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THE CANTILEVER BRIDGE ACROSS THE ST. LAWRENCE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S A.

(For Description, see opposite Page.)

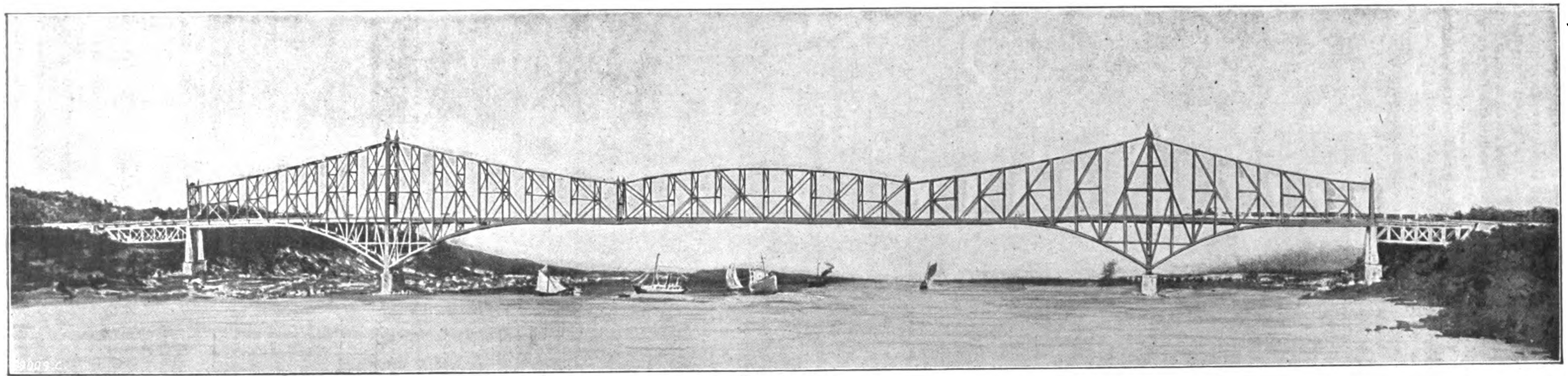
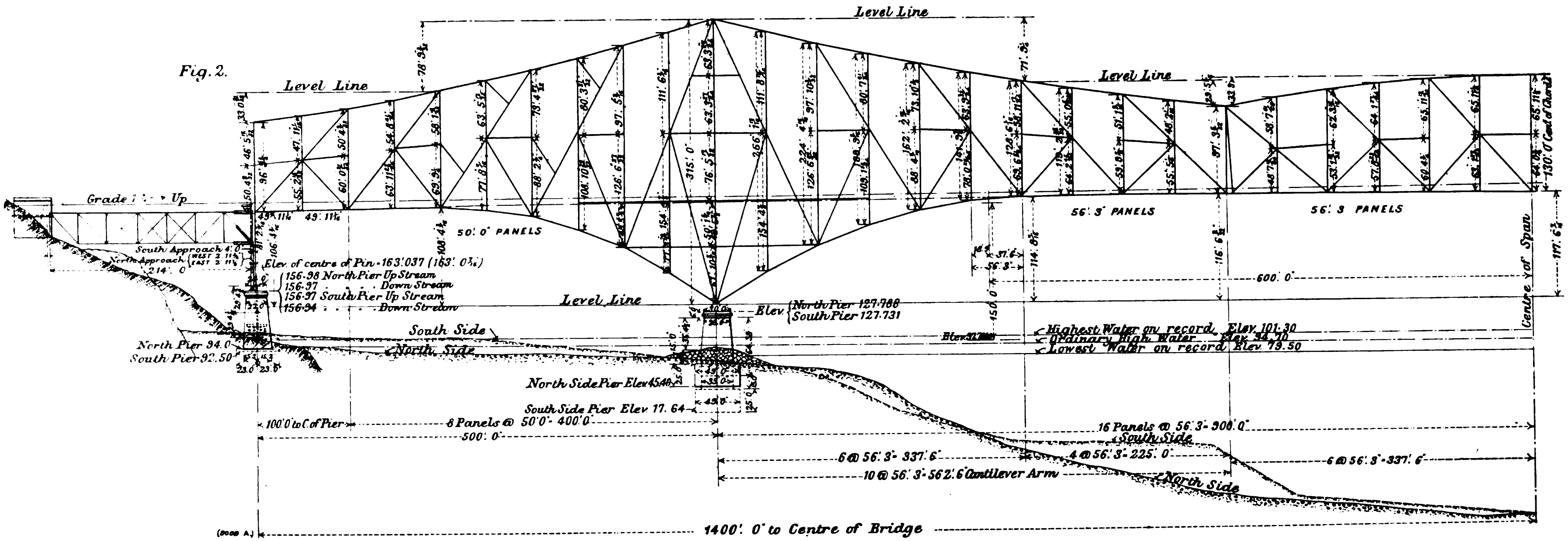


FIG. 1.

Fig. 2.



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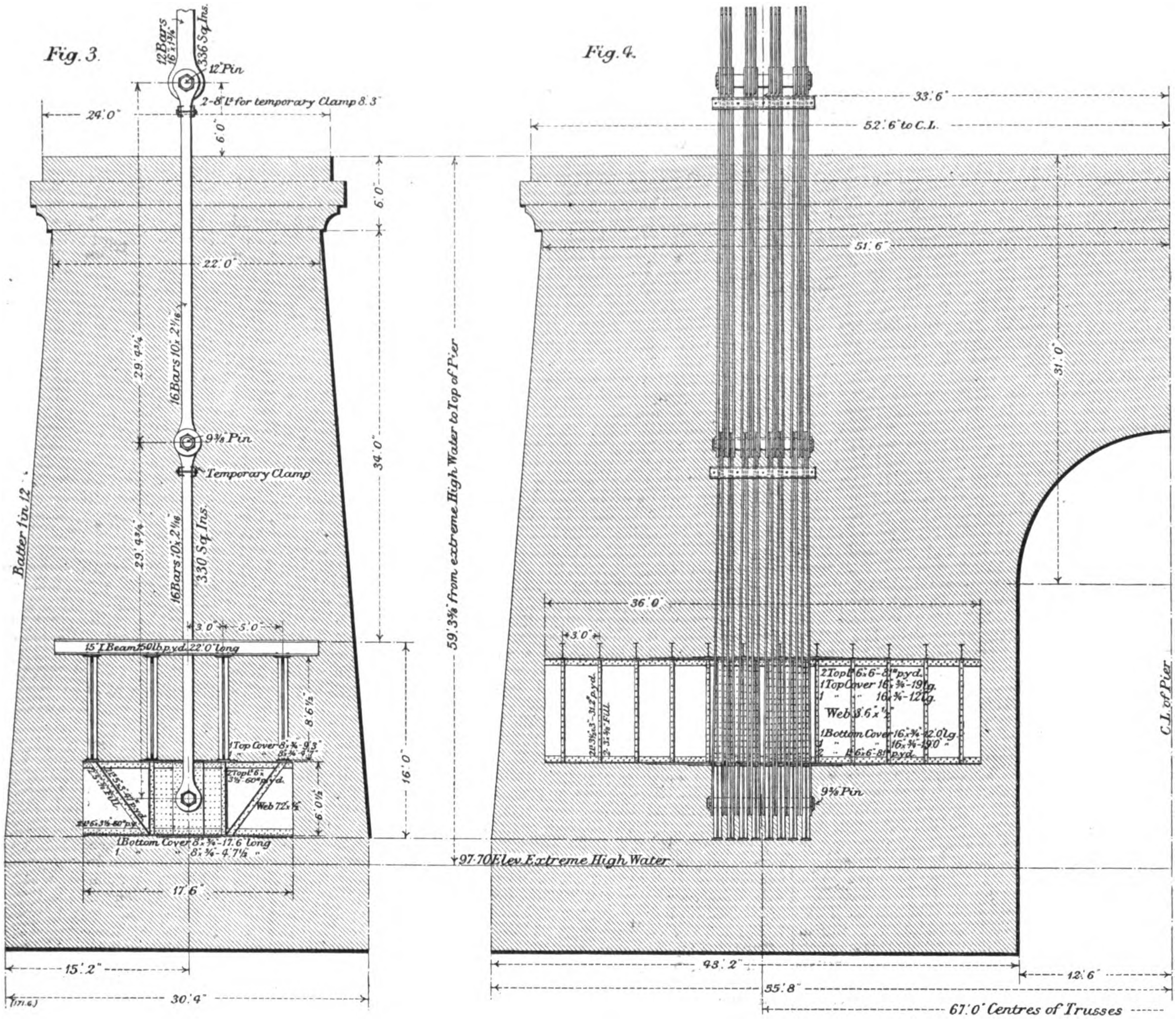
THE QUEBEC BRIDGE DISASTER.

On the afternoon of Thursday, August 30, a large portion of the great cantilever bridge in course of construction across the St. Lawrence, at a point some 8 miles below Quebec, collapsed and fell into the river, carrying with it some eighty to ninety workmen, of whom sixty-one are supposed to have lost their lives. This death-roll, lamentable as are its proportions, has, unfortunately, been exceeded in some previous bridge disasters; but in certain other aspects the catastrophe at Quebec is unparalleled. Perhaps its most serious feature lies in the fact that the structure was not a small bridge, the contract for which had been secured at a low price by a struggling firm, compelled to find their profit in cutting down scantlings, and using material only just capable of meeting the requirements of the inspector, but an edifice which, when finished, would have had the largest span in the world, exceeding that of the Forth Bridge by 90 ft., whilst the contractors were the Phœnix Bridge Company, of Phœnixville, Pennsylvania, probably the premier bridge-builders of America. In normal years this firm turns out 40,000 tons of bridge-work per annum, and the ability and honesty of its staff are universally acknowledged. The consulting engineer for the structure was Mr. Theodore Cooper, of New York, who is, perhaps, the leading bridge consultant of the United States. The Quebec bridge was to have been built on the same general system as the great Scotch structure, but the details were very different. Its general appearance is well shown in Fig. 1 on the opposite page, whilst its principal dimensions are given in Fig. 2 on the same page.*

It consists, it will be seen, of two anchor spans of 500 ft. each, and one cantilever river span of 1800 ft. This great span has been necessitated by two considerations: first, the great depth of the river, which for a large proportion of its total width has a depth of 200 ft. at high-water; and secondly, by the extraordinary ice "shoves" which occur at the commencement of every spring. The two river piers, therefore, have been founded near the banks in but 10 ft. of water, and the anchor spans are dry at low tide. Though 350 miles from the sea, the tide has here a range of 15 ft., and the current attains at times a speed of 8 miles an hour. Though, as stated, the river is only 10 ft. deep at the site of the piers, it has, owing to the character of the strata, been necessary to carry down the foundations to a depth of 65 ft. below high water, the pneumatic plenum process being used, the caissons being constructed of timber. At the Forth Bridge one of the piers had to be founded at a depth of 89 ft. below high water, and the caissons were, as usual in English practice, of steel.

In the Forth Bridge the main compression members are tubes 12 ft. in diameter, and the main tension members riveted lattice girders. Such features are not well adapted to the American system of bridge-construction, in which every endeavour is made to reduce the amount of work done at the bridge site to a minimum, everything possible being done in the permanent bridge-works of the contractors. This procedure had some special advantages in the present instance, as the severe Canadian winter made it imperative to stop all work on the actual bridge site for five months of each year. The Quebec Bridge was therefore designed, in accordance with the ordinary American practice, as a pin-connected structure, the top chords of the cantilever consisting of eye-bars, while the lower chords are not hollow tubes, as at the Forth Bridge, but are rectangular boxes, having four webs, through which pass the pins for the main verticals. The outside dimensions of these struts are given as $4\frac{1}{2}$ ft. by $5\frac{1}{2}$ ft. over all, and they were therefore much less rigid than the 12-ft. tubes used at the Forth Bridge. Great advantages are claimed for the American system of construction in the matter of lightness, and in comparing the Scotch and Quebec bridges it has been pointed out that the dead load in the case of the former is $9\frac{1}{2}$ times the live load, whilst the corresponding figures for the St. Lawrence structure are $4\frac{1}{2}$ to 1, an apparent economy of some 60 per cent., or even more, since the span in the second of the two bridges is 90 ft. larger. The claim that this difference is due to the adoption of the American system of construction is, however, ridiculous, since the weight of a large bridge is fixed almost wholly by the ratio of depth to span, and is little affected by reasonable differences in



Figs. 3 and 4. Sections of Anchorage Pier.

* We reproduce these illustrations from the issue of Engineering of September 22, 1905 (vol. lxxx., page 375), in which we gave an account of the general design of the bridge, with illustrations of some of its details.

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structural detail. Since there is no great difference in this ratio in the case of the Forth and Quebec bridges, the above comparison merely shows that a very much higher factor of safety was adopted at the Forth than at the St. Lawrence. In fact, the live load on the Forth Bridge could be doubled without bringing up the stress on the material of the main trusses to the figures adopted in its rival. At the Forth Bridge the steel used in the compression members was not in any case strained to more than $7\frac{1}{2}$ tons per square inch, inclusive of a wind load of 56 lb. per square foot, reckoned on twice the vertical projection of the structure. The struts were, moreover, 12-ft. tubes, as stated, and thus enormously stiffer than the corresponding members of the St. Lawrence Bridge; whilst the steel used in them had a strength of 34 to 37 tons per square inch. At the St. Lawrence the strongest steel used has an ultimate strength of 63,000 lb. (28 tons) only, yet it appears from the drawings to be strained to a very much higher degree.

From the strain-sheet prepared by the contractors it appears that the loads on the lower chord of the panel next the river pier are as follow:—

Live load	4,246,000 lb.
Dead load	11,883,000 ,,
Wind load	± 9,060,000 ,,

The total section provided to take this was 776.2 square inches, so that the maximum stress appears to be 32,400 lb. (14.5 tons) per square inch, which seems exceptionally high for steel of the quality here in question. Of course, the wind load is but an occasional one, and of late years there has been a tendency in the case of large structures to admit of very high working stresses in the case of these infrequent loads. In the bridge under discussion no provision whatever was made for these wind loads unless they exceeded 30 per cent. of the maximum combined live and dead loads; whilst if the proportion was larger than this limit, the scantlings were increased sufficiently to reduce the total stress on the member to a limit 30 per cent. greater than the normal working stress of the material.

In one other respect the bridge was undoubtedly less safe than that at the Forth. The latter is a double intersection structure, and could still stand even if one member showed signs of weakness. Mr. Theodore Cooper is, however, strongly opposed to this system of construction, though it undoubtedly gives greater security than the single intersection system adopted by him. The only objection ever advanced to the double intersection system is that it is impossible to calculate with absolute exactness the distribution of the stresses. A bridge is not, however, constructed for the purpose of making such calculations, but to safely carry a load, and the argument, moreover, comes badly from designers who employ, for tension members, groups of eye-bars, the true distribution of stress over which is really incalculable, and it has accordingly to be assumed as evenly distributed amongst the lot. Rumour has it, indeed, that in times past eye-bars differing in length by $\frac{1}{8}$ in. or more have been assembled on the same pins, and yet have been assumed to be equally stressed by the load. It is conceivable, though, perhaps, not probable, that the St. Lawrence structure might still be standing if it had been more scientifically designed in this regard. It will be seen that the single-intersection system has necessitated the use of very long panels, which have accordingly been divided into sub-panels, so as to avoid an uneconomically long spacing between the cross girders. With the double intersection system the desired close spacing would have been obtained in a distinctly more mechanical fashion, and the main chord would have been better supported.

The telegraphic reports so far received give no clear indication of the manner of failure. Some seem to imply that the bridge overturned, due to overloading the overhang of the structure. The central span was being erected by building it out as an extension of the true cantilever, on the lines followed at the Forth Bridge; and apparently, at the time of the accident, the overhang was about 800 ft. An enormously heavy traveller—its weight being 1000 tons—was being used for the erection of the cantilever proper, and may have been employed also for putting the central span panels in place. We understand, however, that though this was the original intention, it was abandoned, and that it was proposed to use here a much lighter traveller on this portion of the work. Even if, however, the original plan was followed, it does not seem possible that the mistake could have been made of underestimating the dead weight required at the anchorage. We give a section through the latter in Figs. 3 and 4, page 329. The maximum up-lift provided for is, it will be seen, 4,724,500 lb., or, say, 2100 tons per truss. This figure, however, presumably has reference to the condition of affairs after the completion of the structure, but at the time of the disaster it would apparently have been just about the same.

The weight of the shore arm is given as 5590 tons; that of the river cantilever as 6710 tons; whilst the central span was estimated to weigh 2680 tons. The weight of the floor system is, however, not included in the foregoing figures, and it amounted to about 1.3 tons per foot-run. With these data, if the great traveller was at the end of the overhanging arm, the total uplift would be about 4300 tons, or 2150 tons per truss. The weight of the pier, however, seems to be at least 4000 tons per truss, so that even if the conditions at the time of the failure were those above assumed, the edifice seems to have been secure from overturning.

Other accounts of the accident give the impression that the bridge failed by the crippling of the lower chord in the panel next the river pier. If precedent counts for anything, this hypothesis would not be improbable, since almost every bridge or roof failure on record has occured through the crippling of some compression member, failures of tie-bars being very rare. Indeed, the Charing Cross roof is the only instance of such a failure which we can at present call to mind. It is by no means uncommon, moreover, to omit during the erection of a bridge certain of the floor members, as this frequently facilitates the handling of material, and on the first receipt of the news of the catastrophe, it appeared therefore possible that something of the sort had been done here, in which case the giving way of the then insufficiently supported lower chord would have received a ready explanation. The practice referred to, however, is seldom resorted to in the case of very large structures, and, as will be seen on reference to Fig. 9, Plate XLIV., nothing of the sort was attempted at Quebec. This illustration shows the lower bracing of the cantilever arm over the river, and it will be seen that it is complete in every respect, and apparently ample in proportions. A view of the upper lateral bracing of the shore cantilever is given in Fig. 10, which also shows the head of the enormous 1000-ton traveller with which the erection was effected. This was 215 ft. high, and "straddled" the structure as indicated, being supported on temporary girders slung from the main pins of the lower chord in the manner shown in Fig. 9. This traveller was equipped with electric winches, and was designed to lift pieces weighing as much as 105 tons each. In Fig. 5, Plate XLIII., it is shown in position at the end of the cantilever, and another view showing the same thing is given in Fig. 15, Plate XLVI. Its overhang is 66 ft. The bridge members, as already explained, were completed at the permanent works of the contractors, from which they were transported by rail to the bridge site and lifted bodily into place, as indicated in Figs. 6, 7, and 8. The eye-bars were handled in complete groups, being assembled below on temporary pins. With these in place the whole batch were firmly clamped together, the temporary pins removed, and the whole lot hoisted as single units in the fashion indicated in Figs. 7 and 8. The eye-bar system is, of course, a less mechanical method of construction than that of making all connections by riveting, as practised here, but it has some very great advantages in the matter of facilitating rapid erection. At Quebec as much as 330 tons of steel have been put in place in a single day, though the staff of men employed has never exceeded a couple of hundred or so. For the past twenty years, however, riveted connections have been steadily growing in favour in the United States, and, as will be seen from Figs. 11 to 14, Plate XLV., they have been very largely used at Quebec. The figures of the workmen give a good idea of the vast scale of the details, and these, it will be seen, in spite of an unavoidable complication, have been designed in a thoroughly straightforward and mechanical fashion, so that, whatever the cause of the failure, it is evidently not to be sought in flimsy and poorly proportioned joints between the main and subsidiary members. A Press report states that some time before the accident a roadway stringer showed signs of weakness, and twisted. Even if true, however, this circumstance would afford no explanation of the collapse, since such elements of the bridge are quite independent of the main structure, and could be entirely removed without in any way compromising its safety. If, therefore, the lower chord did collapse, as appears probable from the Press reports, the failure must be attributed solely to its inadequacy to stand the load actually applied to it. As has already been pointed out, it was designed with a factor of safety not much more than one half that adopted at the Forth Bridge.

The full stress of 14.5 tons per square inch on the metal would not, it is true, be operative in the case of the finished structure unless a gale of wind were blowing; but though the bridge was incomplete, the temporary stresses due to the dead weight appear to have been nearly $8\frac{1}{2}$ tons per square inch on the lower chord. Taking the overhang as 800 ft. and the traveller as in position at the end, the total bending moment of the dead load about the pier seems to be about 3,700,000 foot-tons, equivalent to a load on each lower chord of about 6500 tons. The corresponding stress works out, as stated, to nearly $8\frac{1}{2}$ tons per square inch. This may, perhaps, have been brought up to 9 tons by rolling-stock and material passing along the tracks. A gale under the conditions named would, however, have increased this stress by 60 per cent., since the bridge is relatively narrow in comparison with its length. The wind load taken by the lower chords is, in short, estimated at 1000 lb. per lineal foot, which on an overhang of 800 ft. amounts to nearly 360 tons. The moment of this about the pier would be 144,000 foot-tons; and as the width is 67 ft., the equivalent wind load on the chord would be about 2140 tons, or, allowing for the fact that the chord is inclined to the horizontal, 3800 tons, bringing up the total stress to rather over 13 tons per square inch. None of the reports, however, mention the prevalence of a high wind at the time of the accident, and, apart from this, the erection stresses on the chord do not appear to be in excess of the designed normal working stress of the metal in the completed structure. It seems possible, nevertheless, that the collapse has really been due to wind action, since it is conceivable that a series of light gusts may have synchronised with the natural period of vibration of the overhanging steel-work, in which case no limit could be fixed to the possible resultant stresses on the chords.

The stress p arising from each foot of horizontal deflection is given approximately by the relation

$$p = \frac{E\,h}{s^2},$$

where E denotes the modulus of elasticity, h the width of the bridge at the pier, and s the overhang. Taking E as 13,000 tons, h as 67 ft., and s as 800 ft., we get $p = 1.36$ tons for each foot of horizontal deflection of the overhanging arm. Strictly speaking, the stress would be somewhat less, since the formula employed assumes a uniform moment of inertia, but this is largely compensated for by the fact that the chord member in question is sharply inclined to the horizontal. It would seem, therefore, that a considerable stress might arise in this way, and this appears, so far, the most plausible explanation of the collapse. As against this, however, it has to be remarked that none of the men rescued are reported as having noticed any notable swaying of the bridge at the time of the accident. Still the periodic time would be fairly large, being apparently, from a rough calculation, about 6 seconds for a complete double vibration.

Fuller details may yield a less recondite explanation of the collapse, and show it to have been due to carelessness on the part of the engineering staff in charge of the erection. The latter, however, consisted of experienced and very able men, unlikely to run undue risks in a work of this magnitude and importance.

Since the above was in type a telegram has been published in the *Times*, which definitely attributes the accident to the buckling of the lower chord of the river arm cantilever. This, it will be seen, is in full accordance with the conclusions arrived at above, and possibly the Canadian mail, due next Monday, may bring evidence of the swaying of the structure, which at present seems the most probable origin of the overloading of material, which has had such a disastrous sequel.

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THE CANTILEVER BRIDGE ACROSS THE ST. LAWRENCE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(*For Description, see Page* 329.)

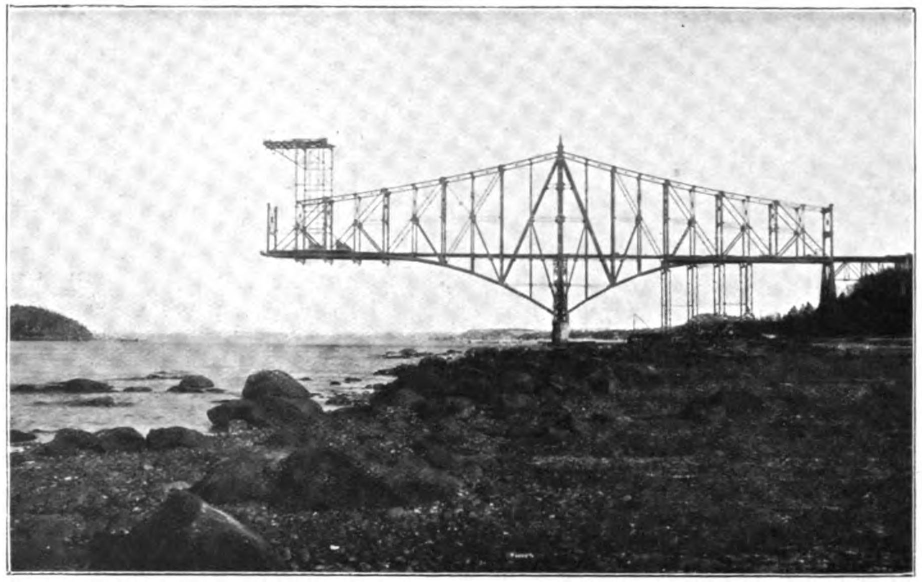

FIG. 5. VIEW OF SOUTH CANTILEVER AND ANCHOR-ARM; TAKEN NOVEMBER 29, 1906.

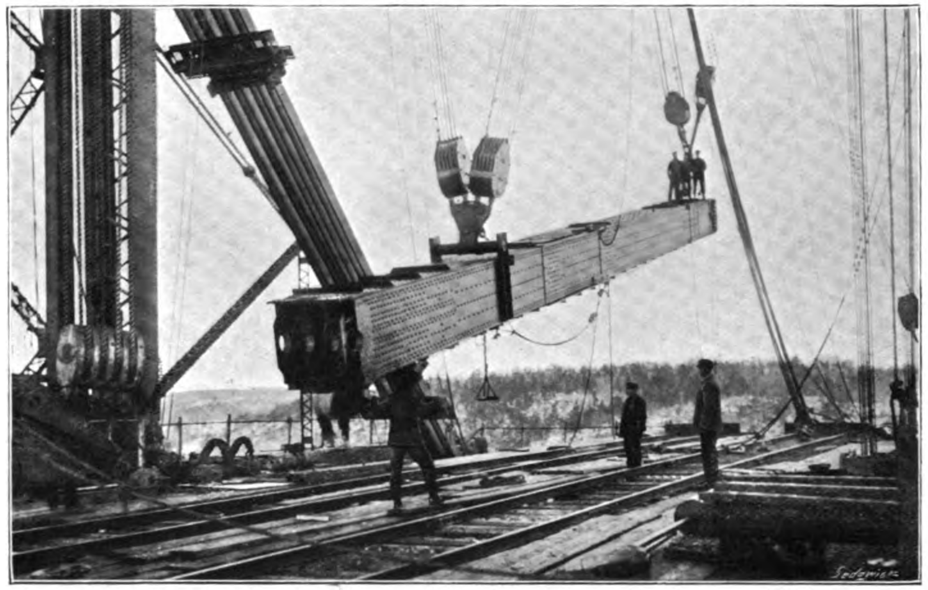

FIG. 6. HOISTING MAIN END POST OF CANTILEVER-ARM; NOVEMBER 21, 1896.

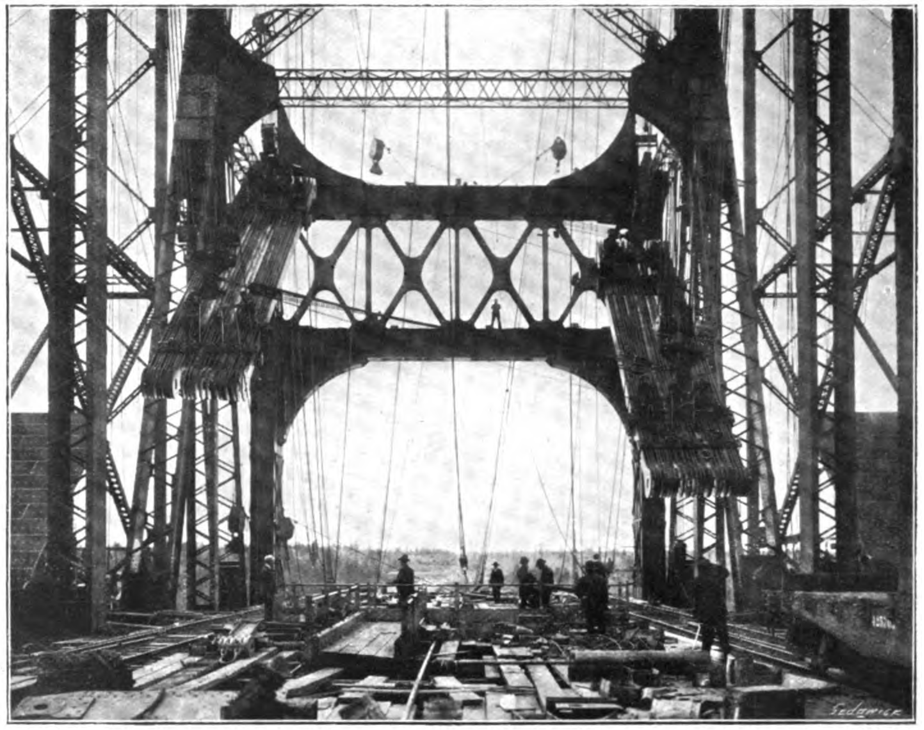

FIG. 7. TWO COMPLETE UPPER CHORD PANELS BEING HOISTED INTO POSITION.

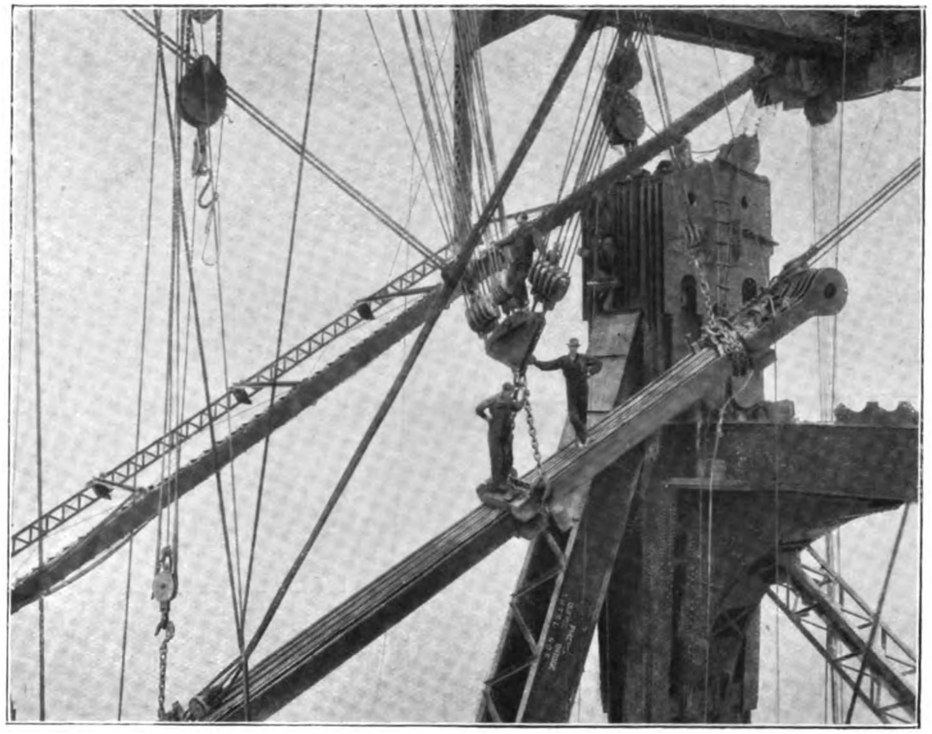

FIG. 8. HOISTING CENTRE PANEL TOP CHORD EYE-BARS INTO POSITION.

