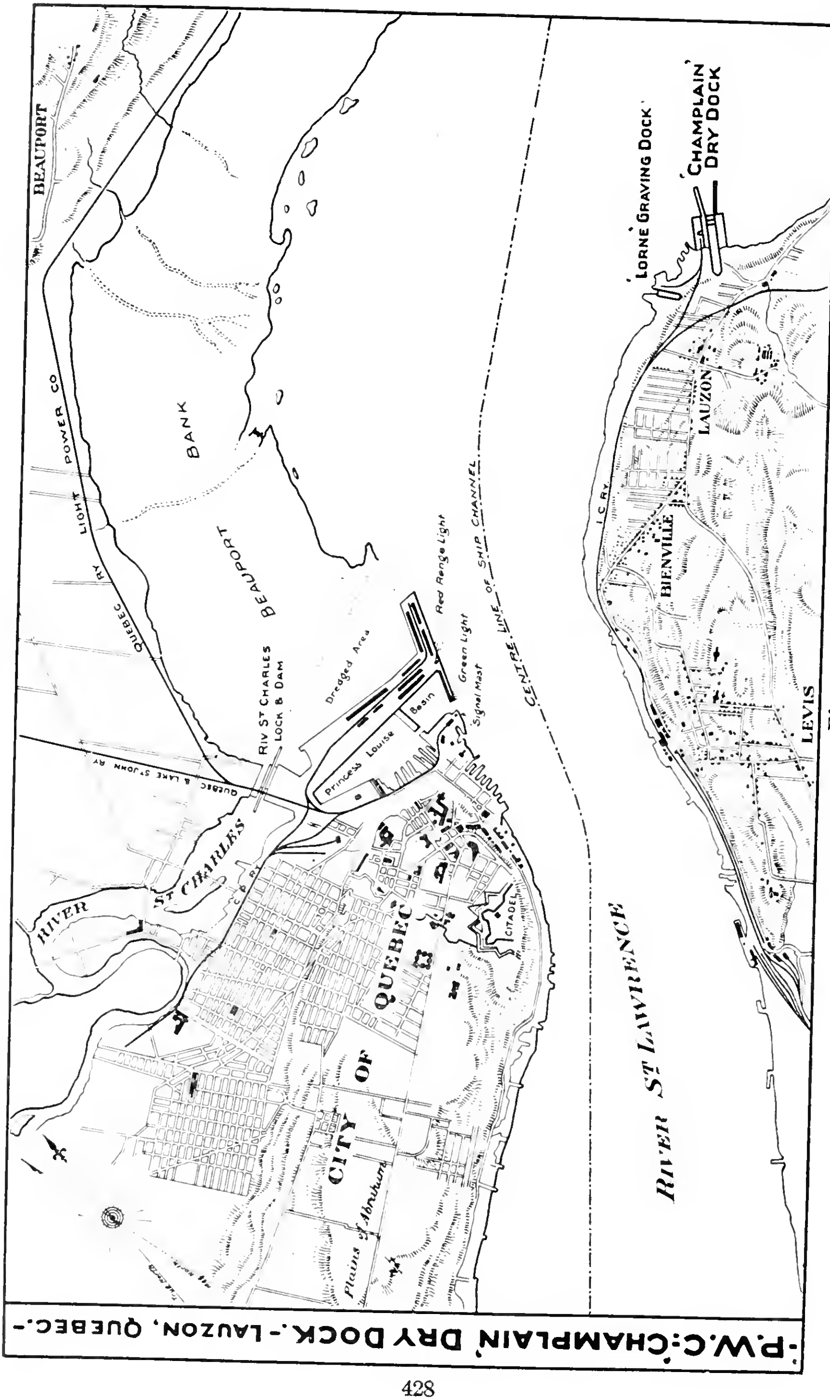
It may be stated that the length of the dock was decided on not merely in anticipation of vessels of, say, 900 feet or over being employed on the St. Lawrence trade, which may not happen for a great number of years, but owing to the great number of applications received every fall from owners of moderate sized vessels for accommodation during the winter, so that repairs may be done at cheaper rates, and the boats be ready for traffic as soon as navigation opens.

The dock is not yet quite completed: small portions of the floor and walls at the head remain to be finished; the boilers, machinery and pumps, although in working condition, require some final adjustment before they are tested and accepted;—the rolling caisson was operated in November last,—the contractors' floating plant was docked and the dock was pumped out. It is fully expected that everything will be entirely completed during the month of July next.

The several classes of works in connection with the construction of the dock have been accomplished in a thorough manner both in regard to materials furnished and workmanship; several minor changes which were found to be advantageous were made during construction. The contractors, in all cases, have shown their willingness to give satisfaction in every way irrespective of cost. It must be noted that the works were started shortly before the war and continued without interruption, except in winter, in spite of increased cost of materials and labour. The time required for the construction of the dock is somewhat over four years. It must, however, be remembered that the working season is only six months in each year,—concrete works have to be suspended during the first days of November and cannot be resumed until the beginning of May.

The total cost of the works under contract will be approximately $3,365,000.00.

The works have been carried on by the Department of Public Works with Mr. Eugene D. Lafleur, M. Can. Soc. C.E., as Chief Engineer,—the writer as Superintending Engineer,—Mr. J. K. Laflamme, A. M. Can. Soc. C.E., as Resident Engineer, — Mr. S. Fortin, M. Can. Soc. C.E., Steel Structural Engineer, has had the approval of plans submitted for the steel structures. The Contractors are Messrs. M. P. & J. T. Davis, and Mr. S. Woodard is their Superintending Engineer.



-P.W.C. 'CHAMPLAIN' DRY DOCK.- LAUZON, QUEBEC.-
'CHAMPLAIN' DRY DOCK
'LORNE' GRAVING DOCK
BEAUPORT
QUEBEC RY LIGHT POWER CO
BANK
BEAUPORT
RIV ST CHARLES LOCK & DAM
Dredged Area
Red Range Light
Green Light
Signal Mast
Princess Louise Basin
CENTRE LINE OF SHIP CHANNEL
QUEBEC & LAKE ST JOHN RY
RIVER ST CHARLES
C P RY
CITY OF QUEBEC
CITADEL
Plains of Abraham
RIVER ST LAWRENCE
I C RY
LAUZON
BIENVILLE
LEVIS