(*To face Page* 328.)

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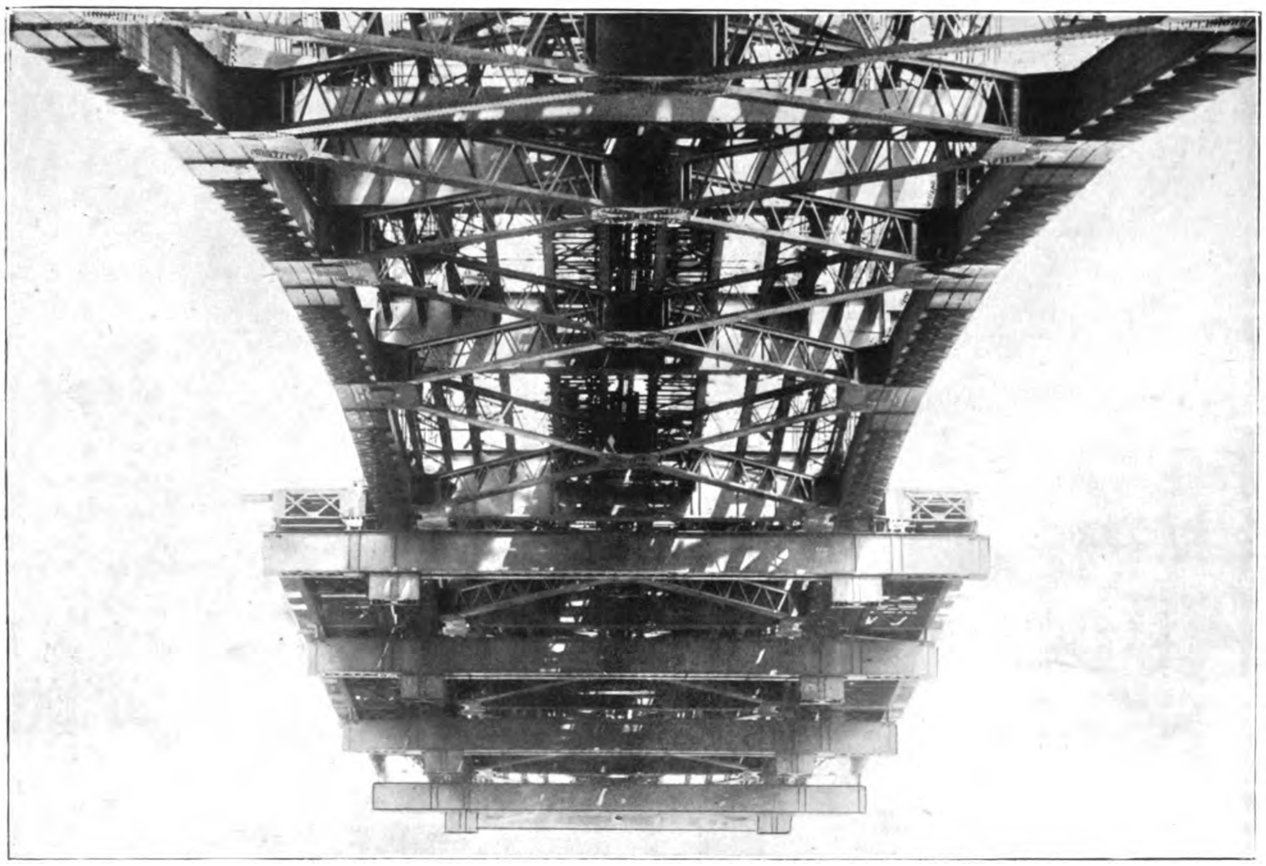

FIG. 9. VIEW OF UNDERSIDE OF CANTILEVER, LOOKING FROM SOUTH MAIN PIER.

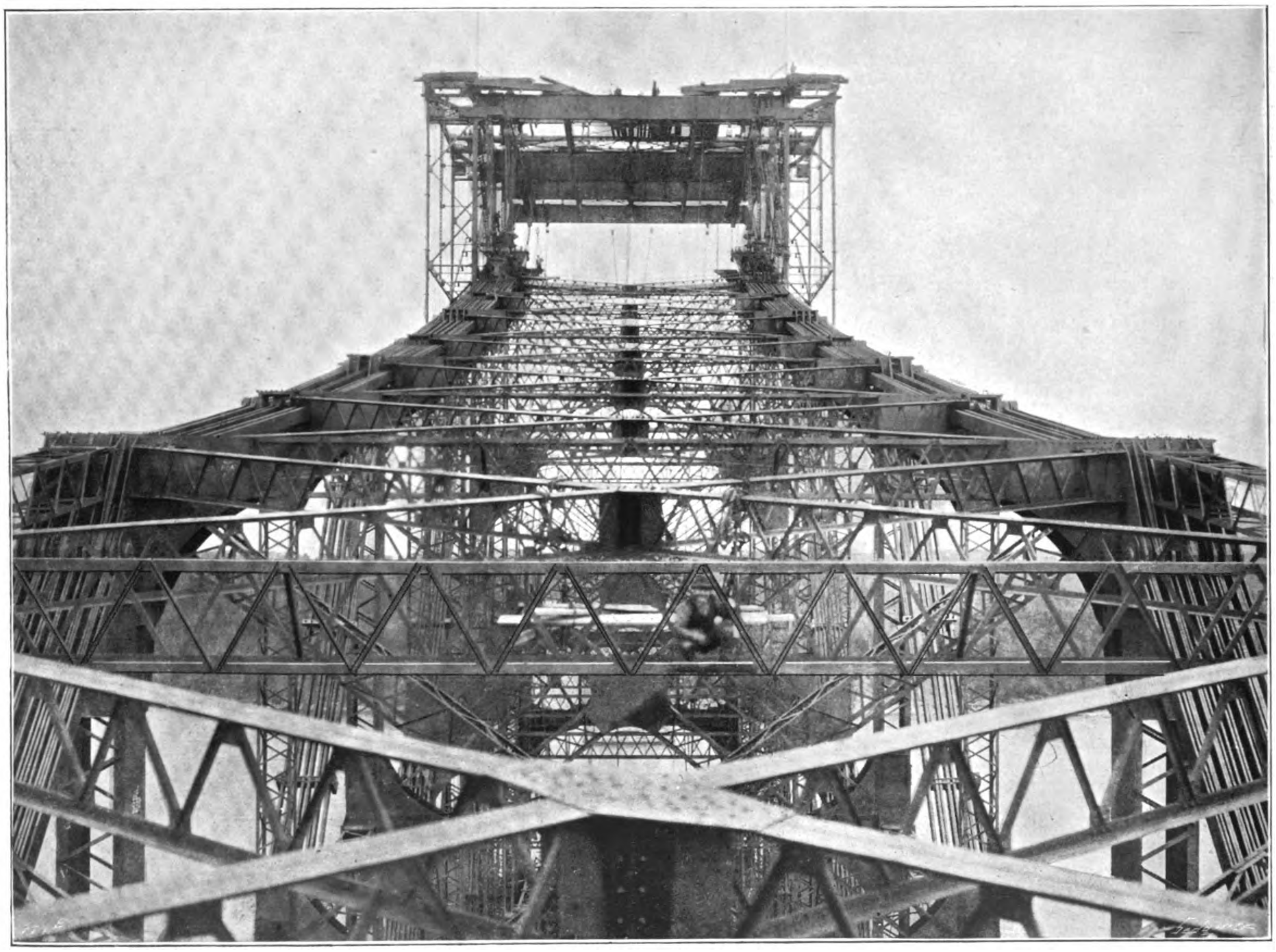

FIG. 10. VIEW OF TOP BRACING OF SOUTH ANCHOR-ARM.

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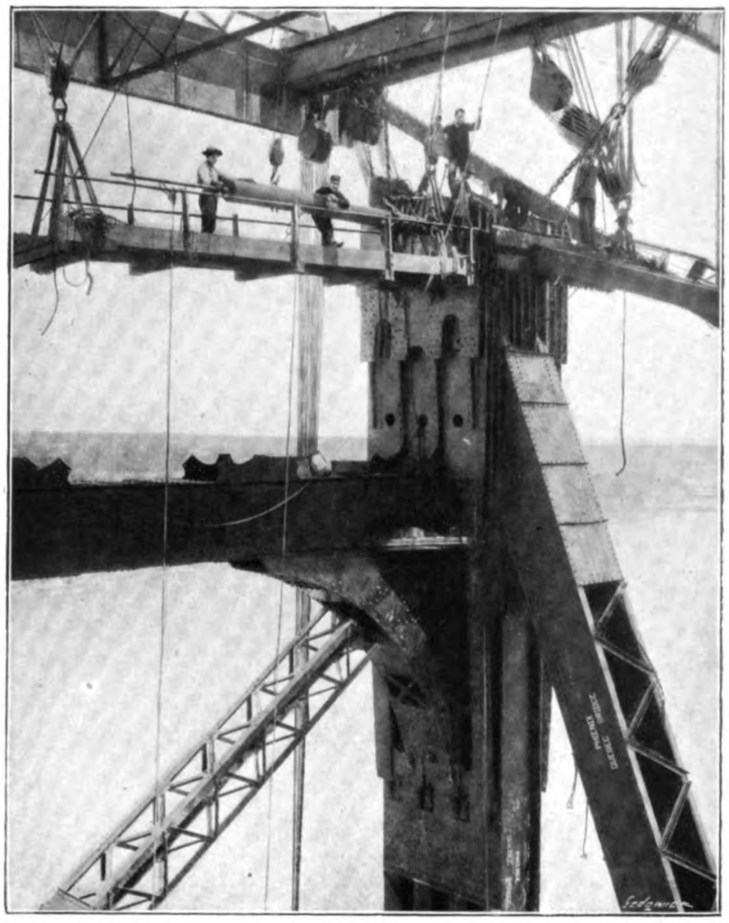

FIG. 11. DRIVING MAIN 12-IN. TOP CENTRE-PIN.

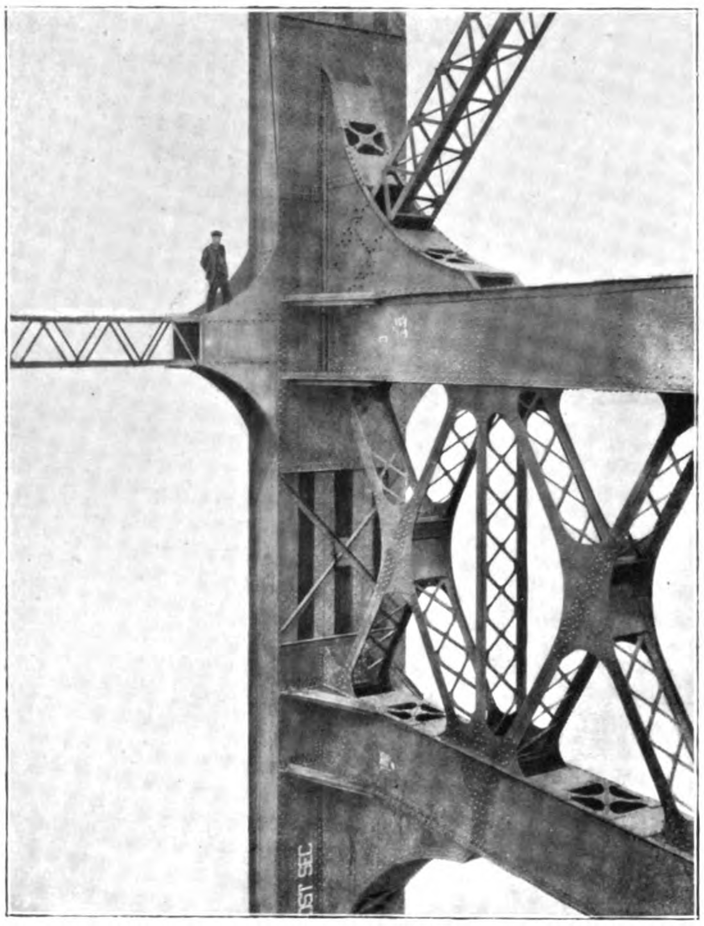

FIG. 12. CONNECTION OF TRANSVERSE BRACING WITH WEST MAIN POST.

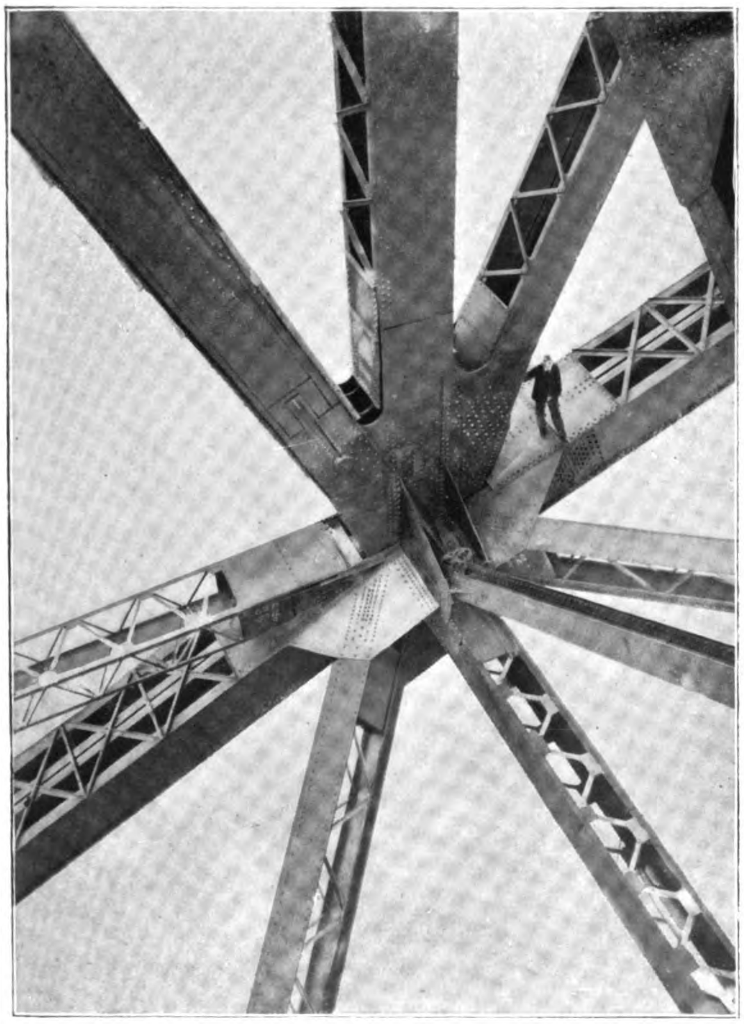

FIG. 13. CONNECTION AT BOTTOM CHORD AT SECOND POINT FROM MAIN PIER ON WESTERN TRUSS OF SOUTH CANTILEVER.

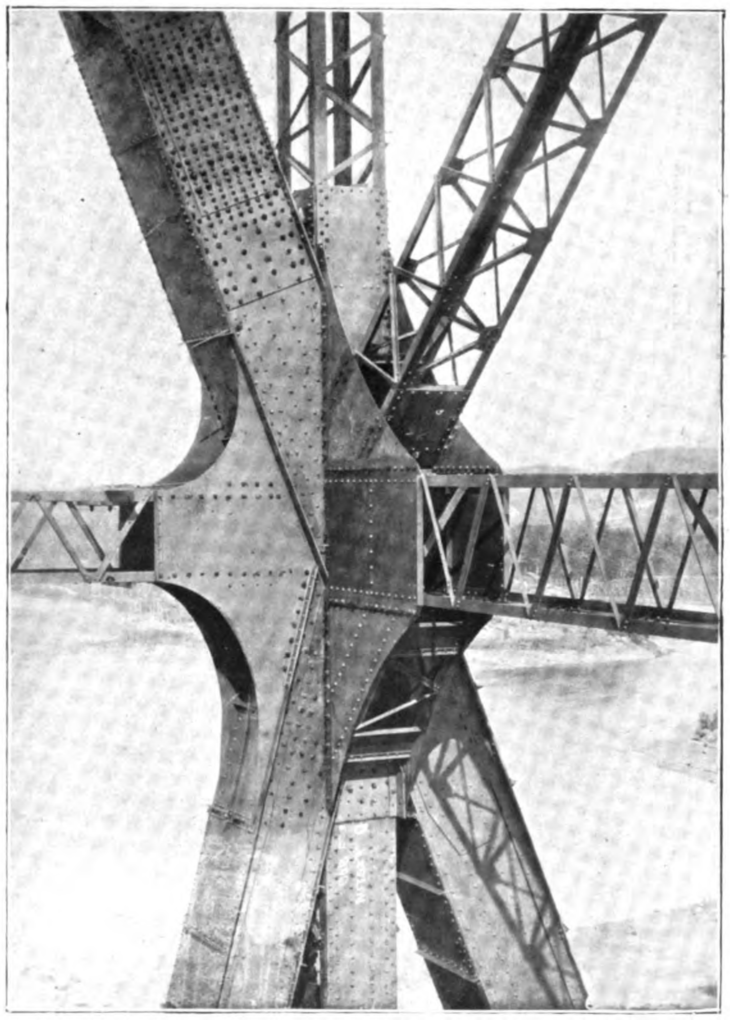

FIG. 14. INTERSECTION OF MAIN DIAGONAL ON ANCHOR-ARM WITH TOP HANGER.

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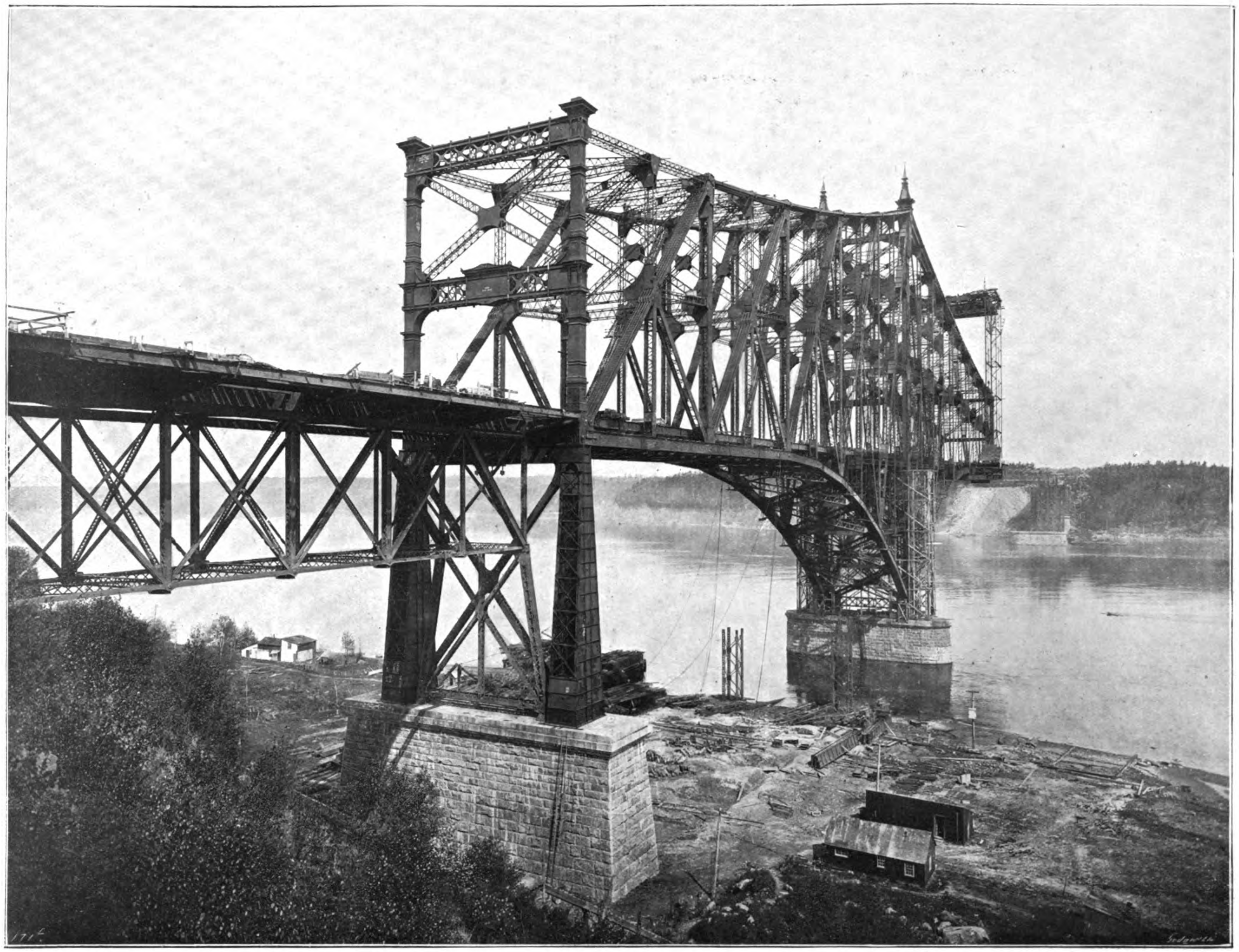

FIG. 15. VIEW OF SOUTH APPROACH SPAN, ANCHOR ARM, AND SOUTH CANTILEVER; JUNE 12, 1907.

ENGINEERING, SEPTEMBER 6, 1907.

(*To face Page* 329.)

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THE QUEBEC BRIDGE.*

By FRANK W. SKINNER, M. Am. Soc. C.E.

THE 4,000,000-dollar four-track bridge across the St. Lawrence River, about 8 miles west of Quebec, is 3240 ft. long over all, about 415 ft. in extreme height above low water, has a centre clearance of 150 ft. above high water, and is notable for its 50,000,000-lb. channel span, 1800 ft. long. All problems of manufacture and erection have been dealt with by Mr. E. A. Hoare, chief engineer, Mr. Theodore Cooper, consulting engineer, and Mr. John Sterling Deans, chief engineer of the Phœnix Bridge Company, who are the contractors and designers of both substructure and superstructure.

In length of span and dimensions of unit members, the bridge is comparable only with the five greatest spans yet built in the thirty years since the advent of structural steel revolutionised engineering and obliterated all former limits of construction; and since the creation of portable mechanical plant of great power, operated by steam, compressed air, and electricity, vastly extended civil engineering construction. Indeed, except the proposed North River Bridge, New York, with a six-track, 2850-ft. suspended channel span, careful designs and estimates of which were seriously discussed about twenty years ago, this great span excels all others that have ever been prominently advocated or commenced.

The two 1710-ft. spans of the magnificent Forth Bridge, so brilliantly created in the early days of steel construction, and so long of unapproached magnitude, lack 90 ft. of the length of the Quebec Bridge, and have less than half its capacity. That both great spans are cantilevers constitutes almost their only resemblance; the Forth Bridge main truss members are all huge tubes, built up *in situ*, with elaborate plant established at the spot, and with all connections riveted, with consequent indeterminate stresses in some cases, notably at the feet of the main towers. The Quebec Bridge members have riveted rectangular webbed members for compression, and forged bars, not capable of transmitting compression, for tensile stresses. All primary joints are pin-connected, and all stresses are positive. All members were completely finished in the shops with highly-developed standard machine-tools used for the regular bridge business of the contractors, and all were erected as complete units, handled bodily, and supported in groups until successive panels were completed and self-sustaining.

The 1600-ft. six-track suspended span of the Williamsburg Bridge, New York, which was described in ENGINEERING in 1905 (see vol. lxxx., pages 541, 577, 643, 755, 787, and 854) ranks next in combined length and capacity; but, with the exception of the main cables, its superstructure involved no large members or difficult feats of erection, and the type is so totally different from that of the Quebec Bridge that no comparison can be drawn between them.

The 840-ft. two-hinge span of the highway and electric-car bridge across the Niagara River and Gorge, just below the Falls, is the next longest span, and the longest arch in the world. Its members are very light, none of them exceeding 10 tons in weight, or 50 ft. in length; its connections were all riveted, and as it was erected by a traveller moving on the old suspension span at the same site which it replaced, the only faint resemblance to the Quebec Bridge lies in the fact that the erection was by the cantilever method, the semi-arch spans being built out from the skewbacks, and temporarily guyed back to the tops of skewback towers.

The 820-ft. span of the Lansdowne Bridge over the Rori Channel of the Indus River at Sukkur, is also a riveted connection cantilever, with enormous members (230 ft. long, weighing 240 tons) in the end panels, which were built up piecemeal *in situ*, somewhat like those of the Forth Bridge, while others were shipped from England in sections, and assembled at the site; and still others, up to 14 tons weight, were erected complete by a system of multiple cableways. The suspended centre span was erected on a temporary falsework span.

The 812-ft. river-span of the double-track Wabash Bridge over the Monongahela River at Pittsburg, Pennsylvania, is the next longest existing span, excepting suspension bridges, and is the longest trussed span yet completed in America. It is a pin-connected structure of the same general type as the Quebec Bridge, and the traveller designed and used for it is the prototype of that used at Quebec. The main span weighs about 7,000,000 lb., contains pieces 130 ft. long, weighing 101,000 lb., and 15-in. pins, 7 ft. long. It was erected by 100 men in 90 days, the cost being only 800,000 dols. for the great span and the remainder of both superstructure and substructure, of which the total length is 1504 ft.

The most marked resemblance and fairest comparison exists, however, between the Quebec Bridge and the still unfinished Blackwell's Island eight-track bridge across both channels of the East River, New York, which corresponds to it in many important features of construction, design, shop-work, and erection. It has five main spans, with a combined length of 3724 ft., and a width of 90 ft., weighing about 86,000,000 lb., and costing, with

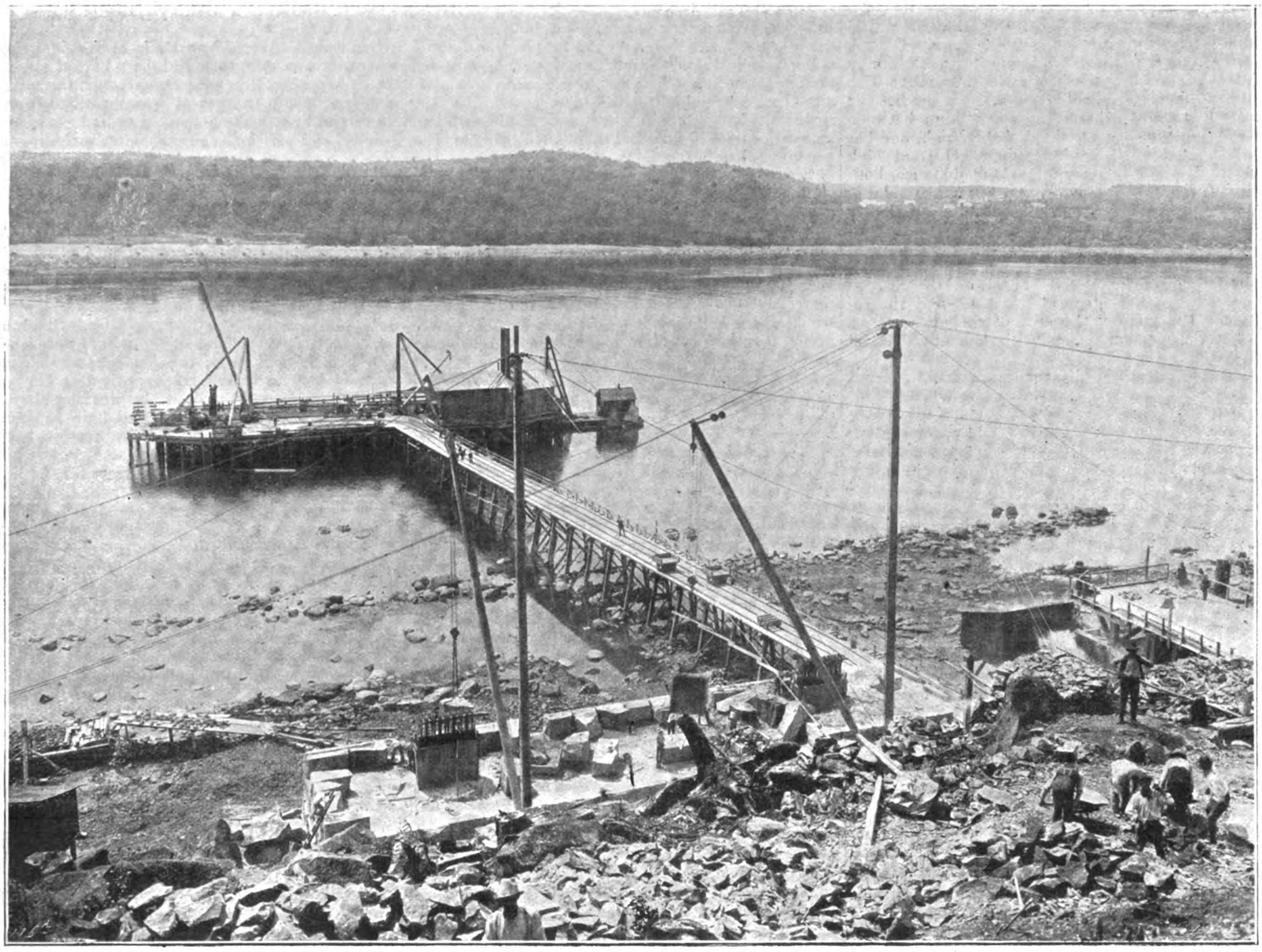

FIG. 1. VIEW ON NORTH BANK, SHOWING ANCHOR AND RIVER PIERS; JUNE 25, 1901.

* [This article was, of course, written prior to the occurrence of the grave disaster with which we had occasion to deal in our last issue. As a complete account of a most important work, however, it has lost none of its interest, and a study of the information which it contains will add much to the understanding of the inquiry now being made into the causes of the failure.—ED. E.]

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approaches and land damages, about 20,000,000 dols. It includes a 984-ft. and a 1182-ft. channel span connected by a 630-ft. island span, with members 100 ft. long, weighing 120 tons each, handled by movable 70-ton derricks and a Z-shaped steel traveller of a similar type to that for the Quebec Bridge, and, like it, has massive steel false-work for the erection of the anchor-spans.

Conditions.—The St. Lawrence River, 2200 miles long from source to mouth, and from 1 to 30 miles wide, is the outlet of five great inland seas, drains a vast empire of 530,000 square miles, including 95,300 miles of lake surface, and has a minimum flow of about 265,000 cubic feet per second. Although not subject to floods, the river has a strong tide at the bridge site, and every winter is covered with ice several feet in thickness, which piles up layer on layer, and in the spring sweeps down in fields miles long.

This bridge, located about 8 miles west of Quebec, will be the only one between Montreal, 160 miles above, and the river mouth, 350 miles below, and will establish railroad and highway communication between Quebec and a large portion of the Dominion of Canada, besides affording a connecting line with railroads from the United States. It crosses the river at a point where the latter flows between steep sandstone banks, about 200 ft. high, and has a width of 1800 ft. at low tide, 2000 ft. at high tide, and is 200 ft. deep for a long distance, both sides of the centre line, with an average tide of about 15 ft. and a maximum current of 8 miles an hour. In order to avoid extreme depth of foundations, and secure the greatest economy of combined substructure and superstructure, the two main piers are located 1800 ft. apart on centres, in water about 10 ft. deep at low tide, and the remaining two piers at the ends of the anchor-arms are 2800 ft. apart on centres, with their sites exposed at low tide.

The bed is of hard cemented strata, covered thickly with large glacial boulders, which are subject to movement by the ice-fields and strong current. Both superstructure and substructure are located to clear the heavy traffic of steamboats, tows, and sailing-vessels, and, as much as possible, to avoid the great ice-fields. The latter, however, reach far beyond the channel piers in extreme cases and, in the winter of 1905-6, piled high around the masonry, reaching up to the lower steel members then erected, but, fortunately, without injury to them, or to the tall steel and wooden false-work in position under the south anchor-arm, which endured the tremendous impact uninjured.

Substructure.—The substructure comprises two heavy U-shape abutments, two anchor-piers, and two main or river-piers, all of fine-dimensioned granite in large blocks, backed, in the piers, with concrete. The work on the north side of the river was built in the summer of 1904, and that on the south side in the summer of 1905, no work of this character being attempted during the severe winter or until after the river was clear of ice in the late spring. This work was executed under a contract, separate from that of the superstructure, which was awarded to Mr. M. P. Davis, Mr. A. A. Stuart, C.E., being the manager.

The two abutments each contain about 2500 cubic yards of masonry, and were built by ordinary plant and methods in open dry excavation. The anchorage-piers (see Plate XLVI. of September 6) each contain about 7000 yards of masonry, and 250 tons of steel girders and eye-bars; and as their footings are carried down only about 10 ft. below extreme high water, were built without difficulty in open excavations. They are 24 ft. by 105 ft. on top, 30 ft. 4 in. by 134 ft. at the bottom, and about 60 ft. high above extreme high water. The base is divided into two separate end portions by an arched centre opening of 25-ft. span, about 50 ft. high. A 36-ft. by 22-ft. reaction platform is built into the base of each portion of the pier, to provide for the maximum uplift of 6,000,000 lb. from each truss, which is resisted by an effective load of 15,000,000 lb. of superimposed masonry. The platform (Figs. 3 and 4, page 329 *ante*) consists of a course of horizontal transverse 15-in. I-beams seated on the top flanges of four longitudinal plate girders, $8\frac{1}{2}$ ft. deep and 36 ft. long, bedded solid in the concrete and connected with the end lower chord pins by vertical chains, each link of which is made with sixteen 10-in. by $2\frac{1}{16}$-in. eye-bars, the lower ends pin-connected to eight transverse distributing girders 8 ft. deep, and $17\frac{1}{2}$ ft. long, with their top flanges bearing against the bottom flanges of the reaction girders. They are set about 3 ft. clear of extreme high water and are proportioned for a maximum flange stress of about 273,000 lb. The reaction girders weigh about 23,000 lb. each, and are proportioned for 480,000-lb. flange stress. The spaces between and around the girders and eye-bars were hermetically sealed by concrete and grout, leaving them inaccessible. In Fig. 1, page 351, one of these anchorages and the north main pier are shown in course of construction.

The main or river piers are each 100 ft. by 28 ft. on top, with pointed up stream ends and battered faces, and are about 107 ft. in total height from the cutting edges of the pneumatic caisson foundations, which were carried down about 60 ft. below high water. Each of them contains about 16,000 yards of masonry and is subjected by the superstructure to a maximum vertical load of 56,000,000 lb., besides the heavy overturning moment due to wind pressure, current, and ice.

The 49-ft. by 150-ft. pneumatic caissons, 25 ft. high, are among the largest ever built, and were made chiefly of 12-in. by 12-in. southern pine timber, planed and framed by a steam plant installed at the site. The walls were made with horizontal outside and vertical inside courses of timber, drift-bolted together and sheathed outside with two crossed courses of diagonal planks spiked on. The working chamber, about 7 ft. high above the cutting edge, was made with three solid courses of timber, and its walls and ceiling were lined with caulked planks. The long walls were braced transversely by two vertical bulkheads across the chamber, each being made of double courses of 12-in. by 12-in. horizontal timbers, tied together by $1\frac{1}{2}$-in. vertical bolts, extending above the deck. The upper and lower courses were dovetailed into the wall timbers, and there were vertical bearing timbers bolted to the walls on both sides of the bulkheads.

The bottoms of the caisson walls were bevelled to a cutting-edge 9 in. wide, shod with a $\frac{3}{4}$-in. steel plate. The cutting-edge was stiffened laterally by 12-in. by 12-in. knee-braces, 3 ft. apart on centres, with their upper ends bearing against the vertical sides of the roof timbers parallel to the walls. There were also 16-in. by 16-in. longitudinal and transverse struts, 12 ft. apart in the plane of the cutting-edge, dovetailed and strap-bolted to the walls and halved at intersections, where they were braced by vertical struts and pairs of $1\frac{1}{2}$-in. vertical tie-rods.

Above the working chamber the open caisson served merely as a cofferdam, and was made with a single outside course of horizontal timbers and two courses of diagonal sheathing planks. It was braced by longitudinal and transverse 12-in. by 12-in. braces, 6 ft. apart in alternate courses, bolted together at intersections. The four lower courses in six lines of longitudinal struts were cross-braced with 6-in. by 12-in. planks, forming stiffening trusses.

Each caisson was provided with six 36-in. riveted steel air-shafts, having flanged cast-iron connections to the deck. The cross-section of the shaft cylinder had an offset recess for the ladder clear of the full-size buckets. The shafts were surmounted with air-locks $5\frac{1}{2}$ ft. in diameter and 12 ft. high, with an extension at the lower end to allow the double-hinged circular door to swing clear of the bucket. The upper doors were also made with two leaves, hinged to links moving in guides, to carry them back out of the way of the descending buckets. All doors closed on rubber gaskets, and were operated by shafts counterbalanced at both ends. The bucket hoisting-rope passed through a long stuffing-box in the centre of the upper door, and all joints were so close that very little air-pressure was lost through them.

Water at about 100 lb. pressure for jetting was distributed to the working chamber through six 3-in. vertical branches from a 4-in. main on the deck. Air-pressure was supplied through a 6-in. pipe terminating just below the ceiling in an elbow, with the horizontal end fitted with a leather-faced iron disc hinged at the top, to make a simple check-valve automatically closing to prevent the escape of the pressure, and admitting it freely. Six vertical 4 in. blow-off pipes passed through the deck, terminating above the top of the cofferdam, with goose-necks discharging over the sides, and at the bottoms having plug-valves with a clear way for the passage of sand, mud, small stones, &c. In order to diminish the specific gravity of the ascending column enough to permit it to be forced above water-level by the caisson pressure, a small hole was bored in each blow-off pipe above the inlet, and provided for the aeration of the ascending water.

The caissons were built on shore nearly a mile from the pier sites, launched from ways and towed to the sites, drawing about 12 ft. of water, so that they grounded at low tide in the 10-ft. water. They were filled with concrete sufficient to overcome their buoyancy before excavation was commenced under them. They were sunk by ordinary methods to a depth of about 50 ft., and filled with concrete, which, in the working chambers, was allowed to set under full air-pressure before the air-locks and shafts were removed. The concrete was made from stone quarried from the face of the cliff on the north side of the river, and delivered by gravity to a steam stone-crusher with a capacity of 30 cubic yards per hour. The crusher discharged to a small collecting-bin, with a trap-door, through which the stone was drawn to the charging-hopper of a mixing-machine that delivered to skips on cars running on a 500-ft. trestle to a working-platform surrounding the north pier.

Sand was delivered by carts from the river-shore, where it was dug, to a link-belt bucket-elevator, which deposited it in a rotary screen on the mixing-platform, where the gravel was separated from it. The stone and concrete were handled entirely by gravity, and all the concrete plant, including two derrick engines, was operated by steam from a 100-horse-power boiler, which supplied a high-speed engine located under the crusher platform.

The numerous boulders on the river-bottom made it hazardous to unload materials from boats near the pier except at low water, when the cut stone was delivered by scows to the pier-derricks, and stored in sufficient quantities for several days' supply on the very strong pier-platform, 40 ft. wide, built around the pier on piles. Steel and cement for the north pier and abutment were landed on a dock 4000 ft. from the bridge, and delivered to the site by a double-track railroad. The compressor plant was located on the working platform of the north pier, and comprised four 100-horse-power boilers, three duplex air-compressors, having cylinders 16 in. and 18 in. in diameter, two high-speed engines and electric generators for the lighting plant, two hoisting-engines, pumps, and feed-water heater. The plant for the south pier and abutment was similar, but different in arrangement.

General Design of Superstructure.—The superstructure has two pin-connected trusses (see page 328 *ante*), 315 ft. in maximum depth and 2800 ft. long between centres of anchorage pins. These trusses are in vertical planes, 67 ft. apart, and form two 500-ft. anchor-arms, two $562\frac{1}{2}$-ft. cantilever arms, and one 675-ft. suspended centre span, itself $128\frac{1}{2}$ ft. longer than the Louisville span, built at the same shop, and longer than any other free truss-span yet built. The bottom chords of the suspended span are horizontal, all other top and bottom chords are parabolic arcs, carefully designed for economy of material and artistic outlines. There is a clearance 150 ft. high and 1200 ft. long over the centre of the channel. The trusses carry a single floor on a 1 per cent. grade at an elevation of 160 ft. above water-level. This supports, between trusses, two railroad tracks, two street-car tracks, and two highways, and is intended for future cantilever side-walk extensions to a total width of 82 ft. The strain-sheet of the anchor-span, showing the live, dead, and wind-load stresses in the bars, is represented in Fig. 9, Plate XLVII., whilst Figs. 8 and 10 show the arrangement of the upper and lower lateral bracing.

The trusses are connected by lateral diagonal systems in the planes of the top and bottom chords and roadway (see Plate XLIV. of September 6), and by transverse sway-bracing between all vertical posts. All secondary connections are made with riveted members and with riveted connections. All members were completely finished—except that some of them were shipped in several field-spliced lengths—at the shops at Phœnixville, Pa., about 600 miles in an air line from the site, and were shipped and erected as units, the maximum dimensions and weights of which were limited by the capacities of the rolling-mills, machine-tools, and special transportation facilities. Single pieces were over 100 ft. long, and weighed 105 tons (see Plate XLIV., September 6).

The two vertical posts and bearings and transverse bracing on each main pier (Figs. 10 and 11, Plate XLVII.) make a bent nearly 415 ft. high by 75 ft. wide over all, which is heavily trussed to carry a total load of 56,000,000 lb., and weighs about

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3,000,000 lb.—more than a first-class double-track 400-ft. span railroad bridge. The weights of each anchor and cantilever arm and of the suspended span are about 12,500,000 lb., 15,000,000 lb., and 6,000,000 lb. respectively, exclusive of the 8,000,000-lb. floor system.

The general character of the design, type of trusses, and essential features of details and connections of members, correspond to recent high-class railroad-bridge construction in the United States, modified only as necessitated by the unusual magnitude of dimensions and stresses. The structure was designed to standard specifications, and the members were detailed by the regular staff of the bridge company under the direction of Mr. P. L. Szlapka, designing engineer, and Mr. C. Scheidl, assistant engineer, and verified by Mr. Bernt Berger, chief of staff of the consulting engineer.

Great care was taken to avoid ambiguity or eccentricity of stress, and secondary stresses; to secure minimum bending moments; to devise the most simple, economical, and direct connections of maximum efficiency; to avoid the use of special materials; to adapt the work to the standard practice in mills and shops, and the most advantageous use of existing machine-tools; to suit the character and arrangements of connections, members, and the details of connections to the requirements of shipment and handling.

Final designs for the largest members were not approved until sketches had been submitted to the railroad company, and assurance received that they could be transported on standard or special cars. The designs were modified, if necessary, in conference with the consulting engineer, the erection department, and the railroad representatives, and all members were finally built with the regular shop plant simultaneously with the fabrication of a large tonnage of ordinary heavy work on other contracts.

Shipping weights and dimensions were reduced by subdividing the lengths of the largest members to the maximum for shipment and for field assembling without disproportionate increase in erection apparatus and stresses. Some of the longest members were loaded with pivoted sliding bearings on two 33 ft. flat cars, separated by an idle spacer car, and carefully calculated to ride round the sharpest curves in the track.

Some shorter pieces were too wide, high, or heavy for clearance of bridges and tunnels when loaded on ordinary cars, and for these the railroad company designed special steel cars of great capacity, with distributing girders and open floors, through which the pieces could project nearly to rail-level. The longest tension and diagonal members were made in several lengths, spliced in the field, the former with pins, and the latter with riveted cover-plates; they were assembled with service bolts, and afterwards field-riveted at convenience.

Pieces and parts of pieces were made as nearly duplicate as possible, and care was taken to plane abutting edges and surfaces for perfect bearings, and to arrange rivets to develop the full strength of the section with the least possible diminution of cross-section, and to locate them accessibly for advantageous driving; where this was impossible they were replaced by turned bolts, about 500,000 of which were used.

To facilitate erection, and reduce the moments on the main pins, the trusses were designed to require the minimum simultaneous pinning, and to reduce the lengths and moments of the top-chord pins they were made double at each panel point, each connecting one panel of top-chord eye-bars to one end of a transverse link riveted to the top of the vertical post, and having a third separate pin for the top of the diagonal member, as shown in Fig. 9, Plate XLVII. Similarly, the lower ends of the vertical posts have one pin connecting them to the bottom chord, and a separate pin connecting them to the diagonal member, care being, of course, taken to have the centre lines of the members at the same panel point intersect in a common point.

The directions of the lower chords change at panel points where all materials make a butt-joint shop-riveted, while a field-riveted joint 10 ft. beyond it allows all the members of one panel to be assembled and connected in erection before those of the next panel are assembled. The anchor (Fig. 2, page 354) and cantilever arms are divided into uniform panels of 50 ft. and 56¼ ft. respectively, with diagonal members reaching across two panels and connected at intermediate points to sub-diagonals, while the vertical posts are braced by secondary horizontal and diagonal members with riveted connections.

Clearances between riveted members vary from ⅛ in. to ½ in.; eye-bars 2 in. thick are allowed to vary only $\frac{1}{16}$ in. in the head for eccentricity and thickness, and only $\frac{1}{64}$ in. in length; only $\frac{1}{16}$ in. is allowed for packing on the chord pins. All clearances were very carefully checked on the drawings and measured in the finished members, which are marked with identification letters and numbers, according to a comprehensive system designating them on skeleton erection diagrams, on which they were checked off at all stages of manufacture, shipment, storage, and erection, so that the condition of the work was always simply and graphically recorded. Main-chord pins from 12 in. to 24 in. in diameter were fitted to within $\frac{3}{64}$ in., and all centre lengths were measured in the shops with steel bars standardised with the steel tapes used in the field, and carefully maintained at the same temperature as the members themselves, to eliminate errors from expansion and contraction.

Materials.—As required by the specifications, all material is soft or medium open-hearth steel made by the acid method and containing not more than 0.06 per cent. of phosphorus. No work was allowed on it near blue temperature, or between that of boiling water and the ignition of hard-wood sawdust. All material was accepted on tests made from pieces not less than 12 in. long and ½ square inch in cross-sectional area. The medium steel used has an ultimate strength of 60,000 to 68,000 lb.; elastic limit not less than half the ultimate; and a minimum elongation of 22 per cent. in 8 in. It endures without cracking, after heating to cherry-red and quenching, bending to a circle three times its own thickness.

The soft steel used has an ultimate strength of 54,000 to 62,000 lb.; elastic limit not less than half the ultimate; minimum elongation of 25 per cent. in 8 in., and, after heating to yellow and quenching, bends, without cracking, to a circle twice its thickness. The specifications demanded that all steel must endure having ordinary rivet-holes drifted to a diameter one-third greater than the original, without cracking, in the periphery of the hole or in the external edge of the piece.

The rivet steel used has an ultimate strength of 50,000 lb. to 58,000 lb.; elastic limit not less than half the ultimate; elongation of 26 per cent. in 8 in., and must bend 180 deg., close shut, without cracking.

Eye-bar steel, 1½ in. thick or less, was required to endure rivet-steel tests; for thicknesses greater than 1½ in. a reduction of 1 per cent. in the elongation was allowed for each ⅛ in. additional thickness up to a total thickness of 3 in. Full-size eye-bars were required to develop at least 10 per cent. elongation in the body of the bar between centres of pins, and a minimum ultimate strength of 56,000 lb. per square inch. The tension tests, made in a direct-acting hydraulic horizontal machine of 1,000,000 lb. capacity, developed an average total ultimate strength of 58,000 lb. per square inch, and an elongation of 20 per cent. in a measured distance of 18 ft. in several bars tested, the average of the preliminary material tests having been 66,000 lb. ultimate strength, 36,000 lb. elastic limit, and 25 per cent. elongation. Several eye-bars were tested to destruction, but none of them broke in the heads, which had an excess of about 40 per cent. cross-section through pin-hole.

Specifications.—The design was made in accordance with Mr. Cooper's 1901 specifications, slightly modified to correspond with the unusual dimensions and stresses of the members and the proportions and characters of the dead and live loads. All lateral, sway, and portal bracing was designed to resist both compression and tension. Engine-wheel loads were assumed to be distributed over three track ties. Variations of 150 deg. temperature were provided for.

The maximum unit tension stresses permitted in medium steel are:—6000 lb. for members liable to sudden loading; 20,000 lb. wind stress in longitudinal, lateral, and sway bracing; 12,000 lb. live-load stress in longitudinal and sway bracing; 10,000 lb. in bottom flanges of floor-beams and stringers; 10,000 lb. live-load and 20,000 lb. dead-load stresses in bottom chords, main diagonals, counters, and long verticals. Sectional areas of members are the sums of the separate areas required for the dead and live-load stresses. Soft-steel stresses, 10 per cent. less than medium-steel stresses.

Maximum unit stresses in medium-steel compression members:—

Chord segments $\begin{cases} P = 10{,}000 - 45\dfrac{l}{r} \text{ for live loads.} \\ P = 20{,}000 - 90\dfrac{l}{r} \text{ for dead loads.} \end{cases}$

All posts ... $\begin{cases} P = 8{,}500 - 45\dfrac{l}{r} \text{ for live loads.} \\ P = 17{,}000 - 90\dfrac{l}{r} \text{ for dead loads.} \end{cases}$

Lateral struts and rigid bracing, $P = 13{,}000 - 60\dfrac{l}{r}$ for wind strains.

$$P = \tfrac{2}{3}\left(13{,}000 - 60\frac{l}{r}\right) \text{ for live loads.}$$

P is the allowed strain in compression per square inch of cross-section in pounds.
l is the length of compression member in inches; $c-c$ of connections.
r is the least radius of gyration of the section, in inches.

No compression member has a length exceeding 100 times its least radius of gyration for main members, or 120 times for laterals. Soft steel is used in some compression members, with unit strains 15 per cent. less than those used for medium steel.

All members and their connections subject to alternate strains of tension and compression are proportioned to resist each kind of strain; both of the strains are, however, considered to be increased by an amount equal to 0.8 of the least of the two strains.

The strains in the truss members from the assumed wind forces are not considered, except as follows:—First, when the wind strains on any member exceed 30 per cent. of the maximum strains due to the dead and live load on the same member, its section is increased until the total strain per square inch does not exceed by more than 30 per cent. the maximum fixed for dead and live loads only. Second, when the wind strain alone, or in combination with a possible temperature strain, can neutralise or reverse the strains in any member.

The rivets in all members, other than those of the floor and lateral systems, are so spaced that the shearing strain per square inch does not exceed 9000 lb., nor the pressure on the bearing surface of the rivet-hole exceed 15,000 lb. per square inch. The rivets in all members of the floor system are so spaced that the shearing strains and bearing pressures do not exceed 80 per cent. of the above.

An increase of 50 per cent. of the above stresses is allowed for rivets in the lateral and sway bracing. Stresses in field-driven rivets and bolts are reduced one-third of the above. Pins are proportioned for maximum shear of 9000 lb., 15,000 lb. bearing on riveted members, and 18,000 lb. bearing stress. The inclined posts, subjected to both axial and bending stresses, are proportioned so that the maximum fibre stresses do not exceed the limiting compression stress. Where the fibre stress resulting from weight only exceeds 10 per cent. of the allowed unit stress of any member the excess is considered in proportioning the sectional area.

Floor beams and stringers are proportioned for the bending moment to be resisted entirely by the flanges and the shear entirely by the web-plate; the distance between the centres of gravity of the flanges is taken as the effective depth, and the gross areas of both flanges are equal. The webs are stiffened at maximum intervals of 5 ft., and the stiffeners are proportioned to carry the maximum vertical shear with a maximum stress of $P = 10{,}000 - 45\dfrac{l}{r}$.

Loading.—The dead load at each panel point was carefully estimated per lineal foot of the structure, on a basis of 2500 lb. for lumber, 3700 lb. for steel in the floor system throughout, and for trusses and bracing 7800 lb. in the suspended span, 18,510 lb. in the cantilever arm, and 17,880 lb. in the anchor-arm, figures which were very closely verified by the final computations from the actual sections.

Three cases of live loading were assumed for the stresses in the main trusses: (*a*) 3000 lb. per lineal foot for a train of unlimited length on each railroad track; or (*b*) for 900 ft. a train-load of 3300 lb. headed by two "E 33" locomotives (*i.e.*, 33,000 lb. on each of four pairs of drivers 5 ft. apart, and a total wheel base of 48 ft.) on each track; or (*c*) a 550-ft. train-load of 4000 lb. per lineal foot headed by one "E 40" locomotive on each railroad track (40,000 lb. on each pair of drivers, and 26,000 lb. on each of four tender axles, 48 ft. wheel base). No loading was assumed on the electric tracks, roadways, or sidewalks in con-

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THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

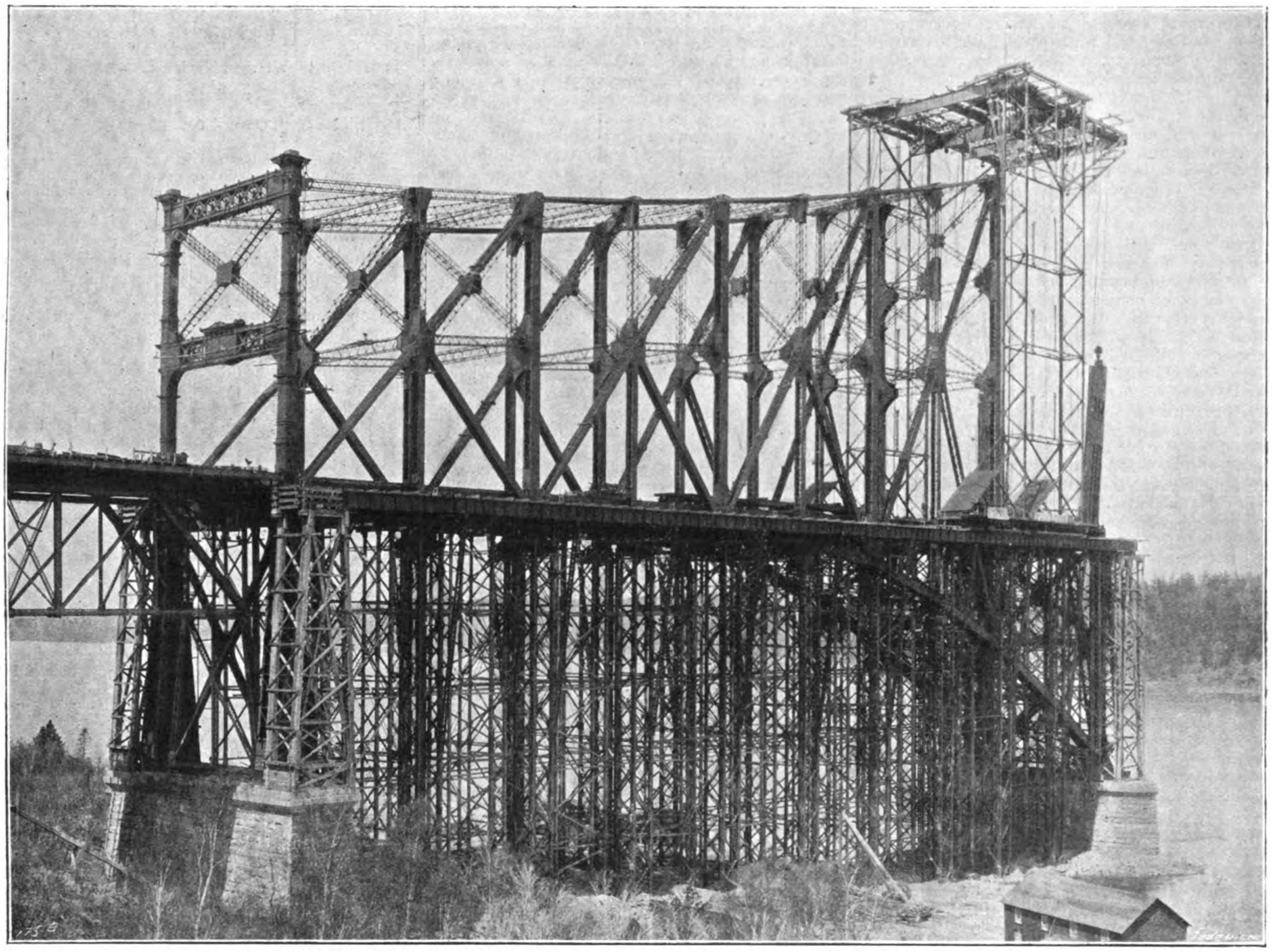

Fig. 2. Last Panel of South Anchor-Arm.

nection with the above loads on the trusses For the hangers, sub-diagonals, and floor system the assumptions were:—(a) A train-load of 4000 lb. per lineal foot, headed by two "E 40" locomotives on each railroad track; or (b) on each electric-car track a train-load of electric cars, each 30 ft. long, and weighing 56,000 lb. on two axles 10 ft. apart; or (c) on each roadway a concentrated load of 24,000 lb. supported on two axles 10 ft. apart. The wind-pressure per lineal foot resisted by the top lateral system was assumed at 500 lb., and that resisted by the bottom laterals at 1000 lb.

(*To be continued.*)

[In view of the great interest excited by the catastrophe, we publish this week many illustrations in advance of the reference to them to be given in the continuation of Mr. Skinner's article. It may be of service to our readers, therefore, if we give a short description of these here. In the meantime it will be noted that the working stresses tabulated by him do not quite agree with those figured in the strain-sheet, Fig. 9, Plate XLVII. The difference presumably arises from the specification quoted by Mr. Skinner, being Mr. Cooper's 1901 standard, which, as stated, was modified somewhat in view of the enormous proportions and unusual character of the St. Lawrence Bridge. Fig. 1, page 351, has already been referred to in Mr. Skinner's text. The shore span in course of construction is represented in Fig. 2. The false-work extended on both sides of the bridge and supported a traveller as shown. This is, we believe, the 1000-ton traveller, though on the approach span the one used was a much

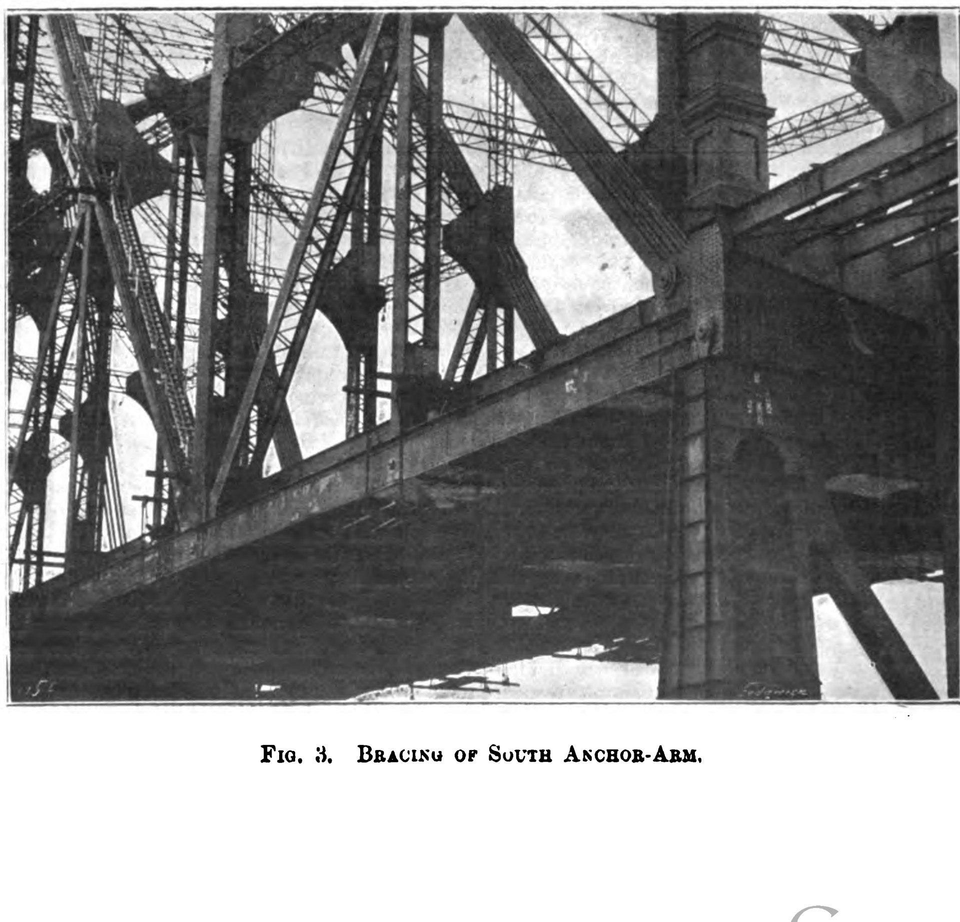

Fig. 3. Bracing of South Anchor-Arm.