Fig. 1

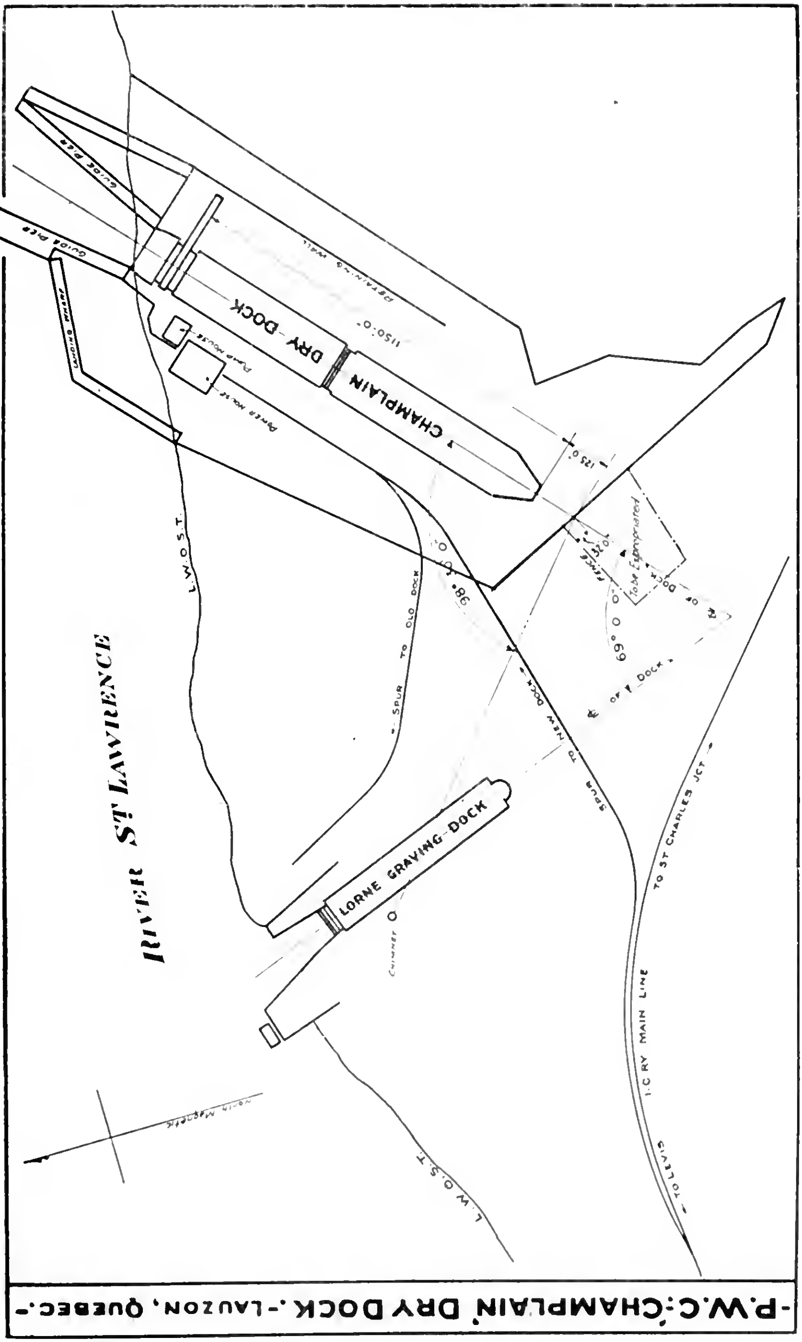
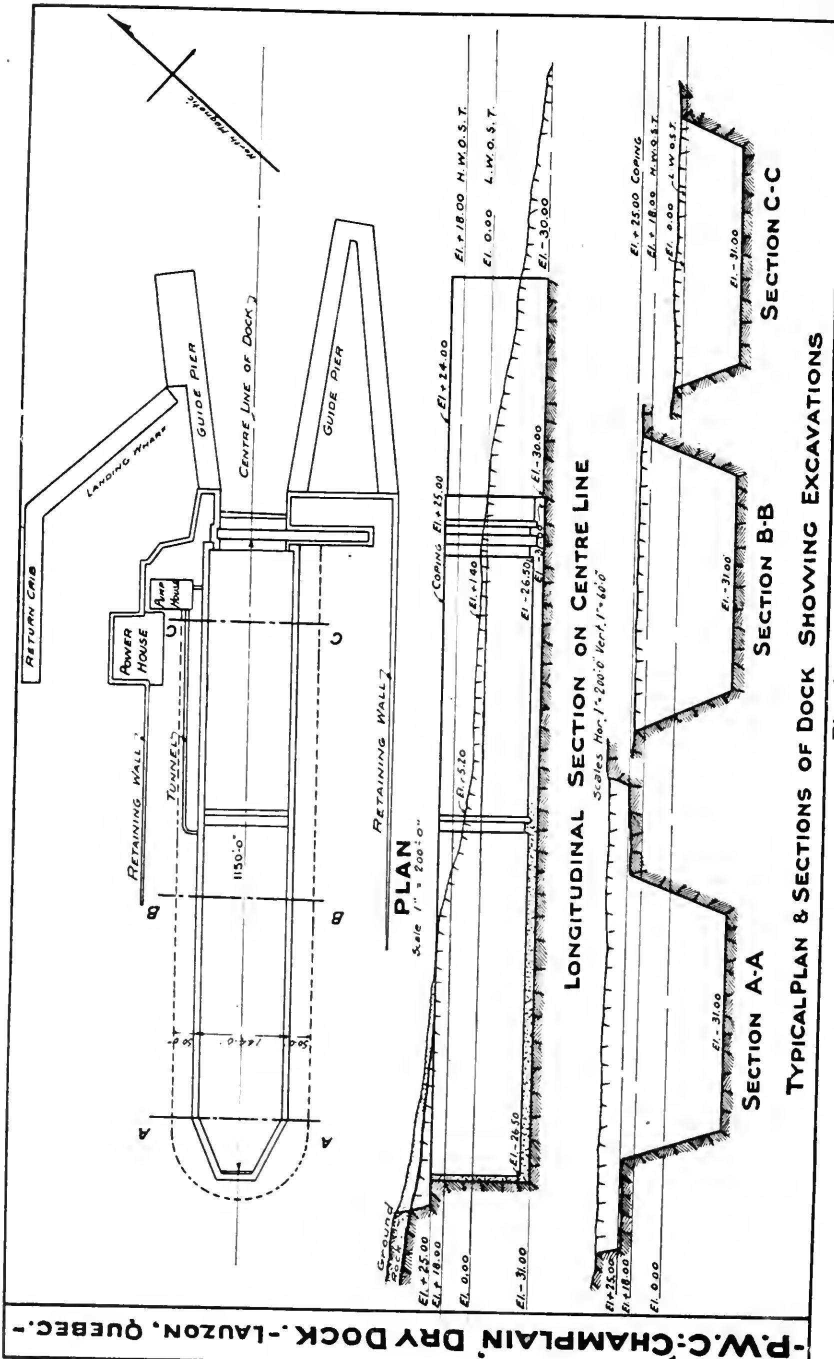
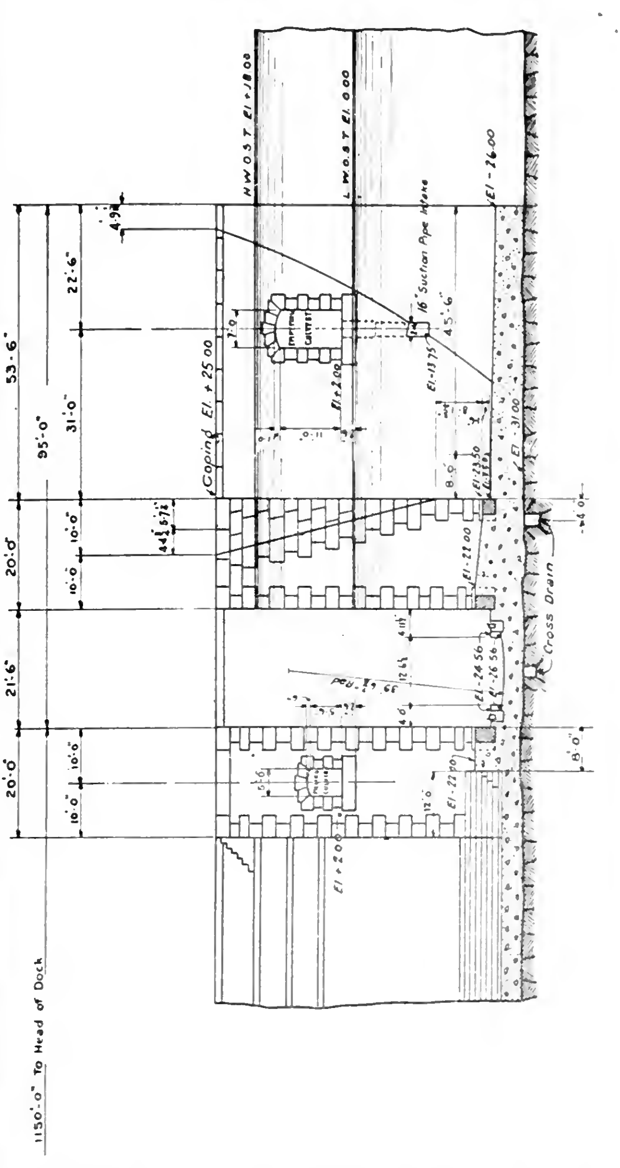