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THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

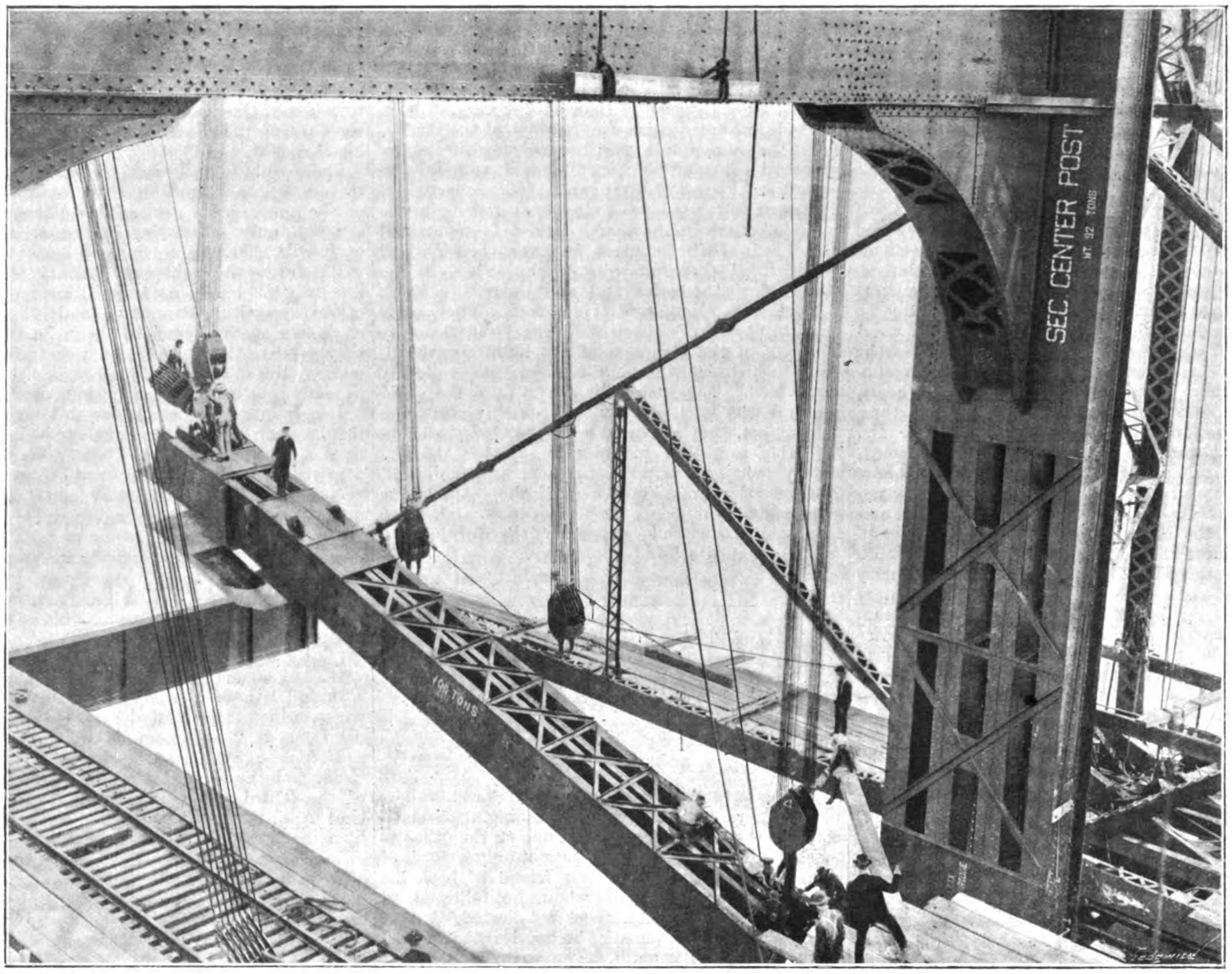

FIG. 4. FIRST LOWER CHORD PANEL OF SOUTH ANCHOR ARM.

lighter structure. In Fig. 3 the arrangement of the girder-work above the anchor-span is represented, but the best idea of this will be obtained from Figs. 16 and 17 of Plate XLVIII., which shows the portal complete. The upward pull, due to the weight of the river-span, is transmitted to the anchorage through the inclined eye-bars shown in Fig. 16, and to an enlarged scale in Fig. 18. The direction of these bars passes, it will be seen, through the anchor-pin, so as to avoid any bending stresses in the riveted work. This precaution is typical of the care taken throughout to avoid any secondary bending stresses in the material. This is undoubtedly judicious, though it is equally certain that its importance is often exaggerated. We also show in Fig. 19, to the same enlarged scale, the joint between the end cross-girder and the lower chord of the truss. From Fig. 11, Plate XLVII., it will be seen that a steel trestle-bent intervened between the end of the bridge and the anchorage proper. Photographs of the scene of the disaster show that this was overturned bodily, but the anchor pier seems uninjured, and the approach span remained untouched. Cross-sections through the cantilever at different points are represented in Figs. 12, 13, and 14. The great length of the pins at the top of Fig. 12 is noteworthy. Plans of the upper and lower wind bracing are represented in Figs. 8 to 10 on the same plate.

The putting in place of the first panel length of the lower chord is represented in Fig. 4, above. The weight handled, it will be observed, was 100 tons. Note should be taken of the fact that the piece includes a portion of the member for the next panel, the joint-pin being some distance from the extreme end. The object of this, as explained in Mr. Skinner's article, was to simplify the erection of the bridge. Each panel could be completed at one operation, so far as the lower chord was concerned, since the "field joints" for this member were not pin joints, but riveted "false" joints, located some distance away from the pins into the next panel.

In Fig. 5, page 358, the erection of the third section of the main posts over the river-pier is represented. This figure should be compared with Figs. 20 and 21, Plate XLVIII. The main shoe and upper and lower pedestals at this pier are represented in Fig. 6, page 358, whilst Fig. 7 shows the same to a smaller scale, after the erection of the main post and completion of the cantilever arms. The pin making the connection between the post and the shoe is 24 in. in diameter.]

TESTS OF EMERY GRINDING-WHEELS.

ON behalf of the Verein Deutscher Ingenieure, and with the assistance of Messrs. Ludw. Loewe and Co., of Berlin, Dr. G. Schlesinger has conducted some tests on the durability and safety of emery grinding-wheels, which he describes in No. 43 of the *Mitteilungen über Forschungsarbeiten*. More than half of the pamphlet of 60 pages is taken up by plates of curves. One of the objects of the research was to demonstrate that the regulations laid down by the Prussian Government in 1897 as to the maximum circumferential speed of such wheels—viz., 25 metres (82 ft.) per second—should be relaxed. Those regulations had been fixed with the approval of engineers and manufacturers. The manufacture of emery wheels had been taken up by incompetent and inexperienced men, and serious accidents had occurred, so that restriction was needed. The emery wheels and the grinding-machines have, however, been so much improved of late that a revision of the regulations appeared opportune. Some laboratory tests on grinding-wheels, made a year or two ago by Grübler, were not of sufficiently practical a character. Dr. Schlesinger reviews these trials, and also the work of Codron, of Paris, of 1902.

Assisted by four engineers and two men, Dr. Schlesinger has, in his trials, adopted workshop conditions, except that he disregarded the safety regulations prescribed by law. Most of the twenty-eight wheels experimented upon were not especially ordered, but taken from the stocks of good firms. The wheel and the test-piece, of cast iron, wrought iron, or steel, were both rotated, and the experiments continued during ten hours for two or four days. The experiments lasted three months. No accident occurred, and in no case did the wheel fly apart, although the speed was raised to the maximum which the driving electric motor permitted. Under the continued strain the wheels began to crumble, emery grains breaking off. The wheels were cylindrical, 500 millimetres (20 in.) in diameter, and 2 in. in width, and were revolved at speeds of 25, 30, and 35 metres (82 ft., 99 ft., and 115 ft.) per second. The test-piece was fed along at different rates, as a rule 12, 18, and 24 millimetres (½ in. to 1 in.) per revolution of the test-piece, which was revolving at a maximum circumferential speed of 30 metres per minute. In extreme cases the grinding-wheel absorbed 30 horse-power, while in practice 6 horse-power will hardly be exceeded. This was done in the destruction tests, during which the grinding-wheel abraded a spiral of its own width on the test-piece, the feed rate being 2 in. per revolution of the test-piece. One of these destruction tests gave the following results :—The diameter of the wheel before the test was 488 millimetres ; after the 2 minutes 55 seconds of the trial the diameter

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THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(For Description, see Page 351)

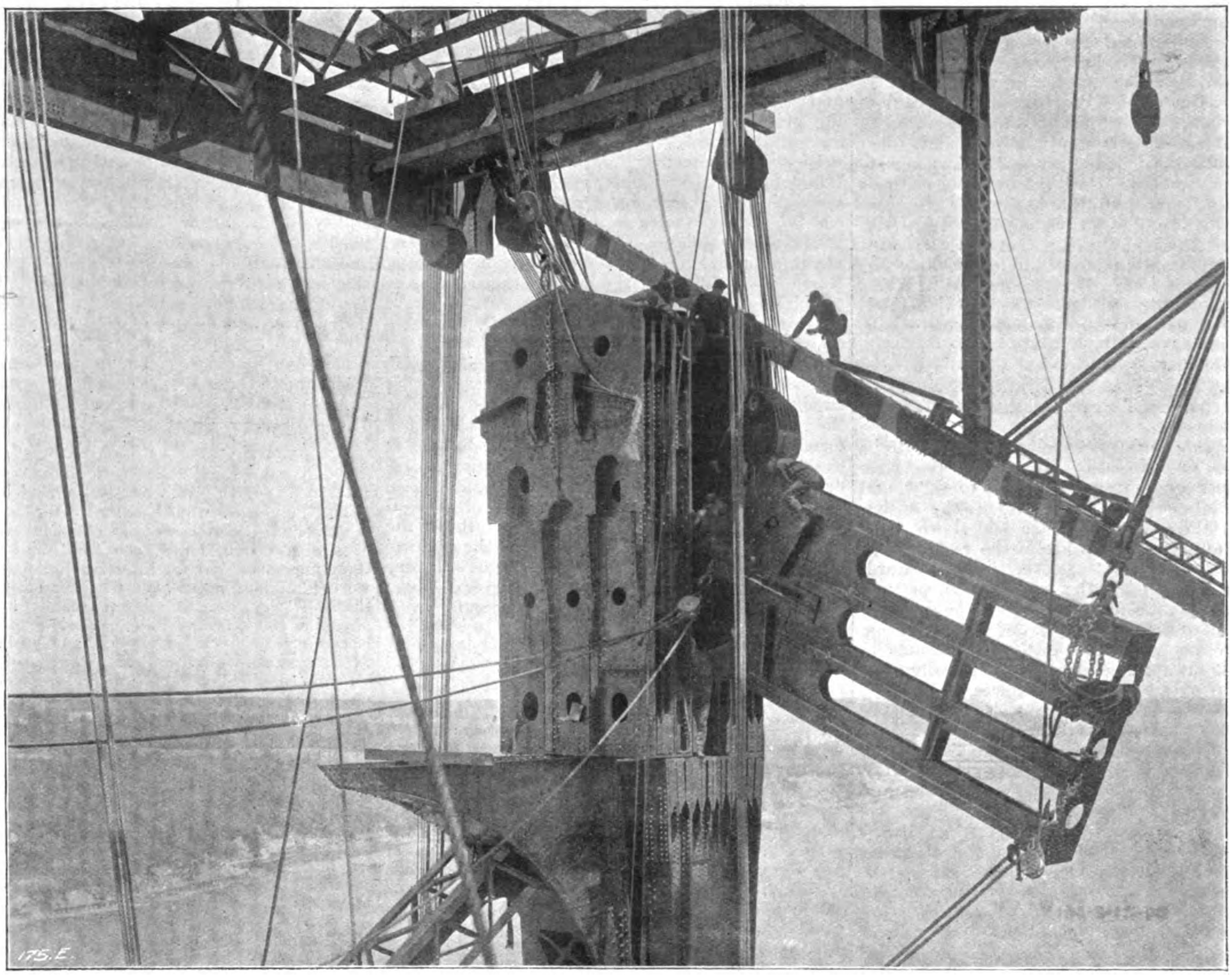

Fig. 5. Erecting Third Section of Main Post Cap.

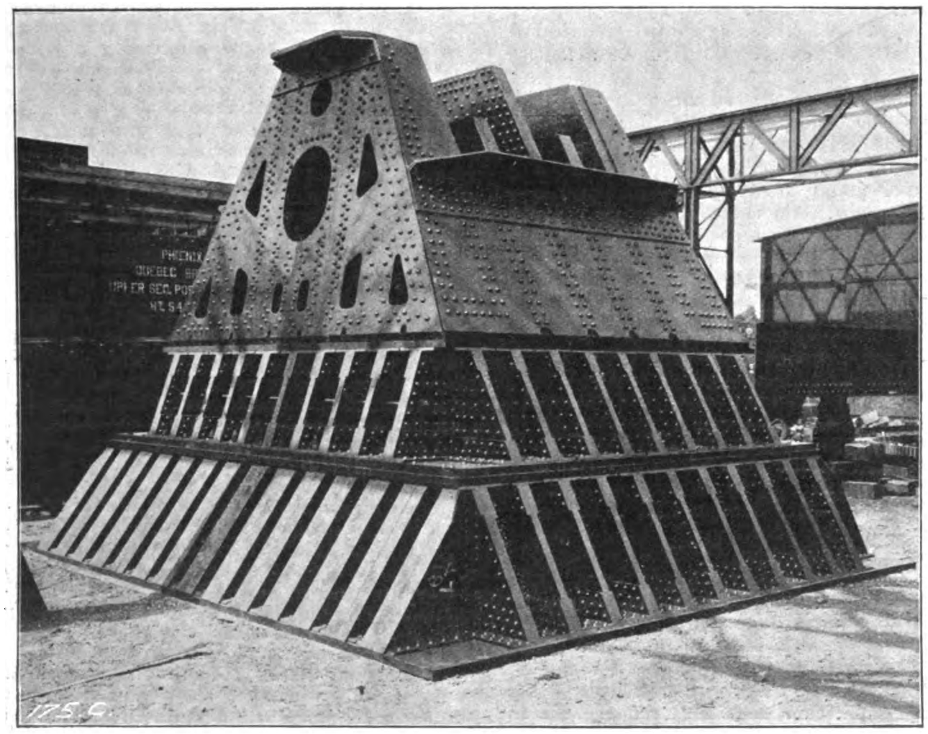

Fig. 6. Main Shoe and Upper and Lower Pedestals.

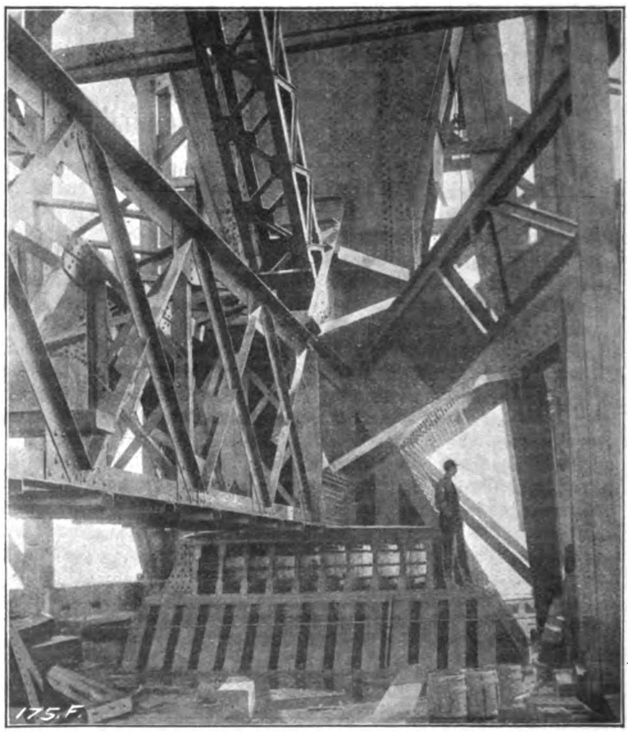

Fig. 7. Connections at Foot of East Main Post.

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THE WRECK OF THE CANTILEVER BRIDGE AT QUEBEC.

(*For Description, see Page* 371.)

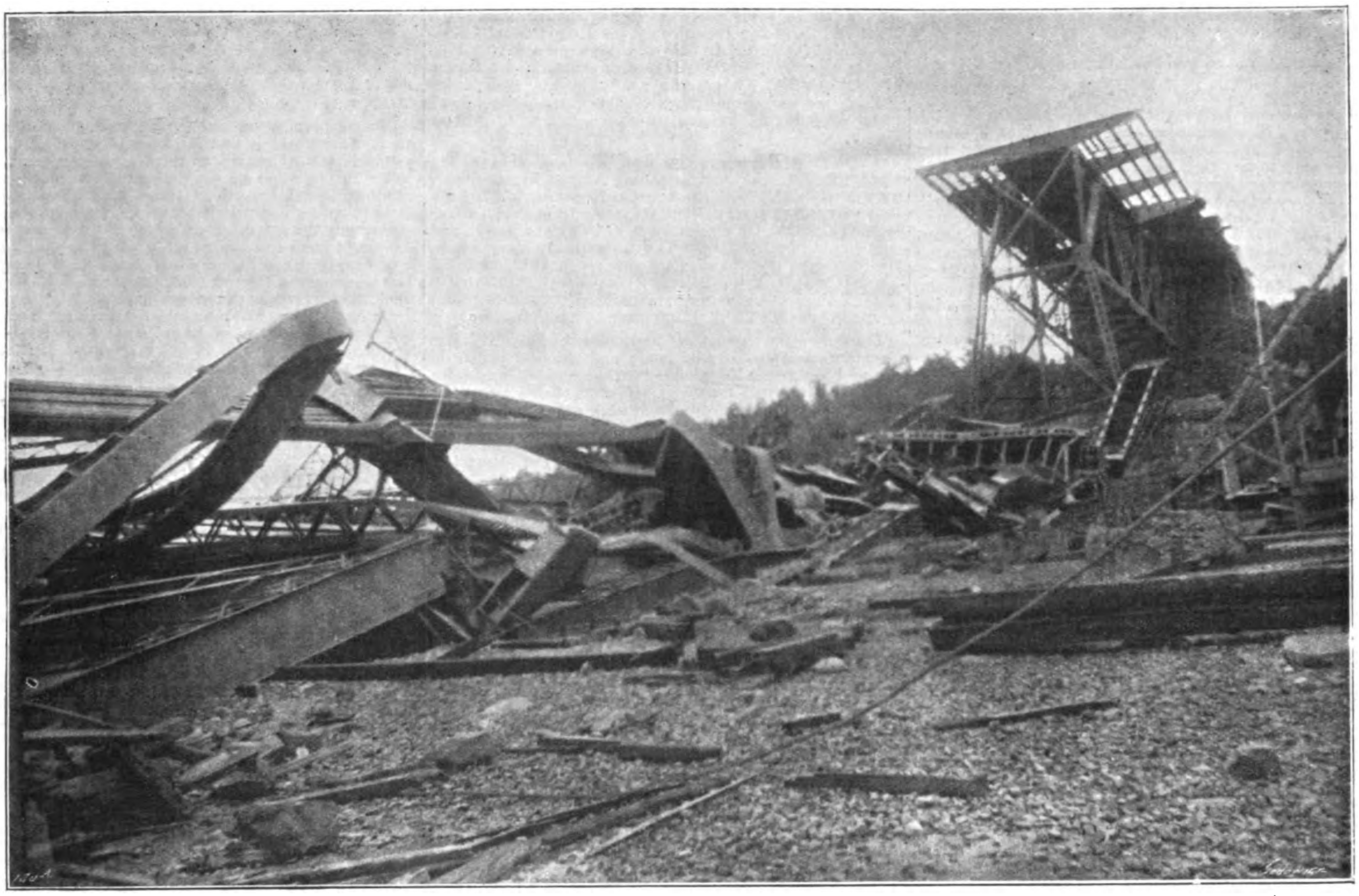

Fig. 1. South Anchor-Arm and Anchorage Pier.

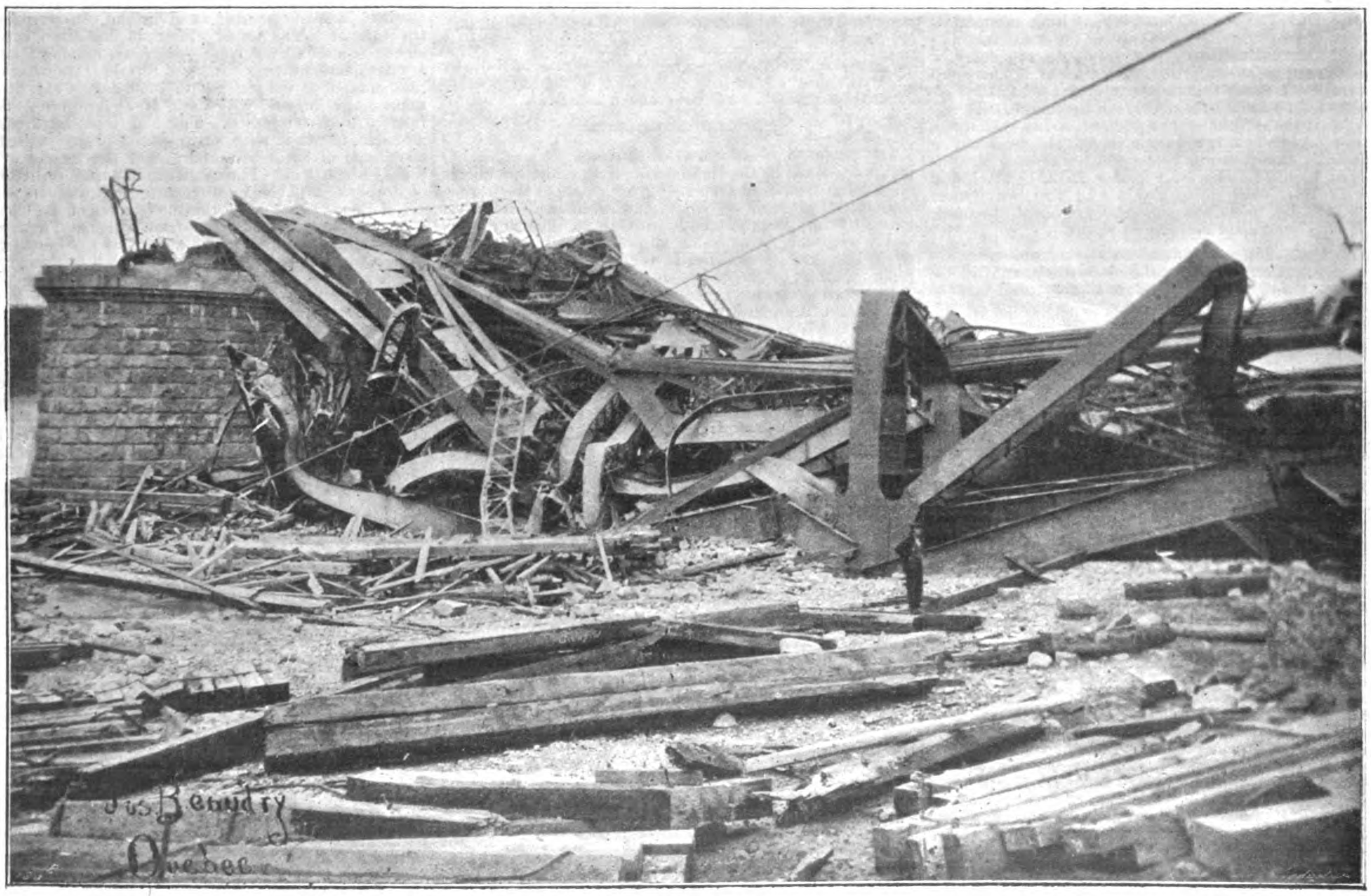

Fig. 2. South Anchor-Arm and South Main Pier.

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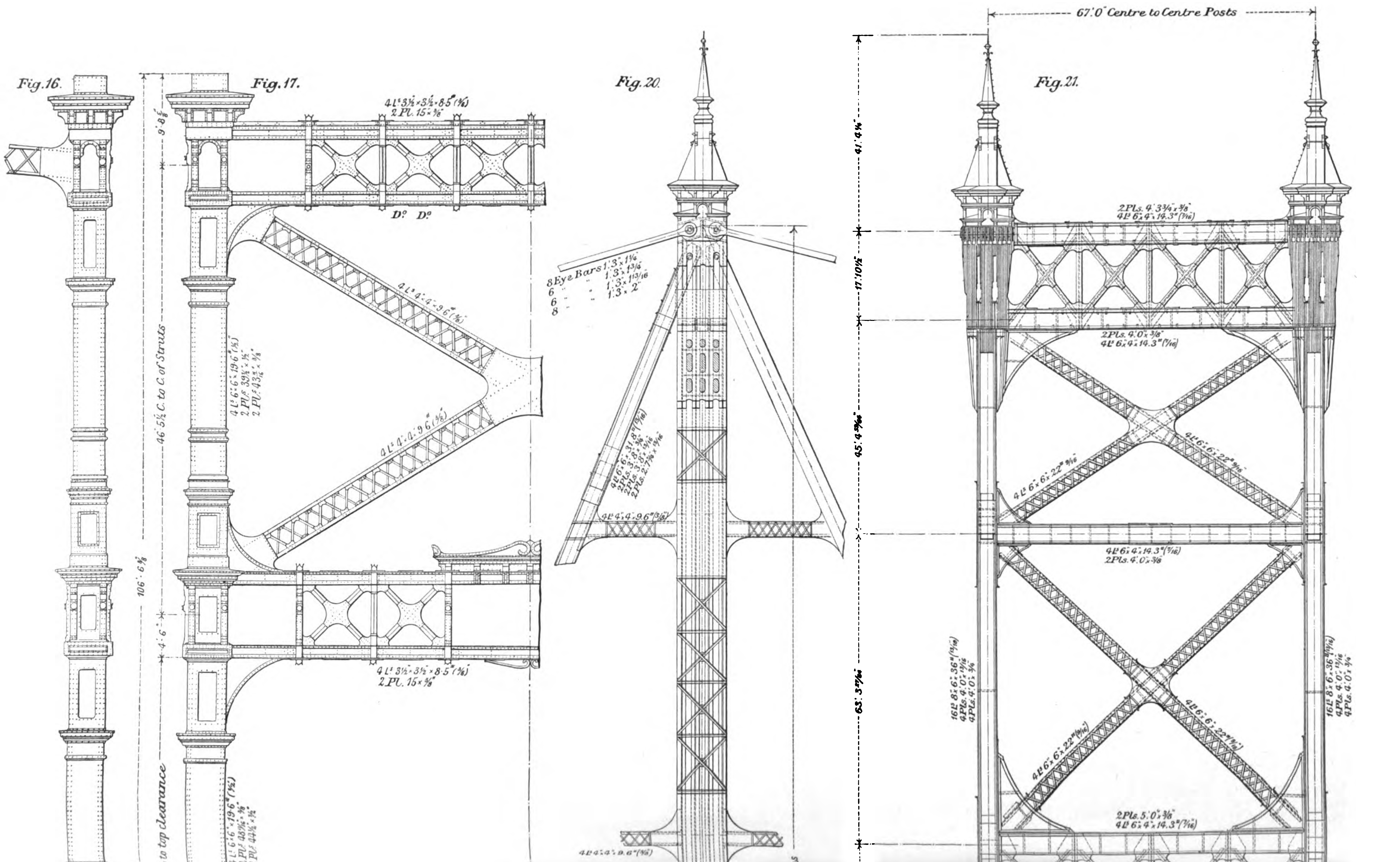
THE CANTILEVER BRIDGE ACROSS THE ST. LAWRENCE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(*For Description, see Page* 351.)



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THE CANTILEVER BRIDGE ACROSS THE ST. LAWRENCE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(*For Description, see Page 351.*)

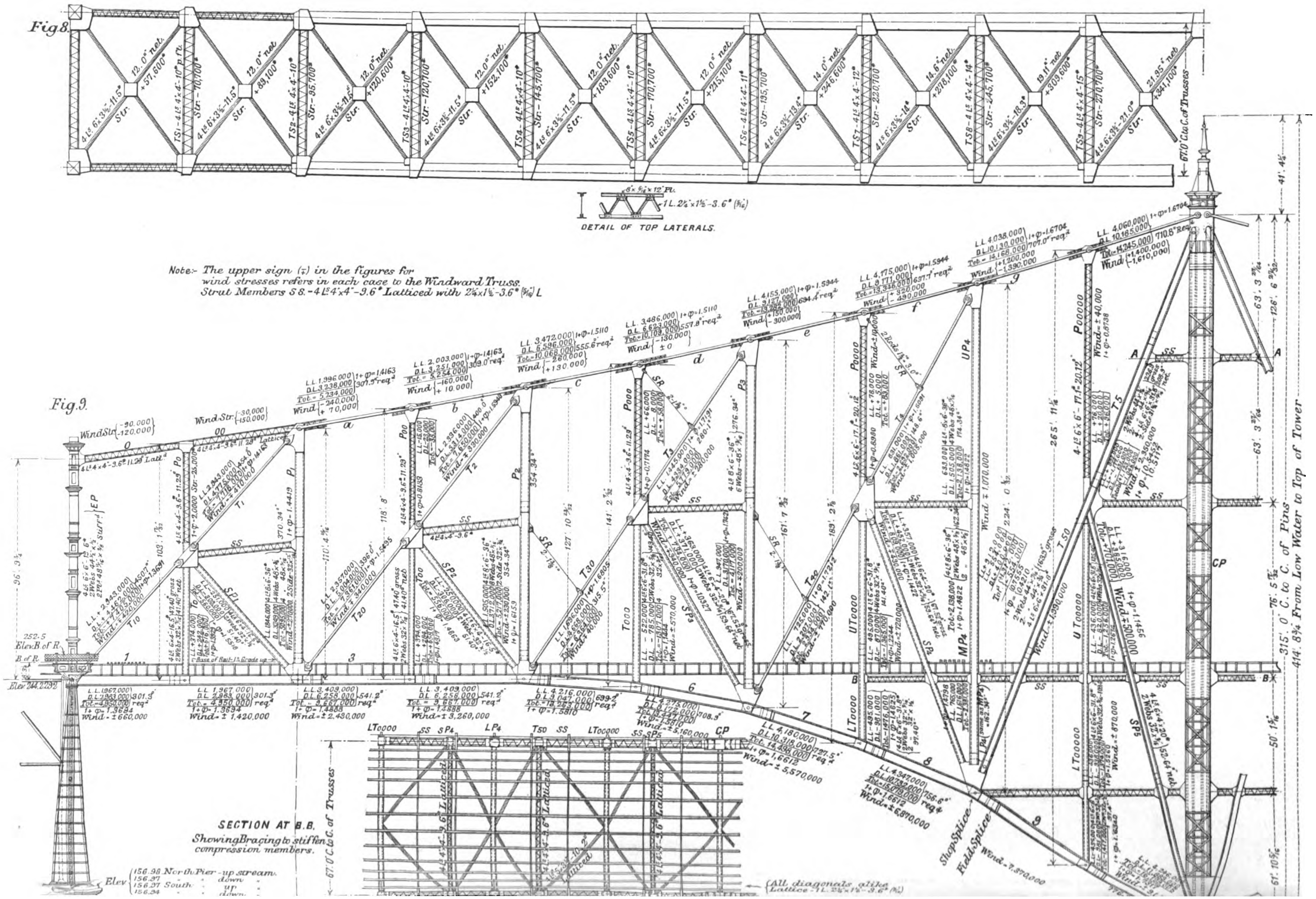

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Fig.10.

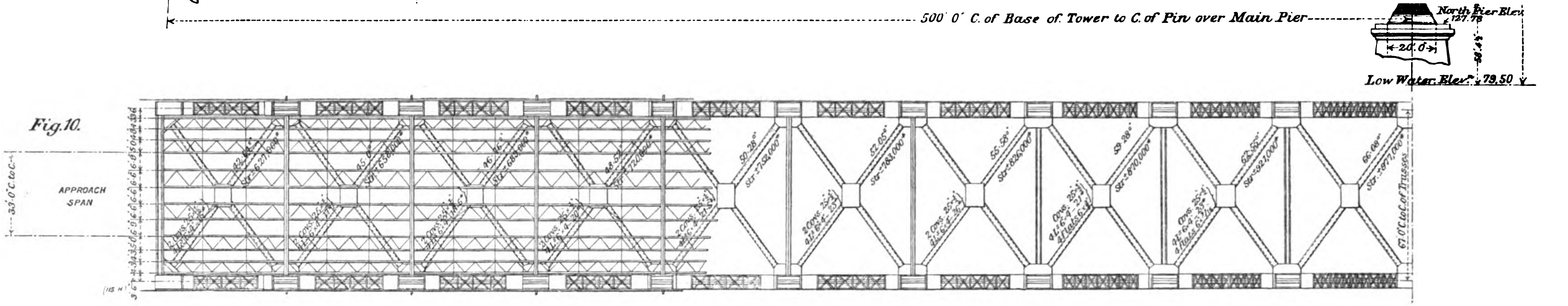


Fig. 11.

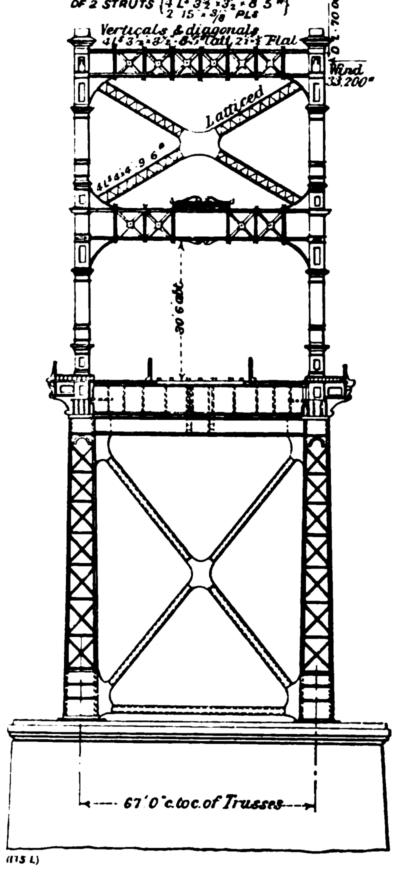


Fig. 12.

SECTION AT P00000 - T00000.

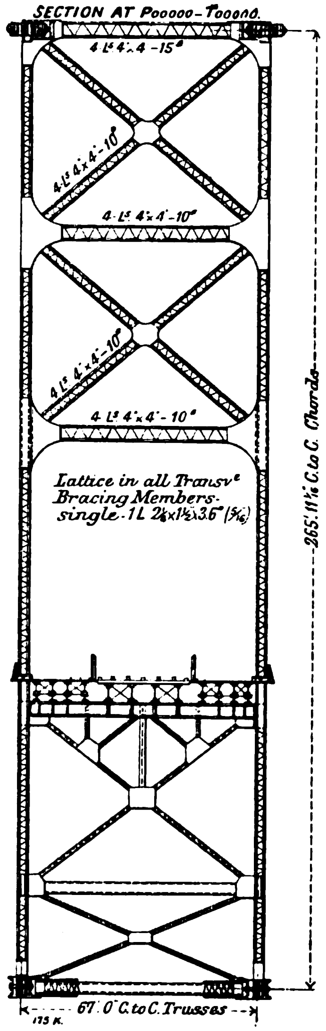


Fig. 13.

SECTION AT P4 & P0000 - T0000.

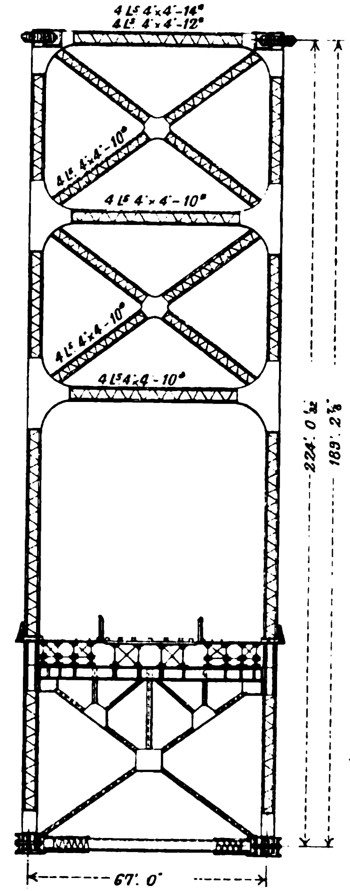


Fig. 14.

SECTION AT P3.

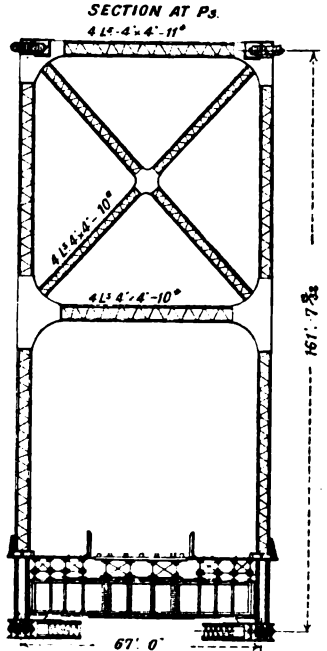
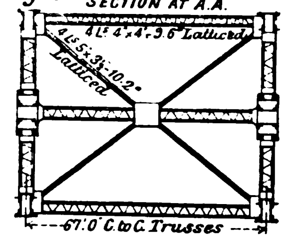


Fig. 15

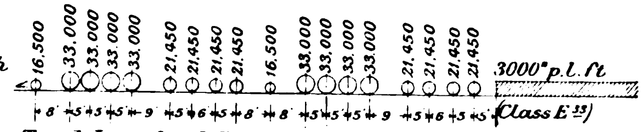
SECTION AT A.A.



LIVE LOAD: I. TRUSSES IN GENERAL

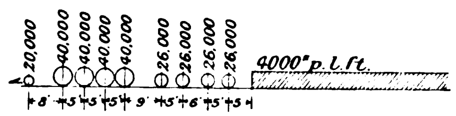
(a) On each R.R. Track 3000 lbs. per lin. ft. unlimited length

or (b) On each R.R. Track



Total Length of Train - 900' 0"

or (c) On each R.R. Track

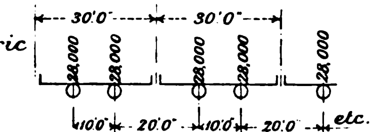


Total Length of Train - 550' 0"

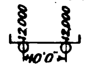
No loading on Electric Tracks, Roadways or Sidewalks

II. HANGERS & SUB-DIAGONALS

(a) On each R.R. Track - 2 - E40 × 4000 lbs. p. l. ft. & (b) On each Electric Track 56,000 lbs. on two Axles.



(c) On each Roadway - 24,000 lbs. on two Axles



III. FLOOR SYSTEM. SAME AS II.

DEAD LOAD: *Lumber - 2500 p. l. ft. Fl. Bms.*

Floor	*- 3700 - - "*	
	6200 - - Tot. from	
Trusses & Bracing	*7800 - - are on Susp. Span*	
	18510 - - - - Cant. Arm	
	17880 - - - - Anchor -	

Wind Bracing { Top Lat'ls 500 lbs. / Bott. " 1000 lbs.

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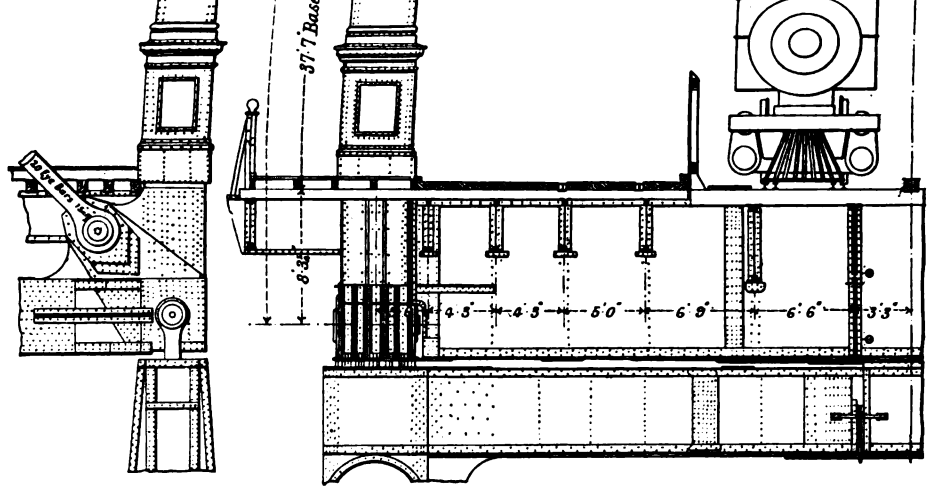


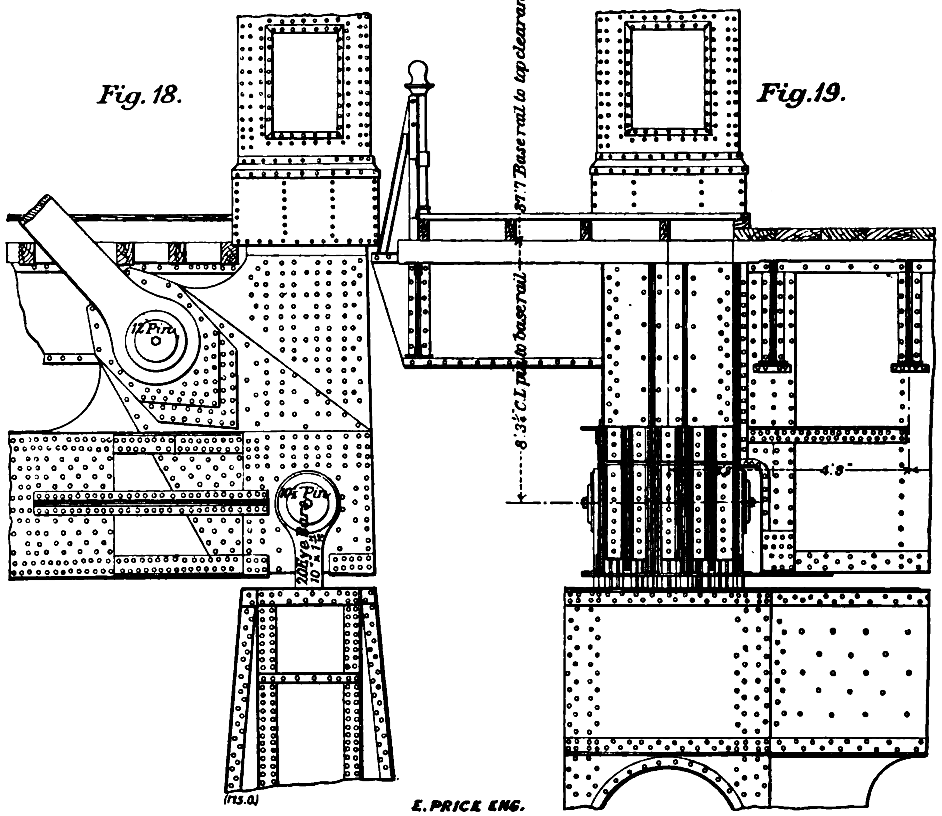
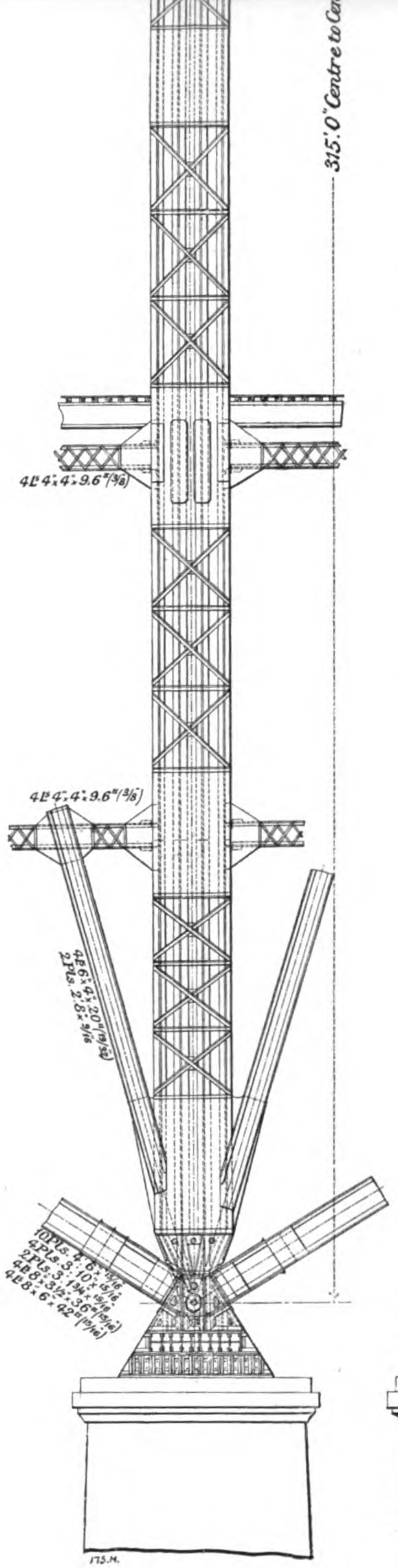
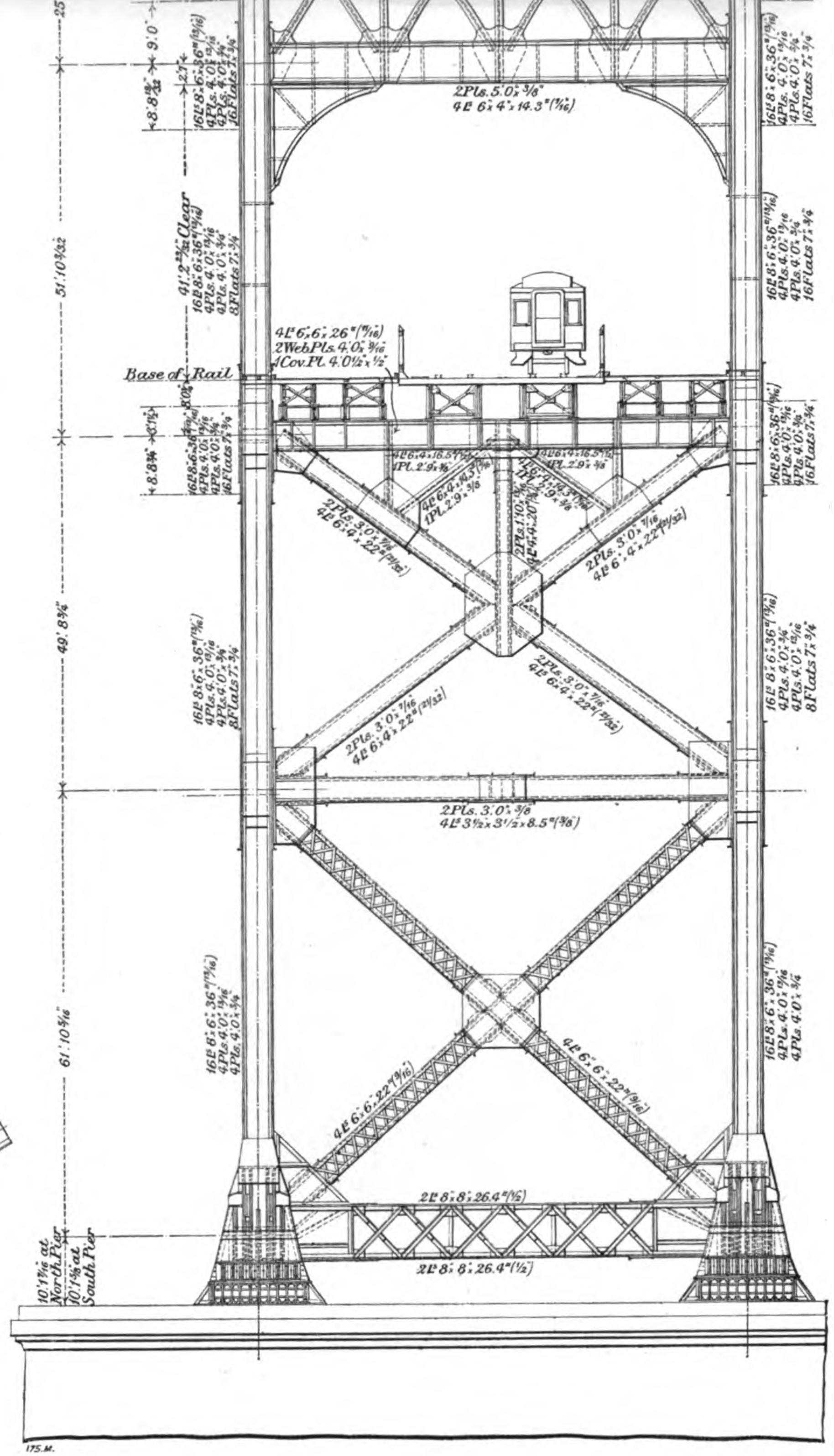


Fig. 18.

Fig. 19.





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THE WRECK OF THE CANTILEVER BRIDGE AT QUEBEC.

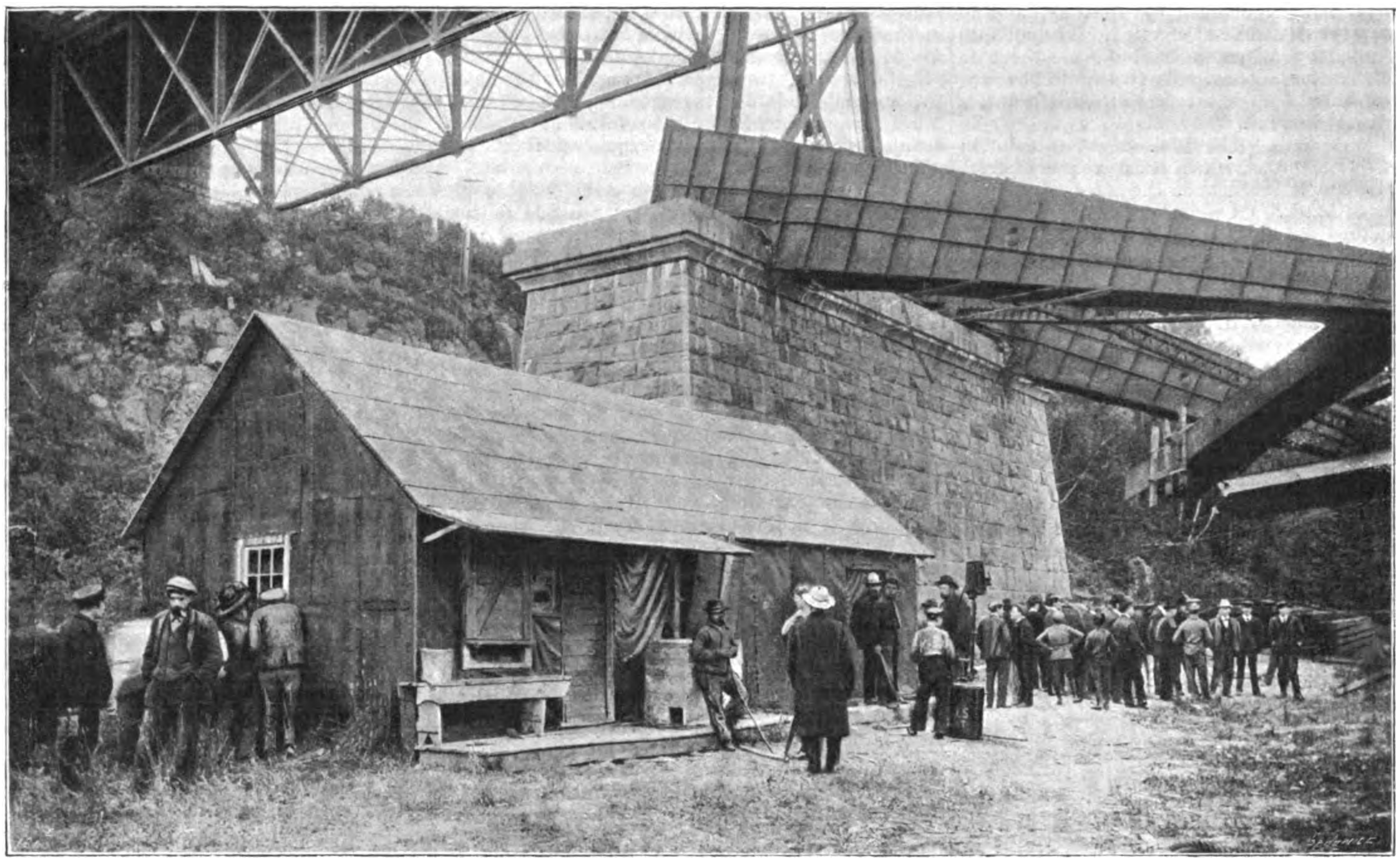

FIG. 3. SOUTH ANCHORAGE PIER.

that of 3060 persons employed in the Vancouver Island collieries 774 were Chinese, 86 Japanese, 55 Hindus, and the remainder whites. Further coal properties are being prospected in the Nanaimo district, and will probably shortly be worked.

In the second of the two districts to which we have referred above as being those which alone produced coal during 1906—viz., the Crow's Nest Pass region—one company operates three collieries, each consisting of several mines. The company last year produced a total of 720,449 tons of coal and 189,385 tons of coke. The collieries are situated respectively at Coal Creek, about five miles from Fernie, on the Canadian Pacific Railway; at Michel, on the Canadian Pacific Railway, 23 miles south of Fernie, and at Morrissey Creek, about 14 miles south-east of Fernie, and connected with the Canadian Pacific Railway and Great Northern Railway. This latter colliery was shut down for nine months out of the year on account of labour difficulties. The seams mined are of a thickness from 4 ft. to 8 ft. in No. 9 mine, and 4 ft. to 12 ft. in No. 5. In some instances long-wall working is employed, and in others pillar and stall. At Michel Creek mines are worked on both sides of the creek, in some cases being driven in 1½ miles, the work here being carried on on the pillar-and-stall system. Returns show that the labour employed at these collieries is all white, the proportion being about two underground to one above. Another company expects to be shipping from this district during the current year.

Several other coal-fields are now being opened up, as, for instance, that at Nicola, to which place a line of railway has recently been completed. Here a seam of clean coal 6 ft. to 8 ft. thick, and of fair coking quality, has been struck. Two companies are at work in this neighbourhood. Little work has been done in other districts in which coal is known to exist, but important investigations were carried out during the past year, proving the existence of important fields which only await transport facilities to enable them to become economical coal producers. One such field exists, for instance, in the Omineca district, in latitude 56 deg., where, in a region through which the Peace River runs, to the east of the Rockies, it is probable that coal will in the future be worked in large quantities. There are here numerous indications of the presence of a very fair quality of bituminous coal. At the present time the land is under Government reserve.

In a rather more accessible district, through the heart of which runs the proposed route of the Grand Trunk Pacific Railway, thus facilitating early development, coal-mining has been already started, several seams having been stripped. This district is south of Hazelton, on the Telkwa River. Here one company has exposed a top seam of 19 ft. 7 in. in thickness, 12 ft. of which, however, contains a few small clay partings. A middle seam showing 17 ft. 10 in. of coal, broken with seams of shale 2 ft. and 2 ft. 7 in. in thickness, and a lower seam showing about 11 ft. of coal, have also been stripped. This coal will form good fuel, but is scarcely suitable for coking. Some of the coal of this district appears to be of a class of semi-anthracite. In the same region coal of a still more anthracitic nature has been found in four seams, measuring 4 ft. 2 in., 4 ft. 6 in., 4 ft., and 7 ft. 3 in. in thickness respectively.

Although large quantities of coal are thus proved to exist in this Telkwa region, it is stated that the geological formation of the district is such that prospecting will have to be carried out very carefully before extensive operations are embarked upon, the district being traversed by dykes and full of faults. It is suggested that a large amount of coal can be won by simply stripping, as is done in Pennsylvania in some cases.

Another field which promises to be of some importance is that known to exist in Graham Island, the most northerly of the Queen Charlotte Islands. On this coal-bed work of a spasmodic nature has already been performed, proving the existence in places of seams varying in thickness from 4 ft. to 16 ft. with thin shale partings. The field extends over a tract of country perhaps 50 miles in length and 15 miles in greatest width. Owing to inaccessibility, the most valuable seams being, it appears, away from the coast, it is probable that considerable difficulty will be experienced in their development. As also nearly all coast surveying still remains to be done for these waters, sea transport will be difficult for some time to come. The qualities of coal already found here vary in quality from a coal giving as much as 24.54 per cent. ash, to coal of anthracite nature. In parts of the anthracite beds, however, the coal is so crushed as to be of little value.

In addition to these fields in British Columbia, it must not be forgotten that in the neighbouring province of Alberta there exists, according to estimate, no less than 140,000 million tons of coal. If the development of coal-mining can keep pace with the other industries, its future should be a very bright one in the north-west of Canada. With trans-Continental lines passing through some of the chief coal districts, there should be no difficulty in finding markets for the whole of the output of the mines. Difficulties are, of course, sometimes experienced in winter, both in working and shipping, but in most of the fields the working season is sufficiently long to be decidedly profitable.

THE FALL OF THE QUEBEC BRIDGE.

ON page 366, Figs. 1 and 2, and in Fig. 3, above, we illustrate the wreck of the Quebec Bridge. In a letter just received, Mr. Skinner, the author of the article on page 351, states that his examination of the débris shows the steel to have been of excellent quality, and the workmanship first class. About a quarter of the central span was complete at the time of the accident. In last week's article we assumed that one-third of it had been erected. The stresses on the metal at the time of the accident were therefore a little less than those then deduced. Mr. Skinner says they were about three-quarters of the estimated live and dead-load working stresses. On the other hand, it is possible that the compression member was of 26-ton steel instead of 28-ton material, as we then believed. Both qualities were used for the bridge, and in view of the American predilection for punching plates instead of drilling them, it is not improbable that this very soft metal was used for the lower chords. Of its kind, no doubt, the metal was excellent, but most practical men will agree with the late Sir Benjamin Baker in preferring to use a much stiffer material for compression members.

Mr. Skinner says that the destruction of the steel work is complete, but that the masonry is practically uninjured. The conditions at the time of the catastrophe appear to have been rather favourable

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than otherwise. The material, as stated, was under less than its designed maximum stress, and the wind was not strong. Almost all the field joints were fully riveted, and the remainder were fully bolted. Work was being concluded for the day, and no members were in course of erection. The collapse was preceded by a loud noise, but was otherwise practically instantaneous. This was to be expected, as when a single member of a single intersection truss does fail, the collapse of the whole must be immediate. The total weight of metal in place was 17,800 tons, which fell distances of from 150 ft. to 400 ft. Much of the wreck is buried in 200 ft. of water. Of eighty-six men at work on the structure seventy-five were killed. The erection, we are informed, was being carefully conducted, and since the metal and workmanship were good, it is impossible to attribute the disaster to anything but defects in the design and proportioning of the structure. The bracing of the lower chords certainly appears light.

An examination of our illustrations will show that the anchor-arm moved riverwards, and thus the approach-span escaped being overwhelmed in the same disaster. It is quite possible that the catastrophe originated some days before its consummation; and, according to a *Times* telegram, the chairman of the bridge company has officially stated that signs of weakness were detected in the lower chord two days before the collapse. In that case the sequence of events would presumably be as follows:- The defect would accentuate under the steadily-increasing weight of the bridge as the erection proceeded, till finally the chord members failed by crippling. The overhanging portion of the steel-work would then turn about the main pins at the top of the column over the river-pier, so that this column would be struck some 20 ft. or 30 ft. from its foot by the rotating mass. Owing to the elasticity of this column and the great inertia of the shore-arm, but little of the force of the blow would be transmitted along the lower chord of the cantilever to the anchor-span, but the foot of the post would be knocked clear of the pier towards the shore, following which the remainder of the shore-arm would turn about the anchor-pier, falling finally to the ground below.

RAILWAY ENGINEERING INSTRUCTION.

THE total capitalisation of the railways of the United Kingdom may be roughly put at 1,200,000,000*l*., and their annual expenditure at about 70,000,000*l*. These figures do not pretend to be anything more than rude estimates, but they are sufficiently accurate to give point to a matter to which we wish to draw attention. With this enormous annual expenditure, and with a considerable proportion of the population either directly or indirectly connected with, or engaged on, work to the order of the railway companies, in practically no school or college can a young man acquire knowledge of the principles underlying railway work. To this stricture there are certain exceptions, as, for instance, the London School of Economics, but even this attempt, although a most laudable one, only touches the fringe of the matter.

The cause is traceable probably to the desire of colleges to prevent students from specialising too early in life, before they have either had adequate opportunity of seeing whether their natural inclinations or interests lie in any definite directions. The President of the Institution of Civil Engineers recently took occasion to discourage students of the Institution from too early specialisation; yet, on the other hand, in his presidential address in November last, he referred to his predecessors in that chair as more judicious than he, in that while he had spent forty years in wandering in many different wildernesses, they had confined themselves to particular regions.

The average college engineering course leaves one at the end of the three or four years with a very unpractical knowledge of anything but pure design. Even design is, as a rule, only given as illustrative of theory, and considerations that arise continually in practice are too often ignored. As practically all engineering work results from some economic cause, or is performed in order to attain some economic end, it is somewhat surprising that so little heed is paid to this side of an engineering education. Even the making of a bolt can be done profitably or unprofitably, but all that is taught relates to the dimensions to be adhered to. This may seem a somewhat futile example. If so, let the subject of railway engineering be considered. Lectures are often nothing more than a mere review of current practice. No instruction is given on the reasons which have led to the adoption of the methods at present in vogue. No facts are unfolded relating to the principles underlying systems of organisation, trading, or finance, all of which, the student finds on leaving college, often have a much stronger bearing on the solution of a problem than questions of pure engineering.