-P.W.C. "CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-
RIVER ST LAWRENCE
CHAMPLAIN DRY DOCK
1150'0"
125'0"
GUIDE PIER
RETAINING WALL
LANDING WHARF
L.W.O.S.T.
SPUR TO OLD DOCK
SPUR TO NEW DOCK
To be Expropriated
69°0'
LORNE GRAVING DOCK
I.C. RY MAIN LINE
TO ST CHARLES JCT
TO LEVIS
L.W.O.S.T.

Fig. 2



-P.W.C. "CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-
North Magnetic
Return Crib
Landing Wharf
Guide Pier
Guide Pier
Centre Line of Dock
Power House
Pump House
Tunnel
Retaining Wall
Retaining Wall
1150'-0"
144'-0"
PLAN
Scale 1" = 200'-0"
Longitudinal Section on Centre Line
Scales Hor. 1" = 200'-0" Vert. 1" = 60'-0"
El. + 18.00 H.W.O.S.T.
El. 0.00 L.W.O.S.T.
El. - 30.00
El. + 24.00
Coping El. + 25.00
El. + 1.40
El. - 5.20
El. - 26.50
El. - 31.00
Ground
Rock
El. + 25.00
El. + 18.00
El. 0.00
Section A-A
Section B-B
Section C-C
El. + 25.00 Coping
El. + 18.00 H.W.O.S.T.
El. 0.00 L.W.O.S.T.
Typical Plan & Sections of Dock Showing Excavations