It must not be supposed that we consider it possible to teach such things completely in the lecture-room. But it should be possible to instil into the minds of the student the general principles of economics bearing on his work, instead of divorcing the two subjects so completely that at the end of his course he has absolutely no idea of anything beyond drawing-board design, being left to acquire the rest as best he can. Engineering and economics cannot be so separated in practice, and in such a matter as railway work the advantage of having knowledge of both these subjects would considerably assist in the intelligent control of the enormous expenditures we have quoted above. Although, perhaps, not handling direct expenditure, the work of subordinates is largely responsible for economic working; and to ensure this it is necessary for them to take a broader view than that suggested by the average college training.

In vol. lxxxii., page 672, of ENGINEERING, we had occasion to mention a few facts concerning some points of technical education in the United States. We there mentioned, among other names, that of the University of Illinois. This University, centered at Urbana, in the State of Illinois, U.S.A., has a recognised school of railway engineering and administration. It is not expected that from this college men will straightway procure prominent positions in the railway world, and the work of the college is not intended to do more than supplement the experience that is gained in actual practice. Its object is, in fact, to give men an intelligent understanding of the causes influencing railway work, and to suggest to them the lines along which economical solutions to problems may be sought. It is not solely a teaching of present practice (though this, of course, plays a prominent part in the course), but it is also instruction on the causes influencing the development of railway work in the past, and on the application of conclusions thus arrived at to the solution of problems of the present and the future.

The railway work at this college is divided into four courses, which deal with civil engineering, electrical engineering, mechanical engineering, and railway administration. The courses last for four years, and in the first year the education is quite broad in character, including even rhetoric. In the second year the general principles of economics and railway management are introduced among the subjects, and subsequently the students are drafted off to the special branches of railway work they desire to take up. In the civil engineering course, after the general education, in more specialised directions may be found shop practice and subjects such as railway curves, yards and terminals, structures, graphic statistics, materials, applied mechanics, railway management, theory of railway location, railway systems, signal engineering, and railway operation, &c. Similarly, in the electrical engineering course, a first year's work on very broad lines develops into a study of those subjects of value to a man on roads on which electric traction is used, including both mechanical and electrical work, and a consideration of economic influences. In mechanical engineering the same may be said, while in the administration course the economic problems are entered into more fully, and attention is paid to corporation management, industrial consolidations, commercial law, shop systems, passenger and freight service, railway accounting, mercantile organisations, theory of railway location, political science, &c.

Nor is it all class-room work. The university is situated in close proximity to one electric and three steam railroads, and on these actual field-work is done, and demonstrations are given which must be infinitely more instructive than an objectless survey of a common or field. In connection with its electrical railway course, the University maintains an electrical dynamometer car. This car, by the courtesy of the Illinois Traction System, is run over their interurban system between several cities, thus giving opportunities for investigating problems under actual working conditions, which may easily be made a much more impressive and beneficial form of instruction than class lectures on such subjects at their best. By these means students become familiar with actual practice in such matters of speed, acceleration, power, &c., in a manner impossible to the mere "taker-of-notes," while the practice afforded of reading and using instruments for the purpose of making records and deducing results, ensures their thorough preparation for similar work if needed, when the students are subsequently in the employ of an electric railway company.

Similarly the mechanical course is finished off with work in a dynamometer car, which is jointly owned by the University and the Illinois Central Railroad. This car has been worked all over the Illinois Central, the New Jersey Central, the Baltimore and Ohio, and the Cleveland, Cincinnati, Chicago and St. Louis, and the New York Central Railroads. On these lines it has been used for establishing the tonnage-ratings for freight service, so that the students have been doing most practical and useful work, while it was also used in connection with the electrification of the New York Central Railroad. Under these conditions instruction is something on a rather higher scale than that obtainable at technical colleges in this country, and it is to be regretted that such facilities are not obtainable by students over here. The facts we have mentioned do not complete the list of the advantages enjoyed by a student at the University of Illinois. For instance, actual locomotives are fitted for road tests under service conditions on the steam-roads entering the town, in addition to the dynamometer car tests; while in equipment the workshops, &c., of the college are all fairly complete. Nor is the least advantage the maintenance by the college of a library and reading-room of current literature on engineering and economic questions.

This college does not stand alone. Purdue University has for long been doing useful investigation work on locomotive problems, while if another example be needed, it is only necessary to turn to the International Correspondence Schools to find it. These schools own two dynamometer cars, one for use in passenger and one for freight service on railways. These cars are hired out to companies wishing to carry out tests on their roads, so that, although students cannot carry out the actual tests by correspondence, those actually able to take up the work benefit to a great degree, while the others have the actual results of practical investigations placed before them to assist them in their studies. By agreement with the railway companies the cars are transferred from centre to centre wherever there happens to be a sufficient number of the railway employés taking a course of instruction from this school. As the railways do a good deal to foster such work among their men, the centres are fairly numerous, and the scholars in such centres have the benefit of actual work in a dynamometer car. The practical turn seems to be more clearly held in view in such work in the United States than it does over here, and the policy of leaving men to acquire knowledge at the cost of mistakes does not altogether seem the wisest. Familiarity with the economical side of one's subject is often of far greater value than a knowledge of the possibilities of theory, and it is the failure to keep this in view in technical schools that has led largely to the disparagement they often have to face at the hands of successful men.

NOTES.

AFFAIRS IN HONG KONG.

THE progress of events in Hong Kong must always be of interest to those engaged in trade with the Far East. Hong Kong is a British Crown Colony, and is a very important centre of distribution of Western products, as well as a naval station. The last report of the Governor, which has just been published, shows that the colony is developing, although there is nothing very striking to record. The hygienic conditions of Hong Kong have been much improved since the days when to be sent to Hong Kong was equivalent almost to being sent out of the world. In view of the plague, the authorities have been taking special precautions to render existing domestic buildings rat-proof, as the rats are the chief carriers of the plague; and otherwise steps have been taken to improve the sanitary condition of the colony. Still the rate of mortality in 1906 was higher than

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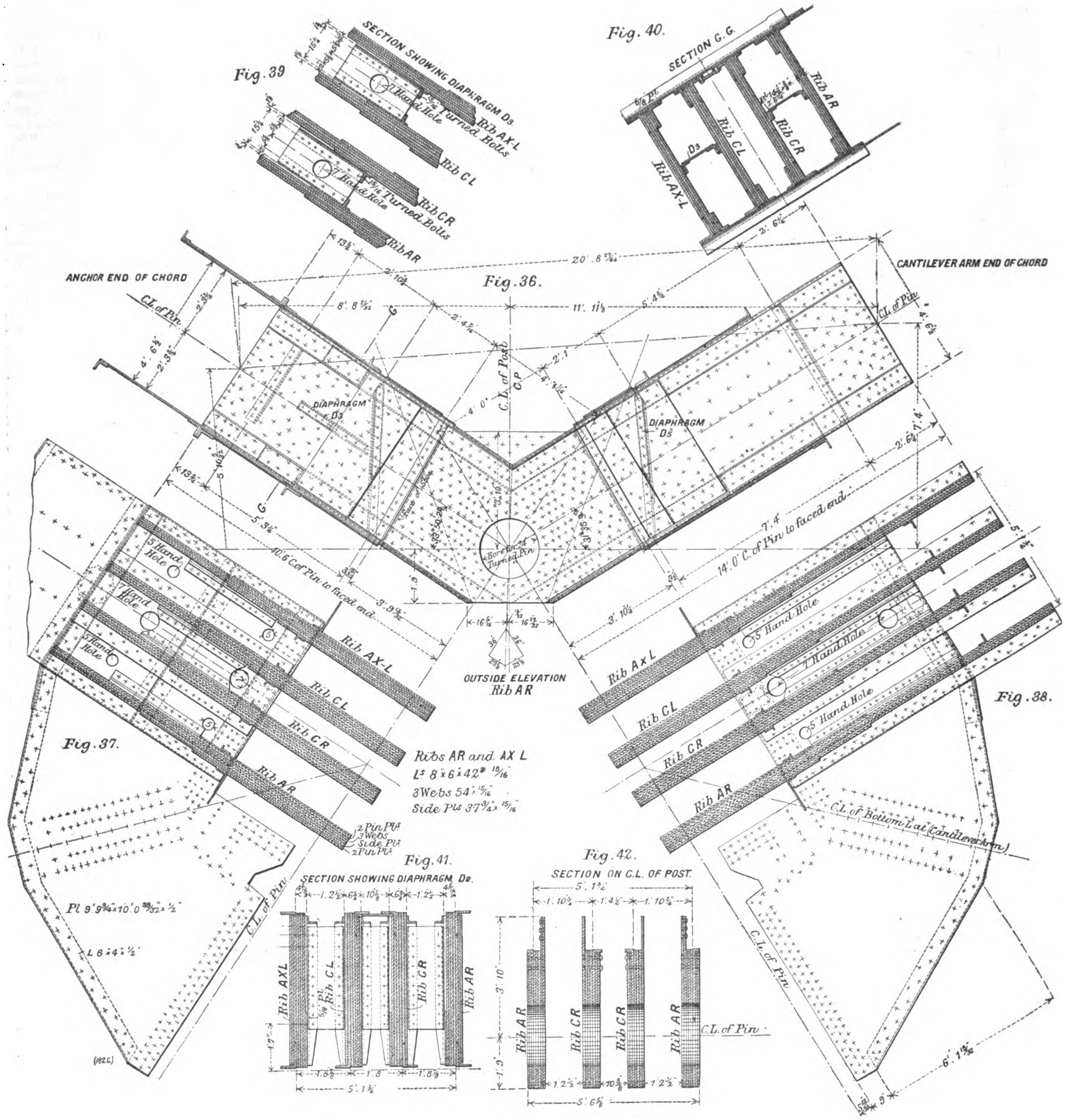
THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(*For Description, see Page* 388.)



FIGS. 36 TO 42. DETAILS OF LOWER CHORD AT JUNCTION OF ANCHOR AND CANTILEVER ARMS.

parallel to and 70 ft. from the present outer wall, the intervening space being filled in with excavated material from the harbour, &c., and topped with a paved and macadamised roadway. As there is a considerable layer of sand overlying the clay, the whole of the foundations will be encased by sheet piling. It is further proposed, probably at a later date, to project a timber jetty from the seaward end of the West Pier Extension, which will afford better facilities for quickly discharging large vessels.

The proposed new works at Whitby Harbour are shown in Fig. 50, and are estimated to cost about 75,000*l*. They include the extension of the present piers by some 500 ft., the provision of 7 ft. of water at low-water of spring tides from the entrance to the bridge, and the construction of a fish quay (marked A on the plan).

RAILWAY TRUCKS IN RUSSIA.—The number of goods trucks upon the Russian lines was estimated last year at 385,000. This total compares as follows with the corresponding number of trucks upon the Russian network year by year since 1897:—

Year.	Trucks.	Year.	Trucks.
1897	212,337	1902	309,506
1898	232,353	1903	325,976
1899	249,600	1904	335,008
1900	276,389	1905	356,000
1901	297,320	1906	385,000

The proportion of goods trucks per mile of railway in Russia was higher last year than the corresponding proportion in the Austro-Hungarian Empire and the United States of America. It was lower, however, than in Germany and France.

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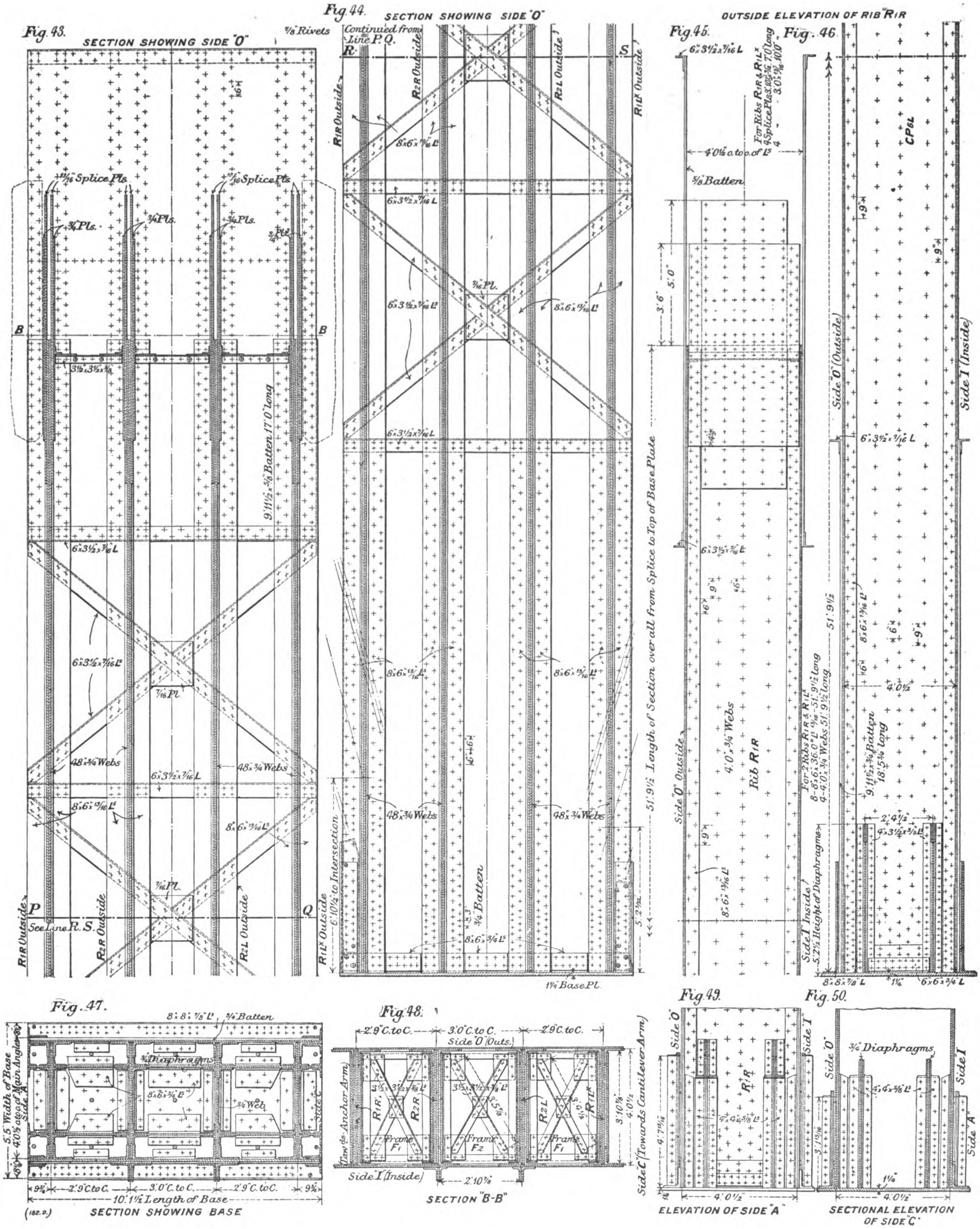
(*For Description, see Page* 388.)



FIGS. 43 TO 50. DETAILS OF VERTICAL POST, ON MAIN PIER.

THE QUEBEC BRIDGE.*

By FRANK W. SKINNER, M. Am. Soc. C.E.

(*Continued from page* 355.)

DETAILS OF MEMBERS AND CONNECTIONS.

Bottom Chords.—The bottom chord panel-lengths vary with the inclination and with the spacing in anchor and cantilever arms from 50 ft to 68 ft, and the chords have a maximum total stress from live, dead, and wind loads combined of 19,150,000 lb., for which a sectional area of 842 square inches is provided.† Throughout both anchor and cantilever arms all sections have a rectangular cross-section about 4½ ft. deep and 5½ ft. wide over all, made up with three or four built channels, with their horizontal flanges connected at the ends by wide ½-in. plates and divided into 5-ft. intermediate panels by 4-in. by 3-in. transverse angles. Each panel is X-braced with 4 in. by 3-in. lattice angles, with two rivets in each end. The lower batten plates project beyond the inner sides of the chords, to form, with connection plates field-riveted to the inner top flanges, jaws receiving the lateral struts and diagonals. The construction of these chord members is well shown in Figs. 22 to 34, Plate XLIX., and in Figs. 36 to 42 on page 386. Views from photographs are also given in Figs. 71 and 72, page 394.

The vertical posts are connected to the chords with 12-in. pins. The ribs are made with bevelled joints at panel points, spliced with shop-riveted web cover-plates. The field-splices, 10½ ft. beyond the panel points (see Fig. 31), have open holes drilled to iron templates and match-marked in the shops. In the inner ribs the top flange-angles are turned inwards, all other rib flange-angles being turned outwards. The maximum lower chord section weighs 105 tons, is the heaviest single piece shipped from the shops, and, except the main post, the heaviest single member. Each field-chord splice has from 500 to 750 1-in. rivets, field-driven by special pneumatic hammers.

The typical bottom chord section in the eighth panel from the anchor end (Figs. 30 to 34 on Plate XLIX.) has a maximum stress, exclusive of wind-load, and section of 15,079,000 lb. and 767 square inches respectively. It is 54½ ft. long and weighs 164,000 lb. Each rib is made with three 54 in. by $\frac{13}{16}$-in. and one 37¾-in. by $\frac{15}{16}$-in. web-plates, with nine horizontal lines of rivets and one 8-in. by 6-in. by $\frac{15}{16}$-in. angle in each flange. The ends of the webs are connected by vertical transverse diaphragms.

The bottom chords in the river-end panels of the anchor arms are special in that they have no pin-holes, but are spliced with web cover-plates to a short bent section of bottom chord, which is continuous across the main pier and engages the centre bent pedestal (see Figs. 36 to 42, page 386).

The shore-end panel-chord section (see Figs. 22 to 29, Plate XLIX.) has the minimum area of 301 square inches for its maximum stress, exclusive of wind, of 4,950,000 lb., and its ribs are made with single $\frac{15}{16}$-in. web-plates. At the shore end they are slotted to receive a horizontal diaphragm-plate 62 in. long, which affords connection for two intermediate short webs, which, like the projecting ends of the main webs, engage the lower ends of the inclined end-bars, and are stiffened with transverse diaphragms to make a massive pillar, field-riveted to the end of the double-web end floor-beam, and transmitting the heavy lateral stresses to the pier masonry.

The anchor and cantilever-arm bottom chords are connected together across the main pier by a special 40-ton V-shaped section (Fig. 36, page 386) about 23 ft. long and 9 ft. high, which passes continuously through the shoe and pedestal, and has web and flange cover-plate splices, field-riveted at both ends to the regular chord sections. It also engages the 24-in. hollow pin through the foot of the main vertical post which connects all the intersecting members. Its four ribs correspond to those of the regular chord sections, and are heavily reinforced by web-plates continuous across the centre line, uniting the two separately-built wings, and together affording an aggregate length of 15 in. pin bearing.

On both sides of the pin there are top and bottom flange-plates, field-riveted to the inclined plates in the shoe and pedestal, and thus making a very solid connection to the bearing members and transmitting to them the lateral stresses received from the transverse and diagonal struts, field-riveted between very wide, thick, gusset-plates, projecting from the top and bottom flanges of the chord section, and stiffened by single 6-in. by 4-in. flange angles on the outer edges. The chord webs are stiffened and connected by three heavy diaphragms on each side of the main pin, in directions parallel, perpendicular, and oblique to the axis of the chord.

Vertical Posts.—The intermediate vertical posts in the anchor and cantilever arm trusses are from 96 ft. to 225 ft. long, and have a maximum total stress of 2,244,000 lb., with a sectional area of 110 square inches. All of them have open rectangular cross-sections, made with two built channels 48 in. deep, parallel to the planes of the trusses, and two 6-in. by 8-in. flange angles, latticed with small angles. They were built and shipped in sections up to 105 ft. long, field spliced, with riveted web and flange cover-plates.

At the ends and centre points there are wide, thick flange-plates, projecting far beyond both sides, to receive the top chord and diagonal eye-bar pins, and to serve as short links connecting these members in adjacent panels, thus facilitating erection and reducing the lengths and bending moments on the pins. The largest sheared plates used for these connections are 134 in. wide, a dimension corresponding closely with the maximum vertical clearance allowed for members loaded on standard flat cars, and therefore adopted as the limiting size.

Post P1 (see Fig. 3 of Plate XLVII. of our last issue), nearest to the shore end of the anchor-arm, is typical of the intermediate posts, and is about 114 ft. long over all, with a maximum stress of 5,365,000 lb., and a uniform cross-section of 370 square inches throughout. Each of its built channels is made with five web-plates and two 8-in. by 6-in. by $\frac{13}{16}$-in. angles. The upper part is unlike those of the longer posts in that it has only a single pin receiving all the top-chord bars in both adjacent panels. It is provided with five extra ribs, or diaphragms, making in all seven pin-bearings, which are eccentrically extended to receive the 12-in. pin-connection at the top of the inclined post. The pin-bearings are reinforced to maximum thicknesses of 2⅞ in., and, like those in similar members of the truss, were separately riveted to I-shaped cross-sections, afterwards assembled between the diaphragms. The post is made with an upper section 45 ft. 8 in. long, and a lower section 68 ft. 7½ in. long, both together weighing 270,000 lb.

Post P4 has a total stress of 3,620,000 lb., and a cross-sectional area of 174.3 square inches. It is made in two sections, spliced with cover-plates, having about two hundred ⅞-in. field-driven rivets. Each web is made with two 48-in. by $\frac{11}{16}$-in. plates, and has two 8-in. by 6 in. by $13\frac{1}{16}$-in. flange angles. The upper end of the post has two 14-in. pins for the fifty-eight top-chord eye-bars in the two adjacent panels, and a third pin, 12 in. in diameter, for the upper end of the diagonal post. Each of the chord-pins has ten bearings, and the 12-in. pin has four bearings in the main webs and in parallel diaphragms, all of which are much wider than the body of the post, and were separately assembled and riveted as a sort of cap, shop-riveted to the body of the post. The complete post weighs 212,000 lb.

The sub-vertical posts at intermediate panel points are made in two or more sections each, pin-connected to the top and bottom chords, and have rectangular cross-sections made with pairs of built channels, with their flanges turned in. Typical post T oo, at about the middle of the anchor-arm, has a maximum stress of 1,307,000 lb., and has built channels, with 32-in. by 1⅞-in. webs, and 6-in. by 6 in. by $\frac{13}{16}$-in. flanges. The lower end has long flange cover-plates, and a transverse diaphragm for the 116 ⅞-in. field-driven rivets in the floor-beam connection. At its intersection with the main diagonal there are four oblique transverse plates, forming short riveted connection-links with two pins for the adjacent sections of the diagonal eye-bars. These plates project beyond the pins to form jaws for the field-riveted connections to the horizontal and sub-diagonal struts.

Diagonals.—The main diagonal on the shore side of the river-pier is a pin-connected strut 273 ft. long on centres, and weighs about 268,000 lb. The maximum stresses occur in the upper of the four sections into which it is shop-riveted, and are 1,123,000 lb. static load and 2,350,000 lb. wind load in compression, and 1,046,000 lb. static and 2,350,000 lb. wind in tension. The corresponding gross and net cross-sections are 227 and 209 square inches. The general construction is similar to that of the vertical posts, except that there are no wide transverse diaphragms for chord pin-connections. The 38½-in. by 48½-in. rectangular cross-section is reinforced at the lower end with two intermediate diaphragms, to increase the pin-bearings. Where this member intersects the horizontal strut its flange cover-plates are cut to clear the web-plates, and extensions of the latter project beyond the flange angles to form jaws receiving the field-riveted connections of the horizontal struts. The sub-vertical post is continuous at its intersection with the diagonal, and passes between the webs of the latter, and is riveted to wide flange connection-plates stiffened by transverse diaphragms, field-riveted in place after the main members are assembled together.

The sub-diagonal strut in the river-end panel of the anchor-arm is 195 ft. long, and weighs 63,000 lb. It has a cross-sectional area of 59.5 square inches for a maximum stress of 1,201,000 lb., and has a cross-section corresponding to those of the other web members of the truss, made with a pair of latticed built channels with single 32-in. by $\frac{9}{16}$-in. web-plates, and pairs of 6-in. by 4-in. by $\frac{11}{16}$-in. flange-angles.

Main Vertical Posts.—The combined static and wind stresses from suspended span, anchor and cantilever arms, all pass through the main vertical posts on the river-piers, which have a total maximum stress of 10,000,000 lb., plus 1,796,000 lb. developed by bending moment in the centre panel, where the transverse diagonals are omitted to provide clearance for the railroad tracks. The corresponding cross-sectional area is 526 square inches, made up with four latticed webs, forming a 4-ft. by 5 ft. rectangular cross-section. (See Figs. 43 to 50, page 387, and Figs. 51 to 56, page 390.) The post is 315 ft. long on centres, and weighs about 712,000 lb. It is by far the largest member in the bridge, and was shop-riveted in five lengths from about 50 ft. to 76 ft. long, comprising also a 245,000-lb. cap and a 57,000-lb. base-piece or shoe, all field-riveted together. The general cross-section of the post is made with four built I-beams, with their flanges connected by transverse batten-plates and divided into panels, with intersecting lattice angles like the bottom chords. The maximum section has sixteen 8-in. by 6-in. by $\frac{13}{16}$-in. flange-angles, four 48-in. by $\frac{13}{16}$-in. flange-angles, and sixteen 7-in. by ¾ in. flange reinforcement-plates.

Post-Cap.—The vertical components of the 16,000,000-lb. top chord-bar and the 3,500,000-lb. main diagonal strut stresses meeting at the top of the main post amount to about 10,000,000 lb., which is transmitted to the main body of the post through the extended upper part or cap, 10 ft. long, 8 ft. wide, and 20 ft. high, which weighs about 245,000 lb., and is made with twelve vertical longitudinal diaphragms, or pin-bearing ribs, shop-riveted together, and shipped in three sections, one composed of the two centre ribs, and the two outside duplicate sections each composed of five ribs. The ribs were assembled and field-riveted, or bolted, at the site through an elaborate system of field and shop-riveted diaphragms, which transmit the stresses between the connected members and uniformly distribute the vertical components to the body of the main post.

The cap serves as a sort of multiple rigid link with four unsymmetrically-located 12-in. pins, each located on the centre lines of the converging members which intersect at a common point. Each pin receives a single member, thus greatly facilitating and expediting the erection, besides reducing the length and moment of the pins, which would otherwise have required enormous dimensions if an attempt had been made to group all the members, including sixty eye-bars, 2¼ in. thick, on a single pin. It also provided for all the boring and elaborate assembling and difficult riveting of the complicated parts to be concentrated in a single piece of dimensions much more convenient to be turned in the shop and received in the machine-tools than would have been a full length post-section, perhaps 100 ft. long, as are some of the other members.

The longitudinal webs were reinforced by ⅞-in.

* [This article was, of course, written prior to the occurrence of the grave disaster with which we had occasion to deal in our last two issues. As a complete account of a most important work, however, it has lost none of its interest, and a study of the information which it contains will add much to the understanding of the inquiry now being made into the causes of the failure.—ED. E.]

† [According to the strain-sheet published on Plate XLVII. of our last issue, the greatest load is 25,189,005 lb and the area 776.2 sq. in. Some other discrepancies will also be noticed by our readers.—ED. E.]

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plates to build up the pin-bearings to a combined maximum thickness of about $4\frac{1}{2}$ in. The ribs are connected by I-shaped transverse diaphragms. Only the centre transverse diaphragm extends through to the top of the cap, all others are cut to clear the eye-bars. The upper pins receiving the eye-bars engage all twelve ribs, but the lower ones, for the diagonals, engage only four ribs, the rest being cut to clear the post-jaws. The top of the cap is closed by inclined $\frac{1}{2}$-in. field-riveted cover-plates, virtually roofing it to exclude snow. The shop drawings for the cap include nineteen main plans, sections, and elevations, besides several minor sketches and details, and cover a 32-in. by 144-in. sheet.

Shoe.—The foot of the vertical post has a pin-bearing in the pedestal which connects it to the lower chords of the anchor and cantilever trusses, and transmits its load to the pier masonry. For facility of shop work and erection, the shoe, like the cap, was made as a separate piece, and has a $5\frac{1}{2}$-ft. by 10-ft. horizontal top-plate connected to the corresponding base-plate of the lower section of the post by about 200 field-rivets. It is 9 ft. high, weighs 67,000 lb., and has four longitudinal webs meshing with those of the pedestal, and reinforced to give the 24-in. pin a total length of 27 in. of bearing. The webs are connected by four lines of inclined transverse diaphragms, the inner ones being made with plates and angles, and the outer ones with double pairs of angles without web-plates. The shoe is provided with very heavy horizontal and inclined transverse plates perpendicular to the webs in the plane of the top flange of the centre bottom chord section and field-riveted to it. These plates are shop-riveted to the outside webs and reinforced at the centre by heavy brackets with faced ends. Details of its construction are given in Figs. 57 and 62, Plate L.

Pedestal.—The 24-in. main pin at the foot of the pier-post engages only the post-shoe and the corresponding 166,000-lb. pedestal, which is a 11-ft. by 13-ft. frustum of a rectangular pyramid, about 8 ft. high. Although of a very simple type, corresponding with the ordinary bridge truss pedestal, the stresses to be provided for are so great and the bearings are so massive that a large number of pieces were required, and very great care was necessary to ensure perfect bearings and the proper distribution of stresses and connection of parts, and the drawings were therefore elaborate and full of notes and details, and, at a 1-in. scale, covered a 39-in. by 130-in. sheet, and required seven months' time of an expert bridge-draughtsman to make them.

The planed base-plate is 3 in. thick, and is made with three pieces butt-jointed. To it are riveted five vertical longitudinal webs clearing those of the shoe, and each made of four to six 1-in. and $1\frac{1}{8}$-in. plates riveted together. The outside plates have full pin-holes, and the inside ones half pin-holes. The webs are connected to the base-plate with parts of 15-in. channels, having five horizontal rows of rivets through the web. They are stiffened and connected by five lines of vertical transverse diaphragms, each made with four 6-in. by 8-in. angles, with their outstanding flanges engaging and bolted together, since there is not clearance between the webs for driving rivets.

The heavy pressure is distributed to the outer edges of the base-plate by five vertical transverse diaphragms on the outer face of each web. The inclined edges of the diaphragms are stiffened with pairs of 6-in. by 8-in. flange angles, and the outer edges of the main webs are covered by inclined plates, excluding snow and rubbish. The pedestal is provided with heavy horizontal and inclined transverse plates, like those of the shoe, which engage, and are field-riveted to, the lower flanges of the V-shaped centre section of the bottom chord. The pedestal is connected to the bolster by seventy-six $1\frac{1}{4}$-in. vertical bolts. This pedestal and the bolster below it are illustrated in detail in Figs. 63 to 70, page 391. (See also Fig. 6 on page 358 *ante*).