Fig. 3

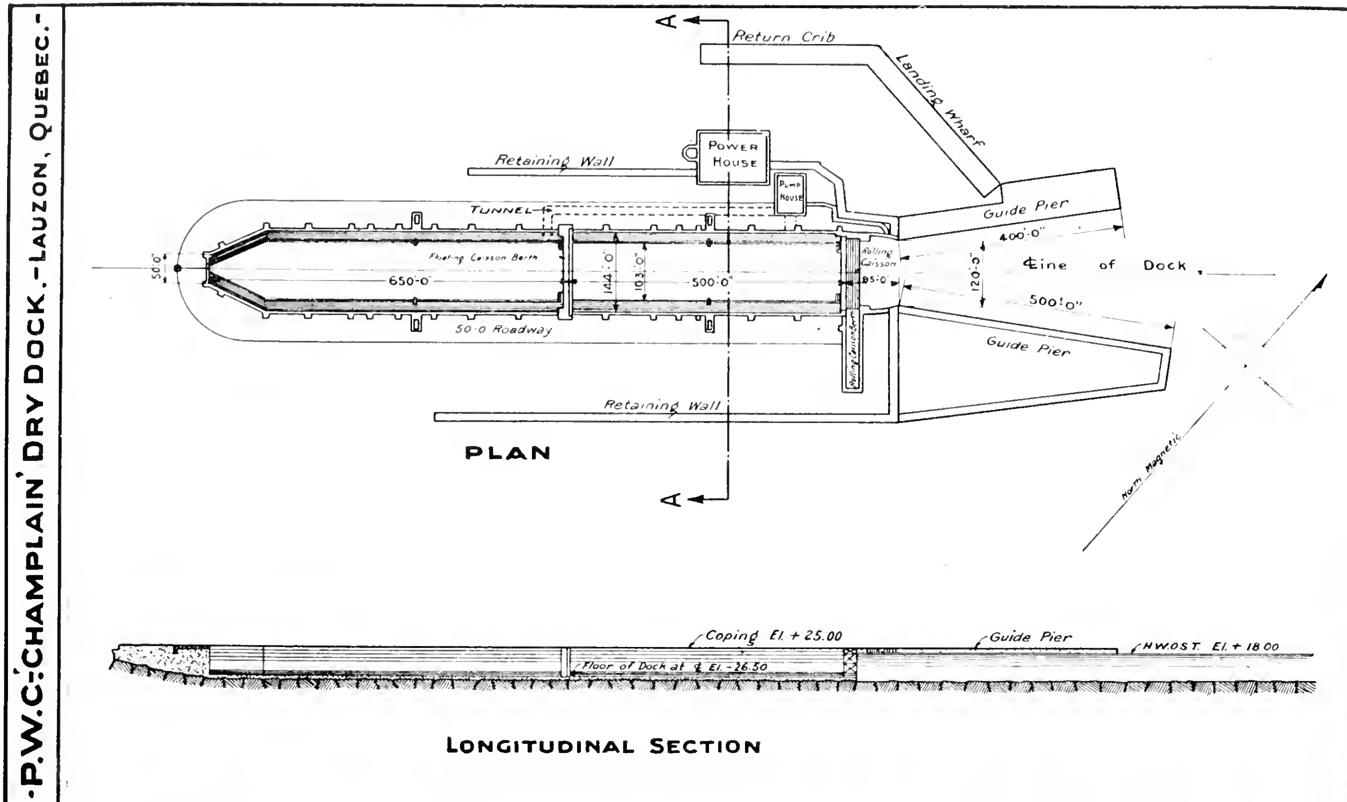


West Side

Part Longitudinal Section Showing Entrance

Fig. 4

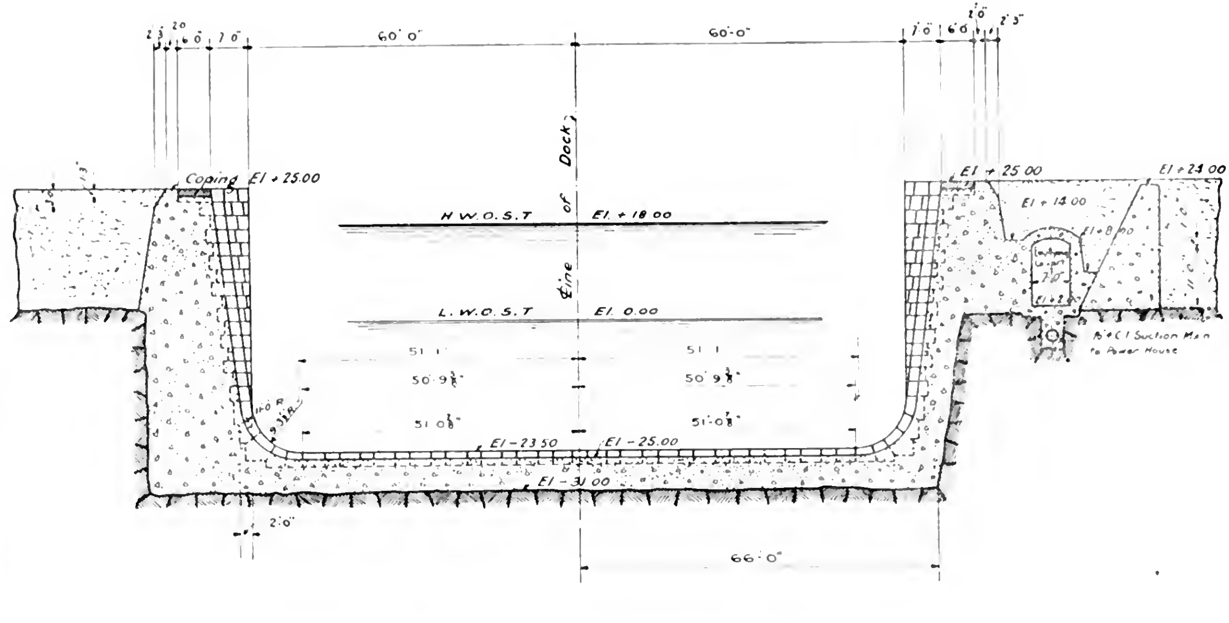
-P.W.C.-CHAMPLAIN DRY DOCK.-LAUZON, QUEBEC.-



P.W.C. "CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-
PLAN
LONGITUDINAL SECTION
Return Crib
Landing Wharf
Power House
Pump House
Retaining Wall
Retaining Wall
Tunnel
Guide Pier
Guide Pier
℄ Line of Dock
400'-0"
500'-0"
120'-0"
Rolling Caisson
85'-0"
Rolling Caisson Berth
Floating Caisson Berth
650'-0"
500'-0"
144'-0"
103'-0"
50'-0"
50'-0 Roadway
A
A
North Magnetic
Coping El. + 25.00
Guide Pier
H.W.O.S.T. El. + 18.00
Floor of Dock at ℄ El. -26.50

Fig. 5

-P.W.C.-CHAMPLAIN DRY DOCK.-LAUZON, QUEBEC.-



CROSS SECTION AT ENTRANCE SHOWING FLOATING CAISSON SILL.

Fig. 6

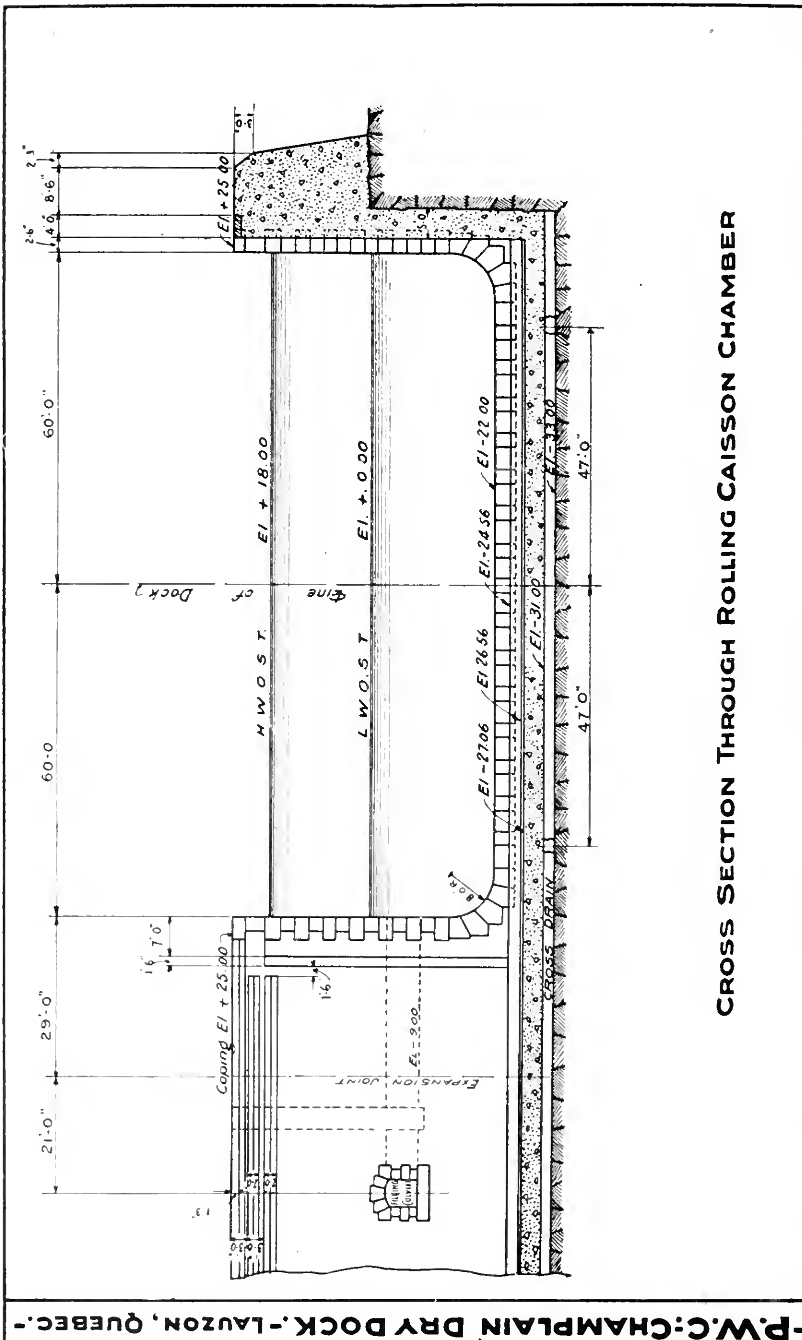
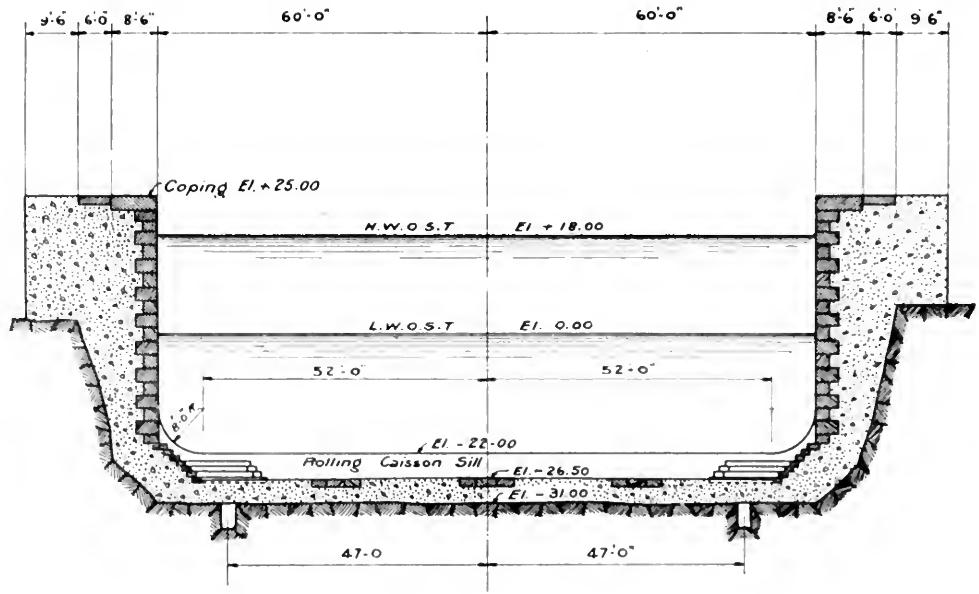