Both the weight and dimensions of the pedestal were so great that it was necessary for the Pennsylvania Railroad to build a special steel car for its transportation. This car had an open floor, made with very heavy plate-girders, distributing the load on the trucks, and making it possible for the pedestal to project down almost to the rail-level. The pedestal was suspended from a box-girder passed through its 24-in. pin-hole, and supported on two pairs of transverse I-beams, with their ends seated on longitudinal beams. Heavy steel brackets were temporarily bolted to the approximately vertical base-plate, and took bearing on the car-frame to prevent the pedestal from oscillating or becoming displaced in transit.

Bolster.—The pedestal is seated on an enormous bolster, designed to distribute the 28,000,000-lb. load uniformly over a large area of the pier masonry, and to reduce the unit pressure to 78,000 lb. per square foot on the granite cap-stone. It is a multiple-web frustum of a rectangular pyramid 74 in. high, with an 18-ft. by 20-ft. base and an 11-ft. 9-in. by 13-ft. $2\frac{1}{2}$-in. top, and weighs 250,000 lb. It was not made of steel castings, as similar members for smaller spans have usually been made formerly, because of the difficulty and delay likely to be experienced in securing satisfactory castings of such size, and on account of the uncertainty of the quality of the metal even when the exterior is satisfactory. The bolster was therefore built entirely of rolled steel (see Figs. 64 to 70, page 391), and great pains were taken to make it entirely of standard materials and secure perfect bearings and integrity of connections, and to provide for simple and convenient operations of manufacture. It is made with an upper and a lower part, each built in halves connected by long through-bolts, and both symmetrical about the longitudinal and transverse centre lines. The lower part, 40 in. high, has a 360-square foot base area, and weighs 155,000 lb. It has eleven vertical ribs, $2\frac{1}{2}$ in. thick, parallel to the bridge axis, and secured to the 2-in. base and top plates with 6-in. by 8-in. by 1-in. flange-angles. It is divided on the centre line of the middle rib, which was thus made with two $1\frac{1}{4}$-in. plates face to face. The halves are connected by sixteen $1\frac{1}{2}$-in. horizontal upset through-bolts in two courses. The dimensions of the 9-ft. by 20-ft. base-plate were about the maximum of those rolled, and the section of the 26-in. by $2\frac{1}{2}$-in. web-plates was too great for the capacity of ordinary bridge-shop shears, and they had to be cut by the cold saw. The upper and lower edges of the webs were planed to bearing on the planed cap and base-plates, and the base-plate is anchored to the masonry with thirty $1\frac{5}{8}$-in. vertical bolts.

It was considered impracticable to grout under the base-plate, and as the use of a lead bearing-plate between it and the masonry was strongly objected to on account of the wedge-like action believed to be produced by the lead penetrating the pores of the stone under heavy pressure, it was bedded on two thicknesses of No. 1 cotton duck saturated with linseed-oil, and about 1000 lb. of red-lead.

The upper part of the bolster is similar to the lower part, but smaller, being only 34 in. high, with a 14-ft. by 16-ft. base-plate corresponding to the cap-plate of the lower part. It has ten $2\frac{1}{2}$-in. by 30-in. web-plates transverse to the bridge axis; the outer ones stiffened, like those in the lower part, by triangular gusset-plates. The halves are butt-jointed on the centre line, midway between the centre webs, and are connected by ten $1\frac{1}{2}$-in. upset horizontal bolts in a single course, passing through all webs, with nuts bearing on the outside ones. It has 2-in. top and bottom plates, planed on both sides, and weighs 95,000 lb.

As the main members of the anchor and cantilever arm trusses converge to the main pier bearings, and their combined stresses are transmitted through it to the masonry, it is the most important point of the construction, and, contrary to usual practice, the members for the first bearing—viz., bolster, pedestal, shoe, and centre lower chord section, weighing together 569,000 lb., and having a total height of about 18 ft.—were assembled at the shops (see Fig. 6 on page 358 *ante*), and by the ease with which they were fitted without change or trimming, demonstrated the accuracy of the drawings and the precision and excellence of the shop-work.

Pier-Bent.—The 56,000,000-lb. dead, live, and wind-load stresses on each main pier are transmitted from the superstructure through the transverse bents of the pairs of vertical posts, braced together in five panels with six transverse struts and four sets of X-braces. This bent is shown complete in Figs. 20 and 21 on Plate XLVIII. of our last issue. The panels are from about 50 ft. to 60 ft. high on centres, and the bracing is entirely of massive compression members with riveted connections. In the centre panel the diagonals are omitted to give ample clearance above the full-width floor-platform, and the upper corners of the panel are knee-braced to a deep horizontal strut, giving it the effect of a portal construction. All of the braces are double, being symmetrical about their centre lines, to give a thickness of 4 ft., corresponding with that of the vertical posts, and to engage the jaw-plates riveted to its outside webs.

The posts are connected at the top and just above the roadway by horizontal trusses 18 ft. and 25 ft. deep on centres respectively, with their top and bottom chords made of pairs of plate girders 52 in. and 48 in. and 60 in. and 60 in. deep respectively. Their web members have I-shaped cross-sections made with pairs of angles, back to back, latticed. The strut at roadway level is made with two 40-in. plate-girders and a 48-in. cover-plate, braced by diagonal and sub-diagonal and sub-vertical members of the panel below, so as to form a Fink truss, carrying the floor stringers in the same manner as in the other deep transverse sections of the bridge.

The two intermediate struts have double vertical webs, 36 in. and 48 in. deep, and the bottom strut, connecting the pedestals, is a girder 8 ft. deep, with 8-in. by 8-in. by $\frac{1}{2}$-in. flange angles, and 4-in. by 4-in. diagonal members. The X-braces in the panel below the floor-beam are made with built channels, with their flanges turned in and latticed, and those in the regular panels have I-shaped cross-sections made with pairs of 6-in. by 6-in. angles, back to back, latticed and mitred to clear at intersections, where they are connected and spliced by pairs of very large and thick field-riveted flange cover-plates. All latticing here and elsewhere is made with angles, and all conspicuous connection-plates have their diagonal edges concaved, to save weight, and give more pleasing outlines.

Eye-Bars.—The top chords and all the main diagonals, except those in the pier-panels, are composed of eye-bars, 15 in. and 16 in. wide, $1\frac{1}{2}$ in. to $2\frac{1}{16}$ in. thick, and 50 ft. to 76 ft. long over all; dimensions limited by the maximum size of blanks—15 in. by 2 in. by 85 ft.—that the rolling-mills could produce. The heads have an excess of about 40 per cent. over the sectional area of the body, and could at first be made only 32 in. wide, thus determining the 15-in. width of the body and the 12-in. diameter of the pin. Afterwards it was found practicable to upset them to a width of 36 in., thus increasing the bars and pins to 16 in. and 14 in. respectively. Consideration also had to be given to the opposite difficulties of testing thick bars to destruction, and of forging very thin heads. Much time and labour were expended by an expert designer in arranging the eye-bar packing, so as to produce minimum bending stresses in the chord-pins.

The maximum combined top-chord stresses are 15,755,000 lb., requiring thirty eye-bars, with a total cross-section of 711 square inches. The diagonals are made with two or four lengths of eye-bars intersecting one vertical post. When tested to destruction in the direct-acting hydraulic plunger horizontal machine of 2,250,000 lb. capacity, the 16-in. by 2-in. bars, 50 ft. long, were invariably broken in the body in about 15 minutes, developing an average ultimate strength and elastic limit of 57,000 lb. and 32,000 lb. per square inch respectively, and elongating the 12-in. pin-holes to over 14 in.

Pins.—All of the chord-pins, except those in the main posts, are 12 in. in diameter and up to more than 11 ft. long. They have a maximum unit stress of 20,000 lb., are fitted to $\frac{3}{64}$ in., and are all turned from forged bars, and bored through the axis for $1\frac{1}{2}$-in. bolts, securing cast-steel discs covering the ends, and locking the pins in their holes, instead of screwed nuts. The ends of the pins are threaded to receive the tapered pilot-nuts, temporarily engaged to facilitate the connection of members during assembling.

Transverse Bracing.—Horizontal wind stresses are provided for by three systems of lateral bracing in the planes of the top and bottom chords and in the floor, and by sway bracing in the transverse bents between all vertical posts, all members being stiff members with riveted connections, field-connected to jaw-plates engaging their top and bottom flanges, except in the case of deep girders. The top chords are connected at panel-points by horizontal transverse struts, with 4-ft. by 4-ft. rectangular cross-sections made with four 4-in. by 4-in. angles, with their flanges turned in and latticed with $2\frac{1}{2}$-in. by $1\frac{1}{2}$-in. angles. The panels between them are X-braced with I-shaped diagonals 4 ft. deep, made with two pairs of angles, back to back, latticed, one of them continuous and the other cut to clear it, and spliced and connected to

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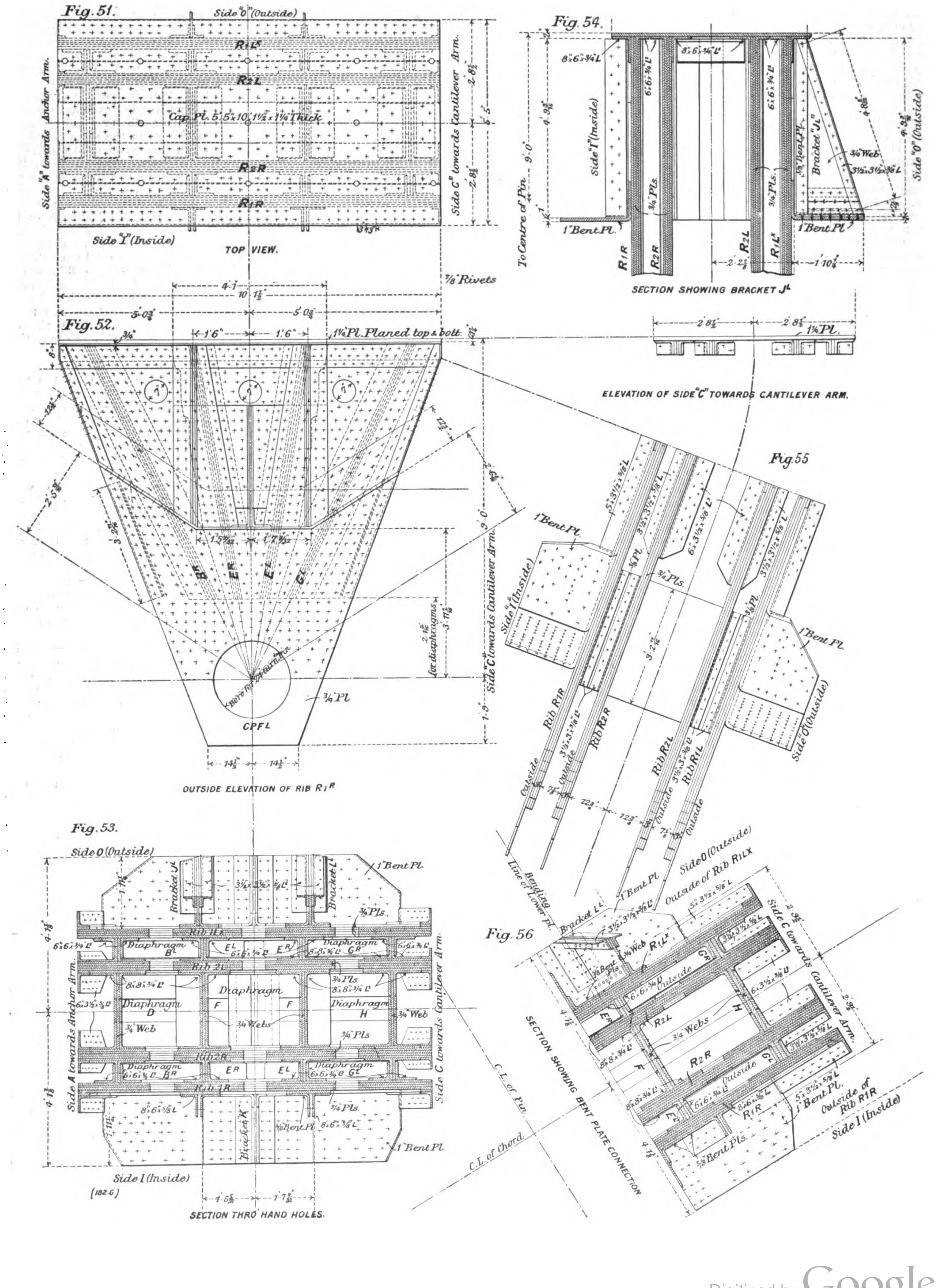
THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(*For Description, see Page* 388.)



THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

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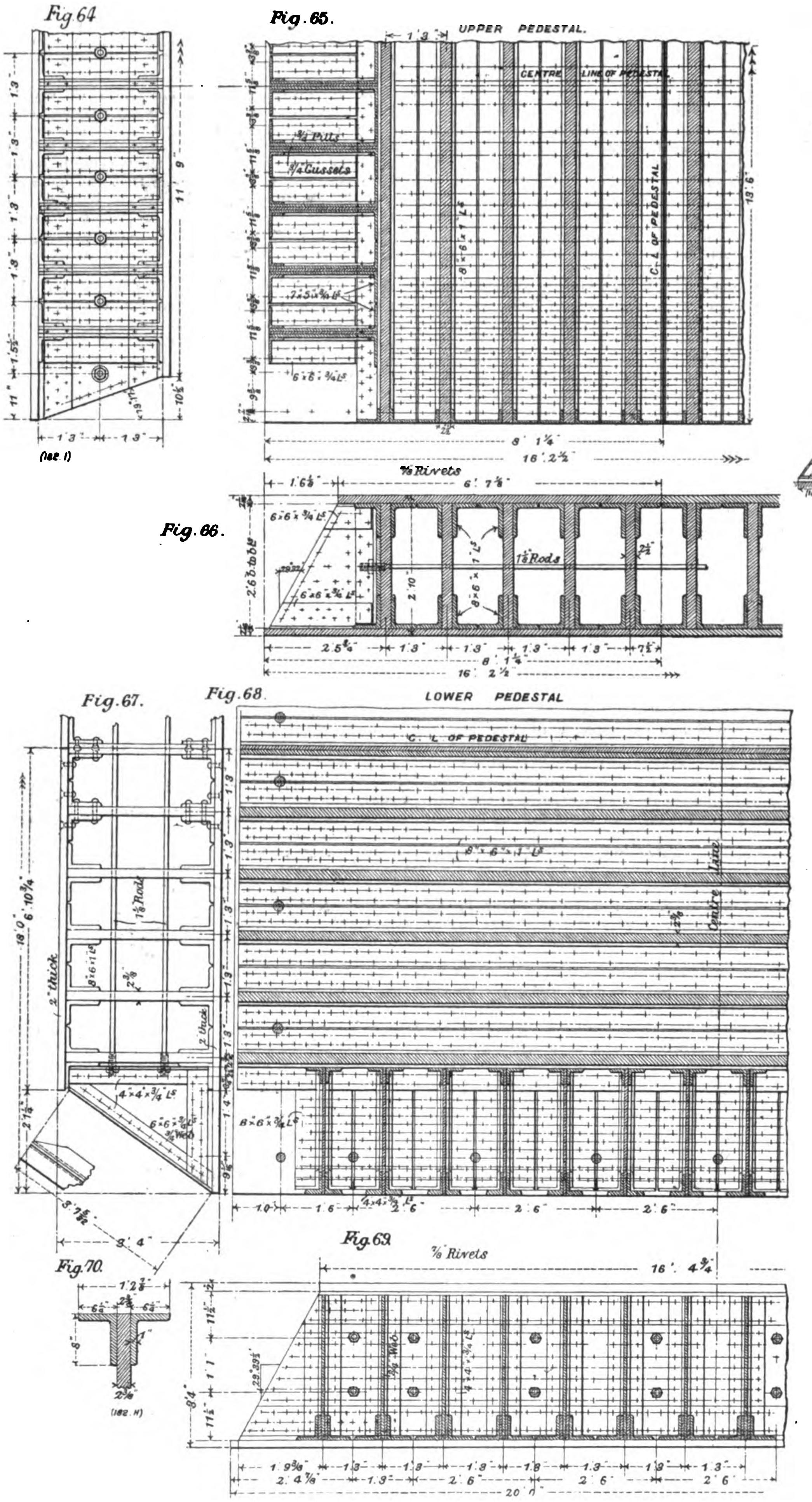

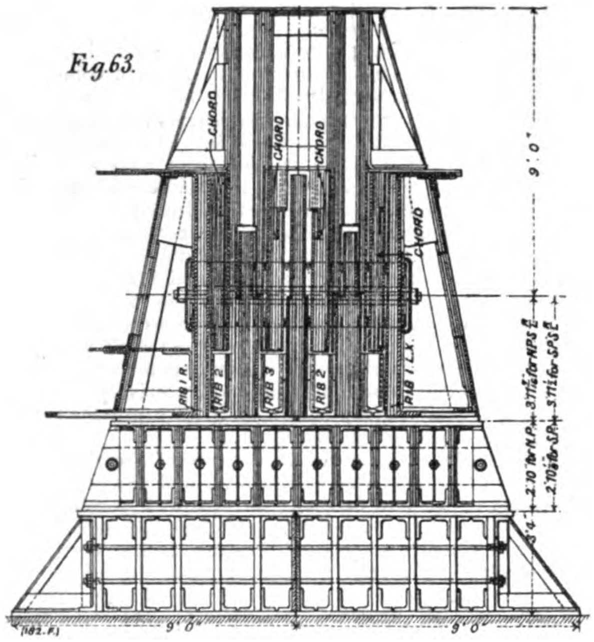

FIGS. 63 TO 70. UPPER AND LOWER PEDESTALS.

it at the intersection by top and bottom field-riveted flange-plates. The maximum top lateral stress is 341,000 lb., with a corresponding cross-sectional area of 17.2 square inches, formed by four 6-in. by 3½-in. angles.

The Bottom Chord Lateral System has transverse struts made with built channels 4½ ft. deep, with vertical webs, and their flanges turned inwards and latticed. The X-braced diagonals have the same depth, and are like the top-chord diagonals, except that their flanges are reinforced by 25-in. by ½-in. cover-plates, and both diagonals of each panel are cut to mitred joints between large field-riveted flange connection-plates at intersections. Their maximum stress is 977,000 lb., for which a cross-sectional area of 66 square inches is provided by four 6-in. by 4-in. angles, four 6-in. by $\frac{7}{16}$-in. flats, and two 25-in. by ½-in. cover-plates. In the floor system transverse stresses are carried through the floor-beams, and diagonal stresses are carried through the zigzag angles riveted to the top and bottom flanges of the stringers.

Sway Bracing between intermediate vertical posts is made with the top and bottom-chord lateral struts, the floor-beams and one or more intermediate transverse struts, according to the depth of the trusses. The panel between the floor-beam and the next higher transverse strut is left open to provide clearance for traffic, and all other panels are X-braced. The intermediate transverse struts have rectangular cross-sections made with four angles, latticed and field-riveted, with the diagonals between jaw-plates on the vertical posts. Above the roadway the X-braces are similar to the transverse struts, mitre-jointed between very large field-riveted flange splice-plates at intersections, and all connection and jaw-plates have their edges cut to smooth concave curves, to present a pleasing and symmetrical appearance. Below the roadway the X-braces have I-shaped cross-sections made with pairs of angles, back to back, latticed, except in panel-bents near the main piers, where the diagonals are special members, forming parts of trusses supporting the floor.

(To be continued.)

THE COLLAPSE OF THE QUEBEC BRIDGE.

By FRANK W. SKINNER, M. Am. Soc. C.E.

THE construction of the Quebec Bridge, to carry two railroad and two electric-car tracks, a highway, and two side-walks across the St. Lawrence River, near Quebec, Canada, forms the subject of articles in the previous and present issues of ENGINEERING. Nevertheless, it will be convenient, for the purpose of the present article, which deals with its collapse, to summarise the chief features here. It is of cantilever construction, with two 600-ft. anchor-arms and one 1800-ft. channel span at a clear height of about 150 ft. above the water. The pin-connected trusses have parabolic top and bottom chords, and are 315 ft. deep over the main piers, where there are two enormous vertical posts, weighing 712,000 lb. each, exclusive of their end connections. The superstructure weighs about 40,000 tons, and has all been completed

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at the shops at Phœnixville, Pa., and nearly all of the south half of the structure had been erected, although no part of the north half had been erected, on account of delay in constructing the service track on which materials are to be delivered.

The work is of characteristic American design, with eye-bar tension members and massive compression members up to 100 ft. long and 100 tons in weight, shipped as completed units from the shops several hundred miles from the site. The general character of the details corresponded closely with those of the largest cantilevers recently completed and now under construction in the United States, and their preparation had been almost constantly in progress for about ten years, under the direction of the most eminent experts and trained bridge-designers, who spared no pains nor expense to secure the best possible structure. Equal pains were taken to secure the highest excellence of shop-work and materials, and all the processes and operations of erection had been minutely planned, special plant constructed, and every contingency fully provided for before the steel left the shops.

About half-a millon dollars' worth of erection-plant has been installed at the site, and the erection had progressed for three seasons with notable safety, success, and rapidity. All of the substructure had long been completed, most of the false-work had been erected on the north shore, all of the south anchor and cantilever arms and three of the twelve panels of the centre suspended span erected, and all their bracing fully assembled and connected, and it was expected that the bridge would be completed during the season of 1909.

Late in the afternoon of August 29 the entire completed superstructure collapsed almost instantly and fell to the ground, a total wreck, killing seventy-five of the eighty-six men at work on it, including all four of the erection foremen and the resident engineer. An experienced foreman on the false-work across the river was watching the men on the erecting traveller as they were making everything secure for the night preparatory to quitting in about fifteen minutes. He heard a loud noise like an explosion, and saw the end of the overhanging structure gradually descend until near the water's edge, swaying a little up and down stream; the traveller fell towards the centre of the river, the main posts collapsed, and the whole 17,000 tons of massive steel-work fell within a few seconds, probably about a half-minute after the first movement. The anchor-arm fell vertically and very nearly in the plane of its axis, on the dry land between the piers and partly over the main pier, where it is entirely accessible. The greater mass of the cantilever and suspended span, except a very small part of the first panel of the former, disappeared below the surface of the water, which, beyond the main pier, rapidly increases to a depth of over 200 ft., with a swift tidal current which precludes recovery or examination of any but a small portion of it.*

At the time of the collapse all known conditions were normal, or especially favourable. The structure was by no means overloaded, as it had been designed to sustain the erection stresses developed by the 2,250,000-lb. steel traveller, nearly 250 ft. high, at the centre of the main suspended span. This traveller had, however, been moved out only to the ninth panel of the cantilever arm, where it had been securely anchored for about nine months, while a 400,000-lb. overhanging traveller had been installed on the top chord and had advanced to the third panel of the centre span about 32 hours previously. It was fully clamped and anchored to the steel-work, and had erected in the fourth panel only the two 34-ton lower chord members, which were still fully supported in its tackles while the shore-end splices were being bolted.

The removal of the main traveller had been commenced and carried on about in proportion to the addition of steel at the end of the overhang, so that the total moment was not greater than would have been caused normally by a single traveller. About 400 tons of steel had been removed from the traveller, including very heavy girders at the top, so that it was less top-heavy than ever before, and even more rigid on account of the temporary addition of transverse braces above the bridge trusses and the framework of a horizontal erection platform.

None of the stresses in the trusses were more than 75 per cent. of those calculated for the combined dead and live working loads. A light steam locomotive was moving slowly near the end of the cantilever, but no hoisting was in progress. A light wind was blowing, but not enough to inconvenience the men or have any effect on the structure, which had satisfactorily endured very severe storms.

As fast as the superstructure had been erected from the shore end of the anchor-arm to the middle of the river, during three seasons' work, all portions had been completed as the traveller advanced. All secondary members had been assembled, and all lateral, transverse, and sway-bracing put in place, and all connections fully secured, either with their permanent pins, or, in case of riveted connections, all field-holes had been filled with service bolts until the rivets were driven.

In the anchor-arm 90 per cent. of all, and in the cantilever-arm 50 per cent. of all field-rivets had been driven, and three gangs of riveters were at work on the top bracing and on the last two bottom chord splices of the anchor-arm. Six riveting gangs were at work in various places on the cantilever arm. At least 90 per cent. of all open holes for field-rivets were filled with the largest service bolts that could be entered.

The erectors were furnished with elaborate diagrams showing the sequence and details of all erection operations, the dimensions and weights of principal members, the special attachments made to connect them to the hoisting tackles, the arrangement of members at panel points, &c., with directions what tackles to use, where to suspend them, how to shift and operate them, how and when to adjust different members, and when the joints could be riveted, &c. Besides this they were furnished with 79-page blue-print pocket-books of sketches and minute instructions for the details of assembling connections, riveting splices, moving, securing and operating the traveller, attaching hoisting tackles to members, &c.

The inspectors rigidly enforced the requirement that the top flange cover-plates of the bottom chord-splices should never, on any consideration, be removed; and that if it became unavoidable to remove the bottom flange cover-plate, special reinforcement angles should first be secured to the webs. It is stated that the riveters never did remove the bottom flange cover-plates, and that they never removed more than five service bolts at once in any splice.

The field rivet-holes were so accurate that one air-reamer was more than sufficient to fit up for nine riveting gangs. The field-rivets were driven by pneumatic hammers, and were of such good quality that very few were condemned by the inspectors. The most vigilant inspection of all the erection apparatus was constantly exercised by the foremen and the men under them, most of whom were experienced bridge-erectors. Three of the most skillful were constantly stationed on top of the traveller to attend the tackles there and watch for any indications of wear or defects in them. If such were discovered, no attempt was made to repair them, but they were immediately replaced with new from a large stock kept in storage.

Many of the secondary riveted joints in the trusses, at first open to allow for the change in position and lengths of members due to the progressive stresses of erection, had been closed and every joint of the structure was close and perfect, and every member in the required position. The main vertical posts, at first inclined about 15 in. towards the shore at the top, had gradually moved to within $\frac{1}{4}$ in. of plumb, and all stresses were normal.

No important erection operations were in progress; no heavy members were being handled. There was no opportunity for a failure of tackles, machinery, or travellers, or for any sudden stress or impact, and no chance that any mistake had been made in important erection methods, operations, or adjustments. Besides lacking the metal for the last three panels of the south half of the centre suspended span, then in process of erection as an extension of the cantilever arm, the weight of the structure fell short of the final dead load by many hundred tons, due to the permanent track-rails, ties, &c., not yet in place.

A few weeks previously the adjustable connections between the cantilever and centre spans had been operated easily and successfully to correct a slight difference in position of the vertical posts in the centre trusses, and the top-chord toggles set at mean position, shimmed solid with steel plates, and the hydraulic jacks removed, thus eliminating possibility of trouble at this point.

Levels taken at the panel-points, after the last movement of the traveller, showed the position of the second panel-point from the extremity to be within $\frac{3}{8}$ in. from its calculated height, thus absolutely verifying the assumptions of elastic limit, modulus of elasticity, primary stresses, and quality of materials. Observations of the alignment showed it to be exact within 1 in., and within the transverse variation due to the varying position of the sun. It is thus evident that the collapse could hardly have been due to a blunder, to any ordinary direct accident, or probably, even, to a single simple factor. It is more likely it was due to the cumulative effects of several influences.

Transit and level observations, made on the anchor and main-pier masonry immediately after the collapse, corresponded absolutely with those made just before it, and showed that no movement whatever had taken place in the substructure, which was, besides, absolutely uninjured as far as could be observed, except for trifling damage to the sharp corners of the coping from the impact of heavy steel members. The anchorage was undisturbed, and the integrity of its connection to the superstructure unimpeached. The south 215-ft. deck approach span was supported at the river end on the anchor-pier, but was independent of it and was uninjured.

The shore end of the anchor-arm was connected to the top of a steel bent, about 95 ft. high on top of the anchor-pier, and this bent revolved over 90 deg. about its lower end, crushing part of the plates forming its base, 17 ft. wide, parallel with the bridge axis, and pulling the long thick anchor-bolts out of their grouted holes in the masonry, without otherwise injuring the steel-work of the bent, or breaking its connections. The anchor-arm moved longitudinally towards the river about 100 ft. to correspond with this, thus advancing most of the panel-points (all except, perhaps, those nearest the main pier, which could not be located at the time of the personal examination from which this description was prepared) almost exactly two panels.

The top chords and top lateral system of the anchor-arm remained on top of the débris in comparatively correct relative position, although the light lateral members were badly broken between their connections at the ends and intersections, which endured remarkably well. The top chords were continuous and almost in line from the shore end to across the main pier, and were in astonishingly good condition, with all connections practically intact, except where, in some cases, the outside bars had been forced off the ends of the pins.

The chord was built up of 16-in. by $2\frac{1}{8}$-in. eye-bars over 50 ft. long, as many as thirty in the maximum panels, and these were almost uninjured. Of course, many of them were severely bent and twisted, but with the exception of a single one which had been sheared square off, there was no indication visible of any tendency to failure in any of them, or of the eye-bars in any of the other tension members. Many of the eye-bars had connections to transverse plates riveted across the web members of the trusses, and these were all in notably good condition.

The compression members, of course, suffered severely, and many of them were completely destroyed, as was inevitable from the momentum of their great mass falling, in most cases at least 150 ft., and in some instances nearly 400 ft. It was noticeable that a large number of their connections endured, and that a large number of failures occurred, as was natural, in the body of the members where the latticing, even that composed of large well-riveted angles, which connected the webs or built channels composing the members, broke through or sheared off their rivets, and allowed the webs to buckle and wreck the member before it developed its full compressive strength. Evidently, if a portion of the same total weight of steel in the member had been distributed in diaphragms, or full-length flange cover-plates, the ultimate strength and efficiency of the members would have been increased, as the complete rectangular cross-sections formed by the tie-plates were generally found whole.

The lower chords fell across pits excavated for

* [Reproductions of photographs of the collapsed bridge were given on pages 366 and 371 of our last issue, and this week we are able to give further views on Plate LII. and on pages 394 and 395.—ED. E.]

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the grillage foundations of the false-work, and acted as girders to support the vertical posts over the centre points. They were thus subjected to enormous shearing and bending stresses, and in most cases were broken through the splices, of which there were two in each panel, one shop-riveted at the bevelled intersection on the pin, and the other field-riveted a few feet beyond. All of these splices were made with double splice-plates on the four thick webs, 4½ ft. deep, and with long thick single top and bottom flange cover-plates, most of them having 1000 or more 1-in. field-rivets each.

In some cases the fracture occurred through the shearing of rivets; in some, through the shearing of the plates, usually through the rivet-holes, and in other cases through the shearing of both rivets and plates. Both of the two lower chord splices, at panel points 8 and 5 on opposite sides of the bridge, which were being field-riveted at the time of the collapse, were found to have been fully riveted or bolted at that time.