CROSS SECTION THROUGH ROLLING CAISSON CHAMBER
-P.W.C. "CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-
60'0"
60'0
29'0"
21'0"
47'0"
47'0"
El + 25.00
Coping El + 25.00
El + 18.00
El + 0.00
H W O S T
L W O S T
Line of Dock
El -22.00
El -24.56
El 26.56
El -27.06
El -31.00
El -33.00
EL -9.00
EXPANSION JOINT
CROSS DRAIN
FILLING CULVERT
8'0" R
7'0"
1'6"
1'6"
2'6"
4'0"
8'6"
2'3"
3'0"
3'0"
1'3"

Fig. 7

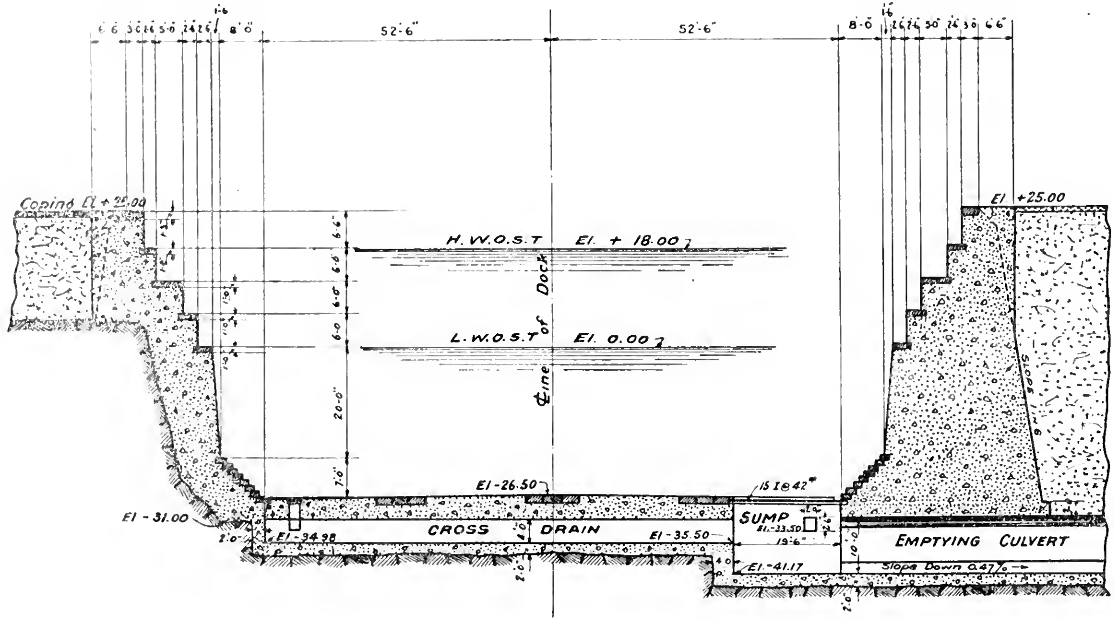
-P.W.C. "CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-



CROSS SECTION LOOKING NORTH - SHOWING ROLLING CAISSON SILL

Fig. 8

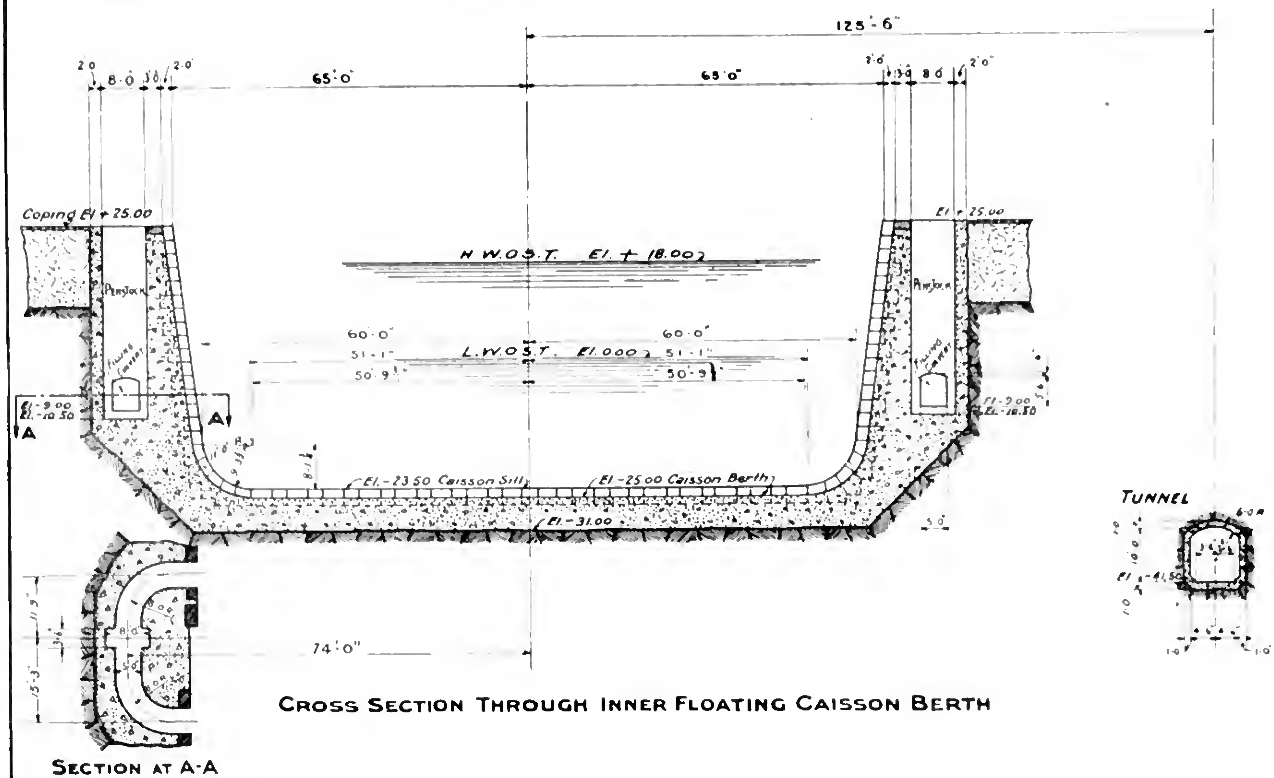
-P.W.C.-"CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-



CROSS SECTION SHOWING CROSS DRAIN, SUMP & EMPTYING CULVERT

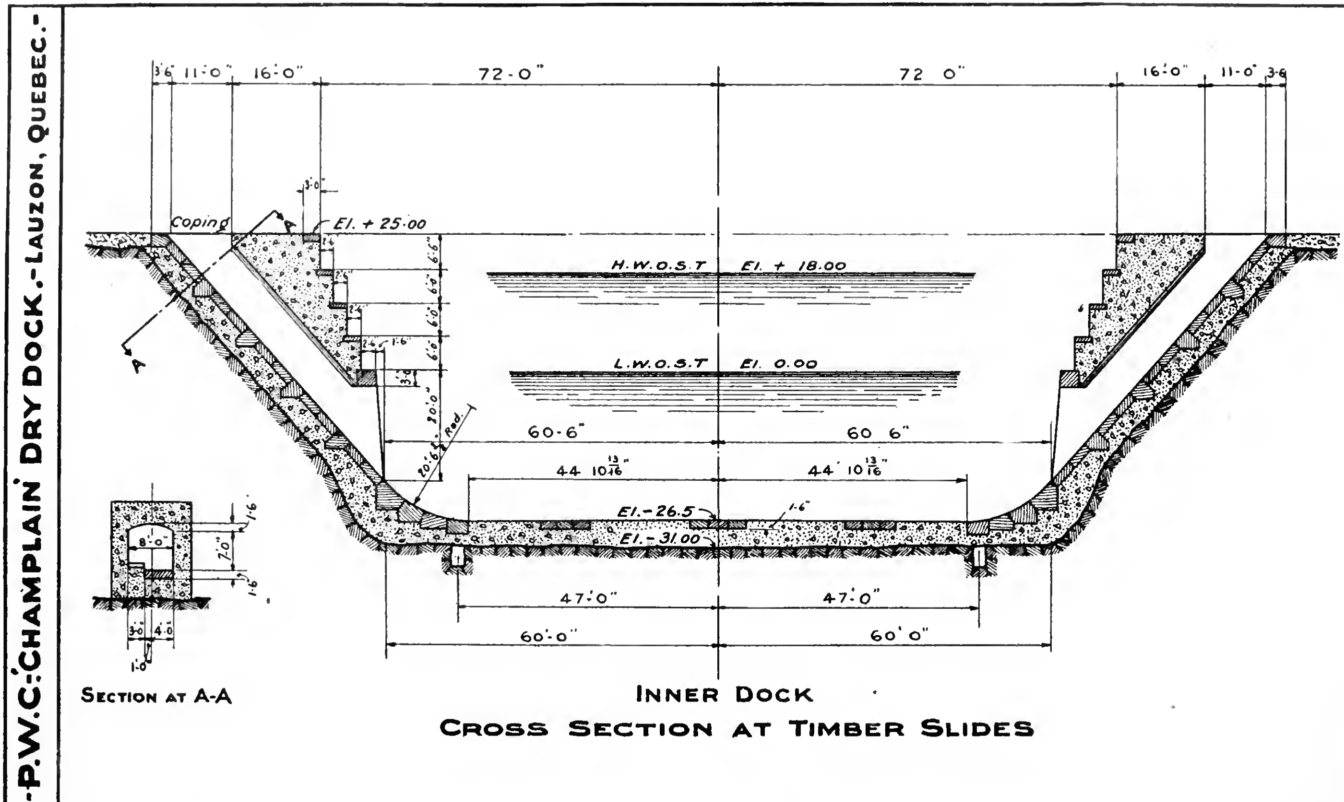
Fig. 9

-P.W.C. Champlain Dry Dock.-Lauzon, Quebec.-



Cross Section Through Inner Floating Caisson Berth

Fig. 10



-P.W.C.-CHAMPLAIN DRY DOCK.-LAUZON, QUEBEC.-
3'6" 11'-0" 16'-0" 72'-0" 72'-0" 16'-0" 11'-0" 3'-6"
Coping
El. + 25.00
H.W.O.S.T El. + 18.00
L.W.O.S.T El. 0.00
60'-6" 60'-6"
44' 10 13/16" 44' 10 13/16"
El. -26.5
El. -31.00
47'-0" 47'-0"
60'-0" 60'-0"
Section at A-A
Inner Dock
Cross Section at Timber Slides

Fig. 11

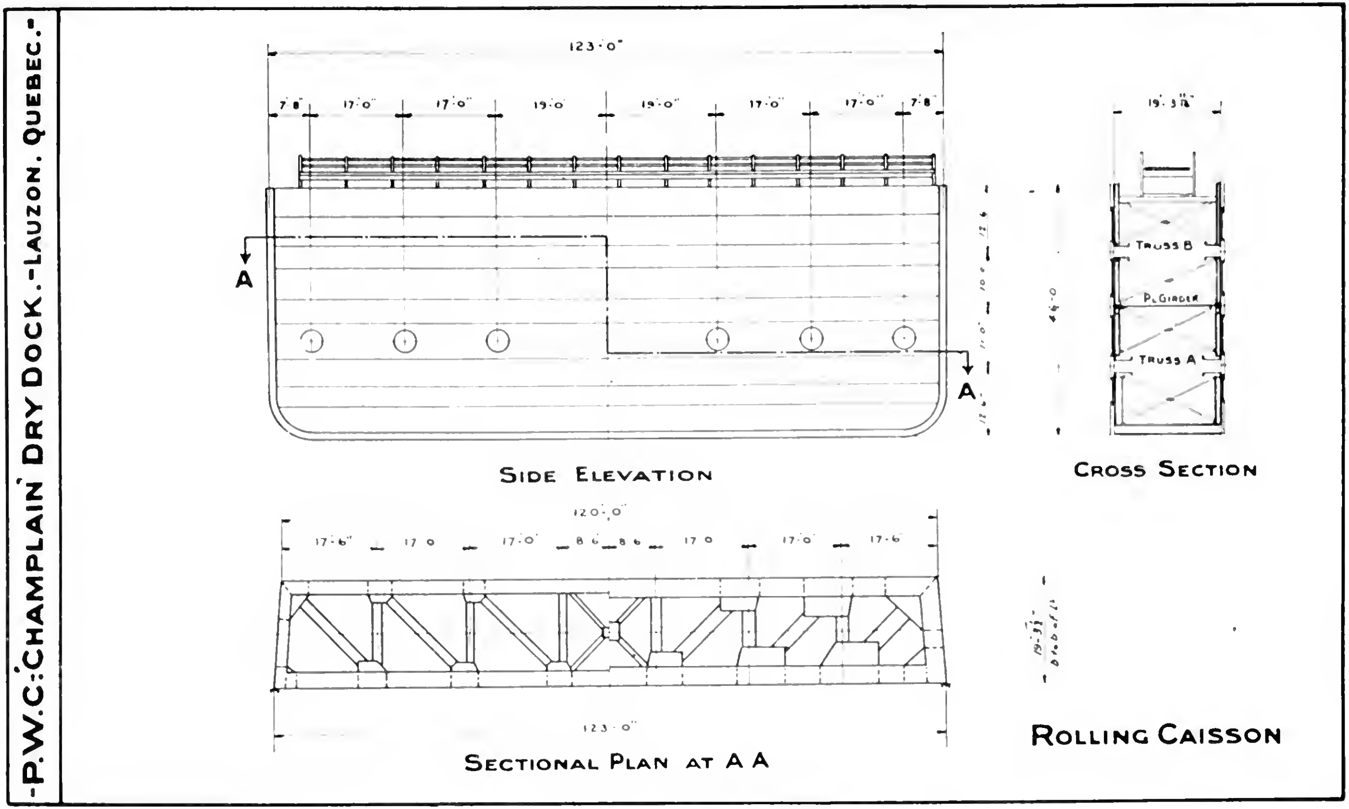

-P.W.C. 'CHAMPLAIN' DRY DOCK. -LAUZON. QUEBEC.-
123·0"
7·8"
17·0"
17·0"
19·0"
19·0"
17·0"
17·0"
7·8"
A
A
19·3⅜"
TRUSS B
P. GIRDER
TRUSS A
SIDE ELEVATION
CROSS SECTION
120·0"
17·6"
17·0
17·0"
8·6
8·6
17·0
17·0"
17·6
123·0"
SECTIONAL PLAN AT A A
ROLLING CAISSON

Fig. 12

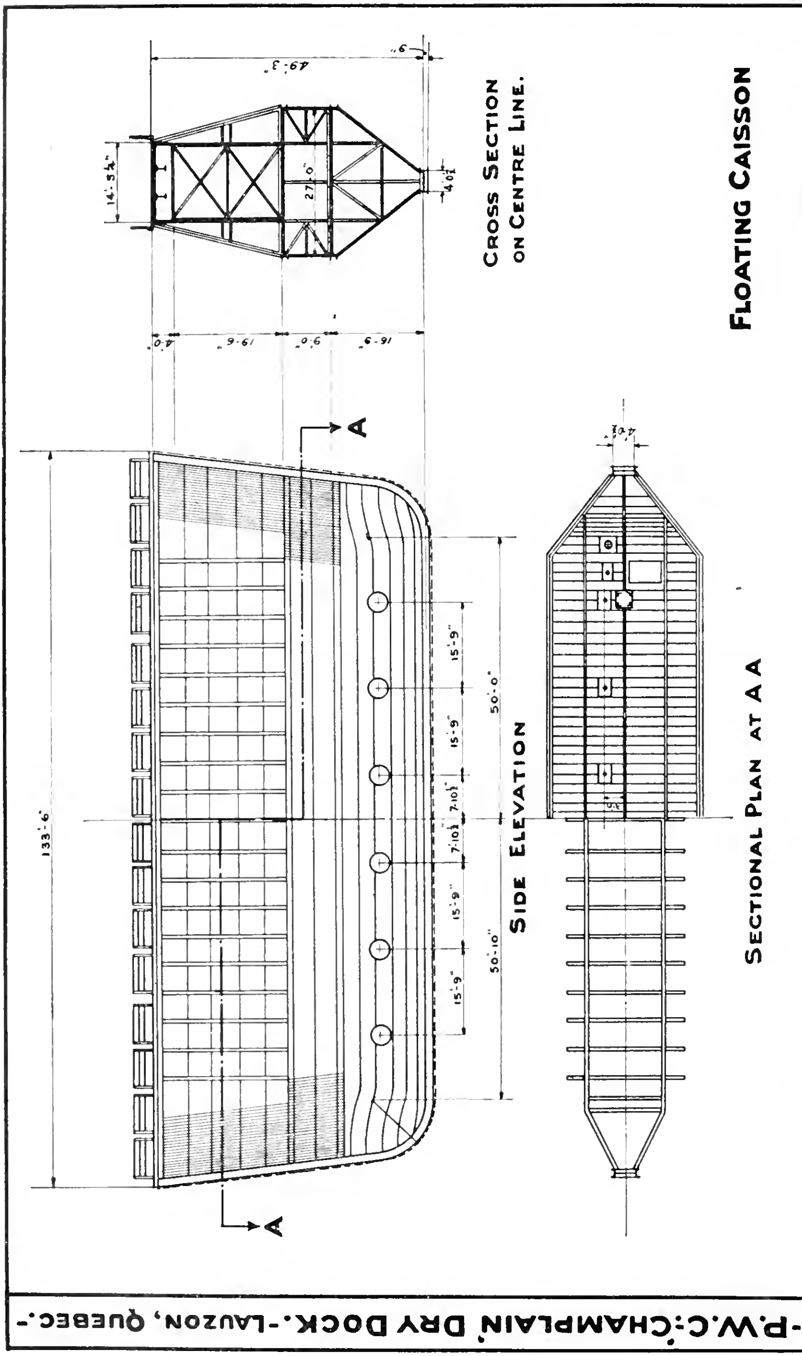

FLOATING CAISSON
CROSS SECTION ON CENTRE LINE.
SIDE ELEVATION
SECTIONAL PLAN AT A A
-P.W.C.-"CHAMPLAIN" DRY DOCK.-LAUZON, QUEBEC.-

Fig. 13

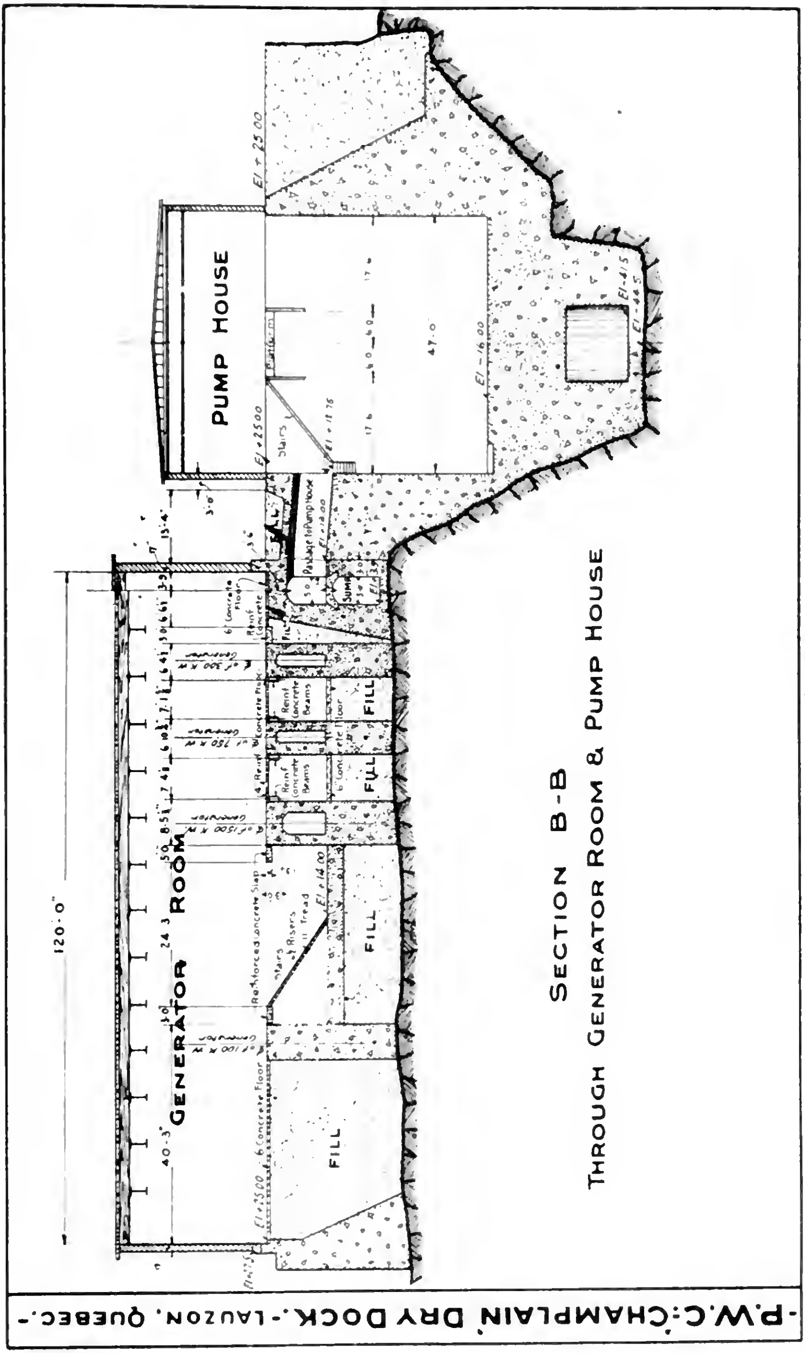
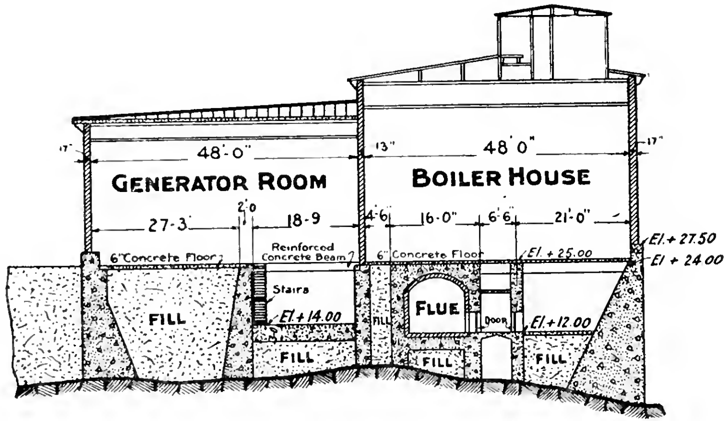

PUMP HOUSE
GENERATOR ROOM
120'-0"
FILL
SECTION B-B
THROUGH GENERATOR ROOM & PUMP HOUSE
-P.W.C.-CHAMPLAIN DRY DOCK.-LAUZON, QUEBEC.-

Fig. 14

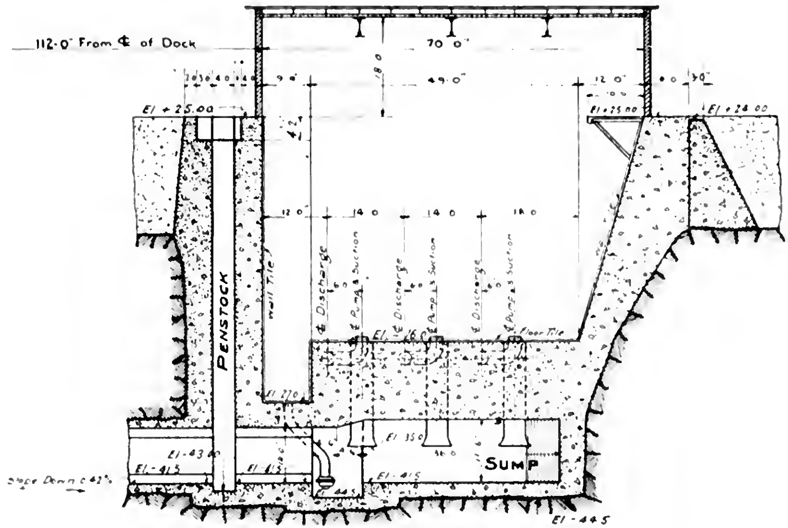
-P.W.C.-CHAMPLAIN DRY DOCK.-LAUZON, QUEBEC.-



SECTION C-C THROUGH POWER HOUSE

Fig. 15

-P.W.C.-"CHAMPLAIN" DRY DOCK.- LAUZON, QUEBEC.-



Section D-D Through Pump House

Fig. 16

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R.A.M.—Report of Annual Meeting, 1918.