The west truss lower chord in panel 9 (counting from the anchor pier, or second from the main pier) was 57 ft. long, and inclined at an angle of about 45 deg. with the horizontal. It was proportioned for a maximum working load of 15,268,000 lb., had a sectional area of 735 square inches, and, like all the other lower-chord members in the anchor-arm, was built with four webs 4½ ft. deep, connected by flange cover-plates at the ends, and by intermediate angle-bar latticing on both flanges, and was about 6½ ft. wide over all.

It has been stated that this member had been subjected to slight injuries at the shop and in transportation, which had been repaired, and were not believed to have caused permanent damage. After collapse this member was found in worse condition than any other lower-chord section visible, being bent to the shape of a letter S. The corresponding chord in the opposite truss was said to be almost as bad, but was not examined by the writer.

The destruction of the steel-work and the horrible confusion of members was most terrible on the main pier, where the enormous main posts were wrecked, both with their feet and shoes on the shore side and their upper ends on the river side; their finials, 40 ft. long beyond the centres of the upper pins, projecting just above high water, and in an approximately horizontal position, pointing east and north-east. One post was broken entirely through, and the V-shape piece in a vertical plane on the river side, separated 20 ft. or more from the W-shape part in a vertical plane on the shore side. The shoes and pedestals, and connected portions of the lower chords, are believed to be still attached to the lower ends of the posts by the 24-in. main pins.

The enormous bolsters which received them are unmoved, and practically intact on the pier top, and between them the débris is piled 20 ft. high. Both end panels of the cantilever arm bottom chord are stated to project over the pier half their length towards the shore, but it was very difficult to identify them positively in the confused mass of wreckage. Of course, the thick plates and angles were torn and twisted, bent, crushed, sheared, and broken in the most violent manner, but the behaviour of the metal, and the appearances of the fractures, showed everywhere the highest quality of metal and workmanship. In several cases the halves of large struts were separated longitudinally for long distances, and in, at least, three cases all the many rows of rivets connecting the plates with which a thick web was built up were entirely sheared for 20 ft. or more, allowing the plates to be completely separated.

An analysis of the balanced stresses at the main pier, the results of suddenly removing one of the reactions there, the longitudinal movement of the anchor-arm trusses in falling, the integrity of the top chords, and the condition of lower-chord member No. 9, are interpreted as strong, if not conclusive, evidence that the initial failure occurred in panel 9 of the lower chord of the west truss of the anchor-arm. But so far there has been no evidence discovered in the wreck tending to indicate in any degree the cause of the failure of the member.

It is, however, very significant that two days before the collapse it was reported to the chief engineer that the webs of west lower chord No. 9 were buckled. He considered this proof of serious failure in a vital truss member as unimportant, and no steps were taken to reinforce the chord, block, clamp, or brace it, and the erection was allowed to proceed and the men to remain on the bridge until the inspector made a special trip to report to the consulting engineer in New York, where telegraphic communication was uncertain on account of a strike. The consulting engineer promptly instructed precautionary measures to be taken and an investigation, but this message was not received at the bridge before the disaster.

In a description of the bridge, published in the *Engineering Record*, of April 1, 1905, after approval by the Phœnix Bridge Company, it is stated that:—"No allowance is made for secondary stresses." In the *Engineering Record*, of August 10, 1907, there are illustrations of the detail of the connection of the end lower chord sections of the anchor and cantilever arms to the main pedestal and vertical post, and to each other across the main pier by a rigid V-shape piece over 20 ft. long, which engages the 24-in. main pin, and has at each end a field-riveted splice to the adjacent lower chord.* Since the disaster it has been pointed out that this type of connection, preventing free adjustment of the chords on the pin, must cause contra-flexure under deflection, and that the secondary stresses and deflections produced in the first two panels on each side of the main pier would, on account of the great depth of the truss, be very great, probably increasing the strains to nearly, or quite, double the primary stresses—a possibility which was doubtless investigated, and in some way provided for by the designers, notwithstanding the statement that the secondary stresses were neglected.†

It is also obvious that it is very difficult to obtain perfect accuracy in the manufacture or subsequent inspection and checking of this member. Any error in the boring, facing, or angle of this piece would be very serious, and would affect the lower chords in the two adjacent double panels. An error of $\frac{1}{32}$-in. in a 50-ft. length of the chord would cause a concentration of pressure and increase the stress 1600 lb. per square inch. An extremely slight inaccuracy in facing the ends of any chord section is more easily made when, as in this case, they are oblique instead of normal to the axis of the piece; and if it tended to make one side shorter than the other, it would certainly develop a tendency for the piece to buckle under stress.

While neither of the above causes may have been adequate to cause the failure of lower chord-piece 9, it may be possible that the combination of all of them—namely, increased stress due to secondary stresses, injury to the member before erection, and possible inaccuracy of construction—produced a cumulative effect of importance in the initial failure. This, however, is subject to the most able and thorough investigation before any opinions are formulated, and the above statements are here made only to cover completely all aspects of the case so far developed.

For the same reason it is proper to say that consideration has been given, particularly before attention was called to panel 9, to the possibility of the disaster being due to dynamic forces, particularly as several attempts have been made within the last year or two, some of them successfully, to deliberately wreck travellers, derricks, and steel buildings and bridges under construction in the eastern part of the United States. These attempts have been attributed to labour troubles, and it has been often necessary to guard the work by special watchmen.

Union men were employed exclusively on the Quebec Bridge, and although a short strike had occurred, and bitter feelings are said to have existed, the strike had been declared off and work resumed a number of days before the collapse. No evidence has been found of malicious causes for the collapse.

The possible causes for the collapse of a cantilever bridge during erection include—faulty design, overloading, violent wind stresses, failure of substructure, failure of anchorage, mistakes in assembling members, joints weakened during riveting, omission of bracing, imperfect connections, mistakes in erection, accidents in hoisting, dropping heavy members, breaking tackles or machinery, &c., overturning or collapse of traveller, displacement or failure of adjustment connection between cantilever and suspended centre span, serious inaccuracies of workmanship, weakened or injured members, defective material, malicious mischief.

Most of these possibilities are eliminated by the facts already established; the remainder, and any others which may be discovered, should be investigated with the utmost thoroughness by the most competent specialists. The engineers associated with the work are making every effort to discover the cause of the collapse; a committee has been appointed by the Dominion Government to make a full investigation, and every facility has been afforded visiting engineers to make a full investigation and secure all possible data.

Although the bulk of the superstructure fell through a vertical distance of from 150 ft. to 400 ft., some of the men working on the highest parts escaped alive, and are expected to recover from their injuries. One of them fell about 400 ft. from the top of the main traveller, another 250 ft. from the top of the small traveller, both, of course, falling in the water, and probably remaining on the steelwork as it went down. Two brothers holding on for riveters at the top of post P 1, about 200 ft. above the rocky ground, fell inside the post and the adjacent transverse strut and escaped fatal injury.

A very careful survey has been made for the accurate location of members in the anchor arm, which, with a small part of the cantilever arm, can be removed with great difficulty and expense. The remainder of the superstructure will doubtless be left undisturbed in the deep water, where it does not obstruct navigation. No authoritative statement has yet been made of the reconstruction of the bridge; but on account of its importance, and the large sum already expended, and because the substructure and more than half of the superstructure is uninjured, and the great loss is still only a moderate fraction of the total cost, it seems probable that the construction will be soon resumed.

* This detail is fully illustrated on page 386 of the present issue of ENGINEERING.

† We have commented on this hypothesis on page 401 of this issue.

THE "MAURETANIA."

THE second of the two great turbine-driven quadruple-screw steamers built for the Cunard Company, under agreement with the Government, has now entered upon her trials, and her performance will certainly share with the Atlantic runs of the Lusitania the keen interest of the maritime world. The Lusitania, which has quite met expectations in her maiden trip to New York, notwithstanding fog, will leave on her return voyage to-morrow, and, no doubt, the engineering staff having become more accustomed to the work, she will improve on her performance last week, but it will be some time before any effort will be made to get the best out of the machinery.

The Mauretania left the works of the builders on Tuesday last; and when it is remembered that to-day is only the anniversary of the launch of the ship, it will be admitted that the performances of the builders—Messrs. Swan, Hunter, and Wigham Richardson, Limited—and of the engine constructors—the Wallsend Slipway and Engineering Company, Limited—are worthy of the best records of the British marine industry. An enormous amount of work had to be done to convert the hull of steel into the superb habitable quarters now possessed by the ship; while the installation of boilers and machinery, with all their auxiliaries, representing close upon 70,000 horsepower, necessarily involved careful organisation and large experience.

The movement of the ship down the river was, as our illustration on Plate LIII. shows, a public event of very wide interest, and was carried through most successfully. The distance traversed from the works of the builders to the mouth of the river was just under six miles, and this was traversed in about an hour and a quarter. The vessel was under her own steam, principally for steering purposes, and took the bends of the river very successfully. Our photograph shows the vessel nearing the mouth of the river, and when she passed between the breakwaters she cast-off the tugs, adjusted compasses, and proceeded on a series of preliminary trials, which will not be concluded until to-night or to-morrow. So far everything has gone well, and the machinery is being steadily worked up to meet the severe test which will be exacted later under observation of the Cunard Company and the representatives of the Government.

The Mauretania generally is of the same design as the Lusitania, but a close examination not only of the details of the ship, the decoration of the saloons, and especially the construction of the machinery, discloses a thousand-and-one differences, and these are all of great interest. The engraving we publish on Plate LIII. shows one of these differences at a glance. The Lusitania has square trunks for the ventilation of the stokeholds, and these were covered with hinged covers which could be bent to any extent and swivelled; the Mauretania, on the other hand, has ordinary cowls, as shown in our engraving. At the present moment, however, it is not our intention to enter upon any lengthened description of the vessel. The time has not come for that.

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THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(*For Description, see Page* 388.)

FIG. 71. SECTION 9 OF LEFT-HAND LOWER CHORD OF ANCHOR-ARM.

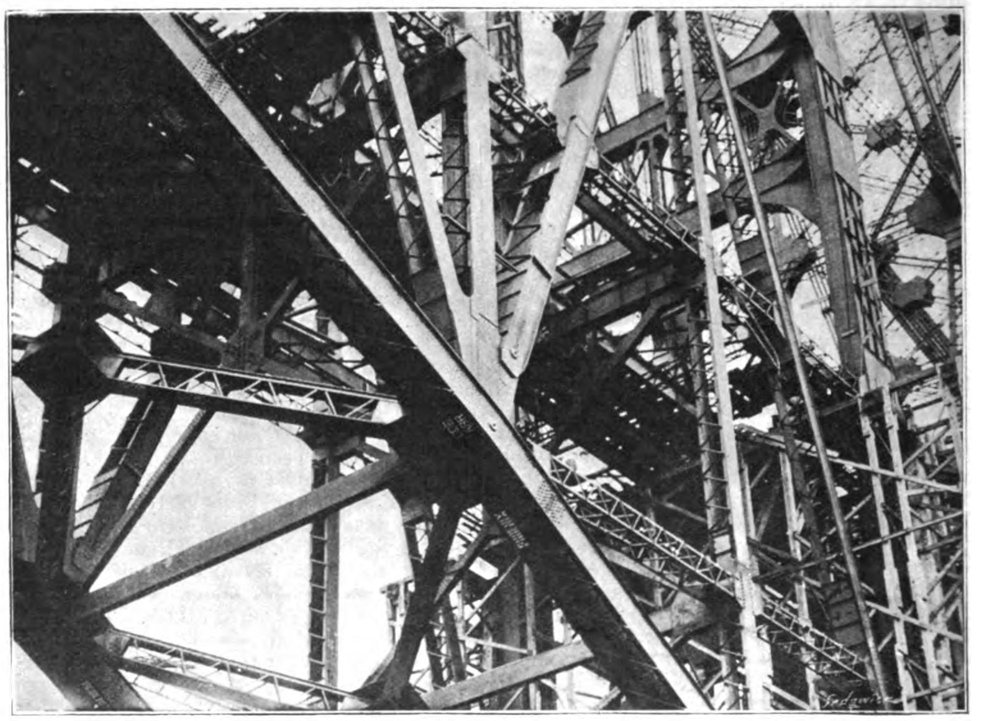

FIG. 72. PORTION OF LOWER CHORD OF ANCHOR-ARM AND BRACING.

DRUMMOND'S SCREW-CUTTING LATHES.

WITH the advent of the motor-car there came a special need for small handy tools capable not only of turning out a great variety of work with rapidity, but also of executing that work with great accuracy. To meet this need the lathe has, among machine-tools, naturally filled a very important place. So many operations can be economically performed upon it that its popularity is not surprising; and there need be no wonder at the great improvements that have recently been made in its design. Not only have these improvements been called for in large numbers by the makers of motor-cars, but their advantage has, perhaps, been recognised quite as much in the garage and in the small repairing-shop, where breakdowns of all kinds have to be attended to and new parts have to be supplied, often at very short notice and under peculiar circumstances. It has come, therefore, that great ingenuity in the design and manufacture of these tools has been displayed, till some of them seem almost perfect instruments for the performance of the many operations expected from them.

Among lathes of the class to which we have alluded, those made by Messrs. Drummond Brothers, Limited, Ryde's Hill, near Guildford, Surrey, occupy a deservedly high place for design and finish, as well as for accuracy of workmanship. They have not been on the market more than three or four years, but during that time their merits have become widely recognised. This being the case, some description of them will be of interest to our readers. The first of these tools to which we shall call attention is a 5-in. centre, self-acting, sliding, surfacing, boring and screw-cutting lathe for treadle or for power, a general view of which is given in Fig. 1, page 398. Repairs to motors and motor-cars is the particular class of work for which it has been designed, and it is claimed for it that it is the only lathe of its size constructed with a special view to this class of work. The object aimed at during its construction was to supply a machine that would enable an ordinary mechanic to undertake special work, so that he need send no repairs away from his own shop. Such particular work, for instance, as the re-boring of a pair of twin cylinders can easily be done upon this machine, an operation which on an ordinary lathe would require special care and skill. It is also claimed that the general capacity of the lathe is much greater than that of any other lathe of its size, for it can deal with work which, if a lathe of the ordinary pattern were used, would require one of 6 in. or even 7-in. size. As an example of this, it will, in rough hard cast iron with the scale on, easily take a cut $\frac{1}{8}$ in. deep with a coarse feed, and $\frac{1}{4}$ in. cut with fine feed, in pieces 14 in. in diameter, worked either by treadle or power; and, although capable of this, its extreme accuracy for light work remains unimpaired.

The special features of the tool are that the slide-rails on which the saddle is guided are formed at the level of the bottom of the gap, and extend right along the gap space, instead of being formed at the top level and stopping short at the gap-space. This is shown very clearly in our illustration, Fig. 1, where the slide is drawn back and the gap exposed. It will be readily seen that with this arrangement the saddle can at any time be run right over the gap-space up to the largest face-plate working in the gap, and there will be no overhang of tool or saddle. The ordinary clumsy gap-piece is in this manner done away with, and all the trouble of removing and replacing it does not exist. The advantage of a straight unbroken bed is nevertheless retained, as well as the convenience of the gap, which is an excellent feature of the tool. The bed is of the box form, with heavy side gibs and the usual cross-braces, which is, of course, a much stiffer form than the ordinary one, which is slit right through from end to end.

The cross-slide of the compound slide-rest is formed by a faced ⊥-slotted boring or milling-table of large size, as will be seen by Fig. 2. The advantage of this is that ordinary boring can be conveniently done on it, and holes parallel with each other can be bored in the same casting, such as twin motor cylinders, at one setting, the distance from centre to centre being easily obtained by the micrometer attached to the cross-slide screw. Self-acting cross and longitudinal feeds are fitted to the table, so that the lathe is suitable for any milling work not requiring a vertical slide. The use of the table when boring out twin cylinders is shown in Fig. 2.

Another feature of the lathe is the construction of the tail-stock. This is rigidly locked to lateral V guides formed solid on the edges of the bed, as well as down the face of the flat surface of the bed. It takes its bearing all along the front edge, and is fixed by two bolts, instead of the usual one central bolt. By this arrangement the full surface of the base is in use to resist the strain of work, instead of about one quarter of such surface. The overhang of the centre is also made just the required amount of the projection of barrel, instead of having, as is often the case, a part of the tail-stock casting projecting forward of the holding-down bolt. The tail-stock barrel is guided the full length of the casting when fully extended. All the self-acting feed motions are by screw, the lead screw being used for screw-cutting only, and it does not

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THE COLLAPSE OF THE QUEBEC BRIDGE.

(*For Description, see Page* 391.)

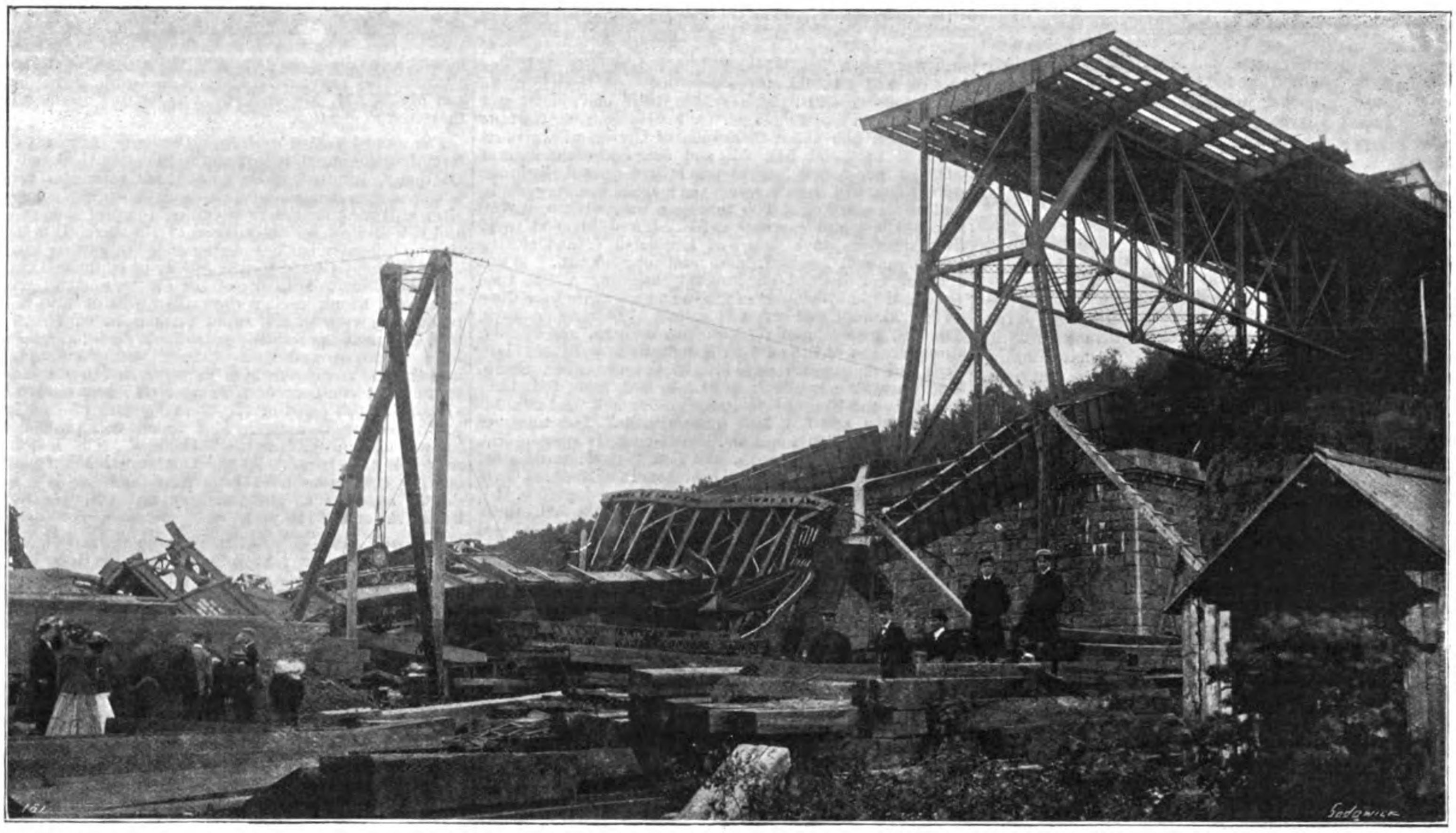

FIG. 5. THE ANCHOR-PIER WITH THE UNINJURED APPROACH SPAN.

revolve when not in use for such purpose. The top slide of the rest can be turned completely round, and it is graduated for taper turning. The lathe is fitted with a set of machine-cut change-wheels for cutting all Whitworth threads from $\frac{1}{16}$ in. 60 pitch to $2\frac{1}{4}$ in. 4 pitch, as well as all gas and fine threads, including 26 pitch, and the usual metric threads. Odd and fine threads up to 120 per inch, right or left hand, can also be cut. The accuracy of workmanship is guaranteed to be within $\frac{1}{1000}$ in., and to a much finer margin where required.

We now come to a particularly handy little tool, suitable for accurate small work. This is a $3\frac{1}{2}$-in. centre self-acting sliding, boring, and screw-cutting lathe, which we illustrate in Fig. 3, page 398. It has been designed for model engineering and precise work, &c.; in fact, for any light work, and it forms a very low-priced tool, capable of turning out excellent work. The makers guarantee that the error in any part of the machine nowhere exceeds $\frac{1}{1000}$ in.; that is to say, in such matters as the alignment of centres through mandrel and tail-stock, the slide of the saddle on bed, &c. This is quite the maximum error allowed. The circularity of the mandrel, tail-barrel, and similar parts is brought to a much higher degree of accuracy. The working limits allowed in the shop lie between $\frac{1}{1000}$ in. to $\frac{1}{10,000}$ in., according to the part. This little lathe, in fact, appears to be made about as accurately as is possible with ordinary methods of manufacture.

The slide-rest of the lathe can be detached by unscrewing one nut, thereby leaving a truly-surfaced $\perp$-slotted, self-acting, sliding, boring carriage, and converting the lathe into a boring lathe for accurately boring cylinders and other long true holes, such as dynamo and engine-bearings, &c. The slide-rest is made to swivel right round, and the headstock may be set over for such work as making small taper plug-cocks, and boring the holes for the same to an exact angle; for which purpose it is graduated. Long taper turning can also be done by setting over the tail-stock, which is so made that it is locked to the outside of the bed, instead of fitting loosely in a slot, as in the ordinary pattern of lathe. For small work and for wood-turning the lathe can be speeded high, as well as low for large work, so that it is well adapted for the great variety of work that often passes through the hands of the amateur engineer and the jobbing machinist.

The bed is strongly made, and has a short gap. It is mounted, as shown, on a cast-iron tray and standards. The mandrel is hollow steel, and has a $\frac{5}{8}$-in. diameter hole through it, which is bored from the solid, and allows rods to pass through when required. It runs in concentrically-adjustable gun-metal bearings. The 8-pitch lead screw is of steel, and runs the full length of the bed, and is fitted with a hand-wheel. It can be thrown out by the lever seen just below the gap in Fig. 3. Change-wheels, for cutting all Whitworth threads from $\frac{1}{16}$ in. to 1 in. in diameter, and also fine gas threads, for brass and iron tube, are provided.

Another very handy lathe manufactured by the same firm is illustrated in Fig. 4. This is known as the "Workman's" lathe, and though not such a generally useful tool as the one we have just described, it is one much appreciated by motor and cycle repairers and general machinists, who are not in a a large way of business, and who, perhaps, are unable to afford a more expensive tool, but who, nevertheless, must have a lathe that will in every way do accurate work, and yet is able to stand a considerable amount of abuse from jobbing mechanics. It has many of the best features of the 5-in. lathe just described, and the workmanship is quite as good. It differs from this lathe, however, in that all parts not absolutely necessary, or those which are merely conveniences, are omitted.

In motor work and, indeed, in any work connected with oil, petrol, or steam-engines a large proportion of the machining done is of a boring character, and for this, ordinary lathes are not particularly well adapted. On the lathe we are now describing this class of work can be carried out easily and well, without any particular skill on the part of the operator, the lathe having been specially designed for this class of work, its capability for taking other kinds of work being in no way impaired thereby. A 4-in. cylinder such as is used on a steam-launch may easily be fixed on this table. A job of this kind may easily be done by treadle. The table of this lathe is also made suitable for home-made jigs or special appliances for unusual work, and there are many other jobs that might be enumerated which can be done by the exercise of a little ingenuity.

The bed of the lathe is of the girder form and the box-type of head-stock is used. By a special arrangement of novel raising-pieces, which are inserted below the headstock, the latter can be raised to 9-in. centres, enabling work 18 in. in diameter to be machined. The accuracy of the lathe is in no way reduced thereby. Screw threads similar to those cut by the lathe previously mentioned can be cut in this lathe. The treadle motion is also similar to that in the 5-in. lathe.

Other accurate tools are made by Messrs. Drummond Brothers in addition to the lathes we have described, but with these we are at present not concerned, though we may at some future time have to allude to them. Some of these lathes are to be seen in the Engineering and Machinery Exhibition, Olympia, Kensington, W., which opened yesterday.

A NEW METHOD OF DETERMINING THE INTENSITY OF GRAVITY.—In the Observatory of Florence, Peter Pagnini is trying a new method of determining the intensity of gravity. With the aid of a chronograph, the rates of two pendulums are compared. The one is an ordinary pendulum, whose period is variable with gravity; the other a horizontal torsion pendulum of the Coulomb balance type, whose period depends only upon its inertia. The oscillations of both these pendulums are electrically recorded on the same tape, and the determinations are hence reduced to the measurement of the records; slight irregularities in the rate of the chronograph do not matter, as the ratio of the two records is the chief thing. The torsion pendulum was first suspended by 1 metre of invar wire 0.477 millimetre in diameter, with which, however, the troublesome temperature influences could not be eliminated. Then a wire of platinum-silver was tried, and a quartz fibre suspension is now to be applied. According to the preliminary account in the *Journal de Physique*, Pagnini was not decided yet whether the method permitted of sufficient accuracy.

LONDON COUNTY COUNCIL SCHOLARSHIPS.—We have received a pamphlet from the London County Council relating to their scholarships, which also gives information concerning the admission to their training colleges, &c. The scholarships awarded by the Council may be divided into two classes. In one of these groups are scholarships of a general nature, enabling scholars to proceed to Universities, &c.; while the other group is provided for scholars who desire to devote their attention to special subjects. In the latter class some 60 art scholarships are awarded to students and young artisans, 120 evening art exhibitions, 250 evening exhibitions in science and technology, 30 trade scholarships for boys, 20 Borough Polytechnic Institute scholarships, 10 free places at the London School of Economics, and other scholarships in connection with the Shoreditch and other Technical Institutes. In addition to these there are numerous awards in subjects which do not immediately interest us. The pamphlet gives a great deal of information concerning the work of the Council, and it may be obtained for 1d. at all the Council and non-provided schools.

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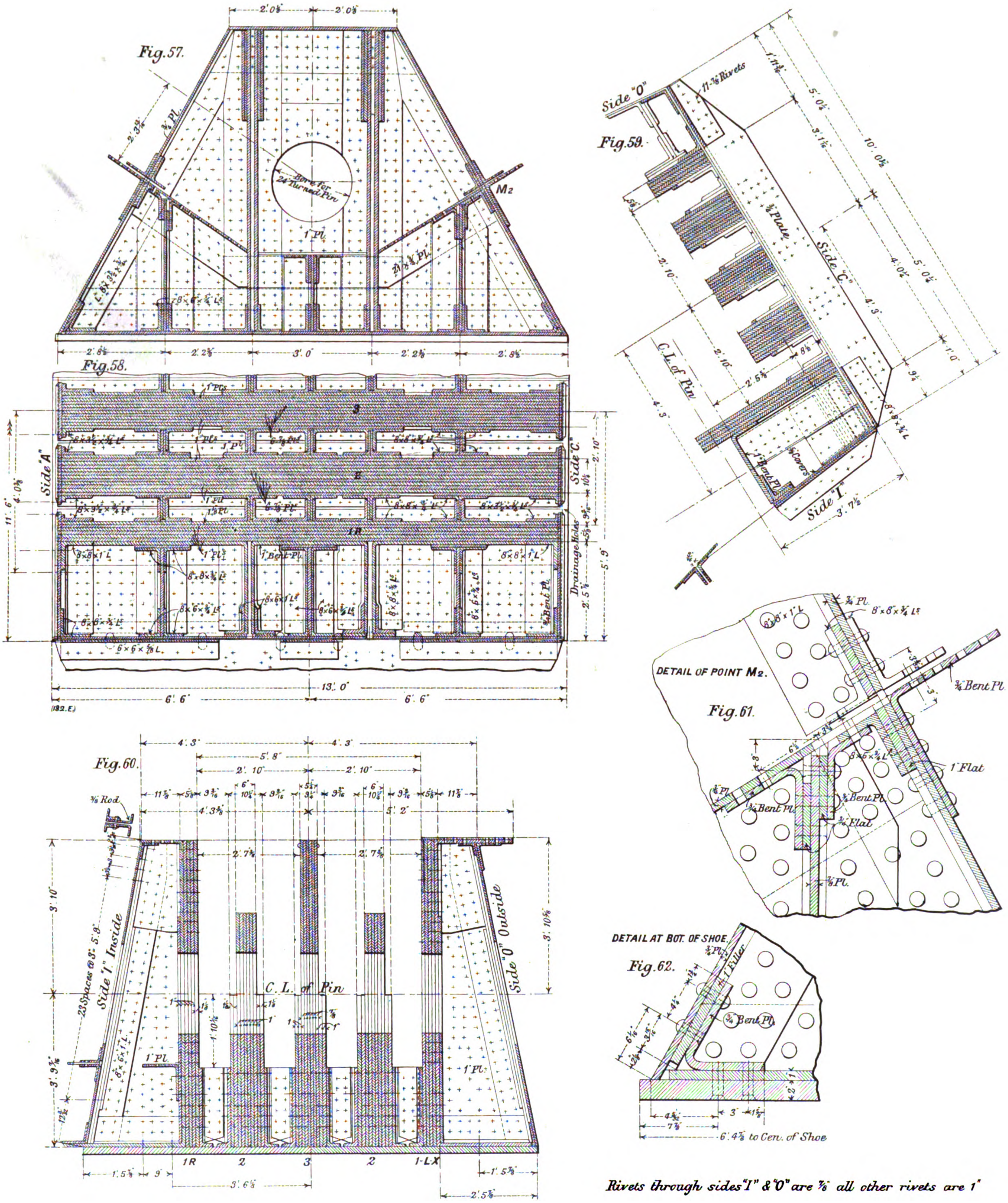
THE CANTILEVER BRIDGE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(For Description, see Page 388.)



Rivets through sides "I" & "O" are 7/8 all other rivets are 1"

Figs. 57 to 62. Main Post Shoes.

(To face Page 392.)

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THE COLLAPSE OF THE QUEBEC BRIDGE.

(*For Description, see Page 391.*)

FIG. 1. VIEW OF THE WRECK FROM THE RIVER.

FIG. 2. THE FALLEN SHORE-ARM.

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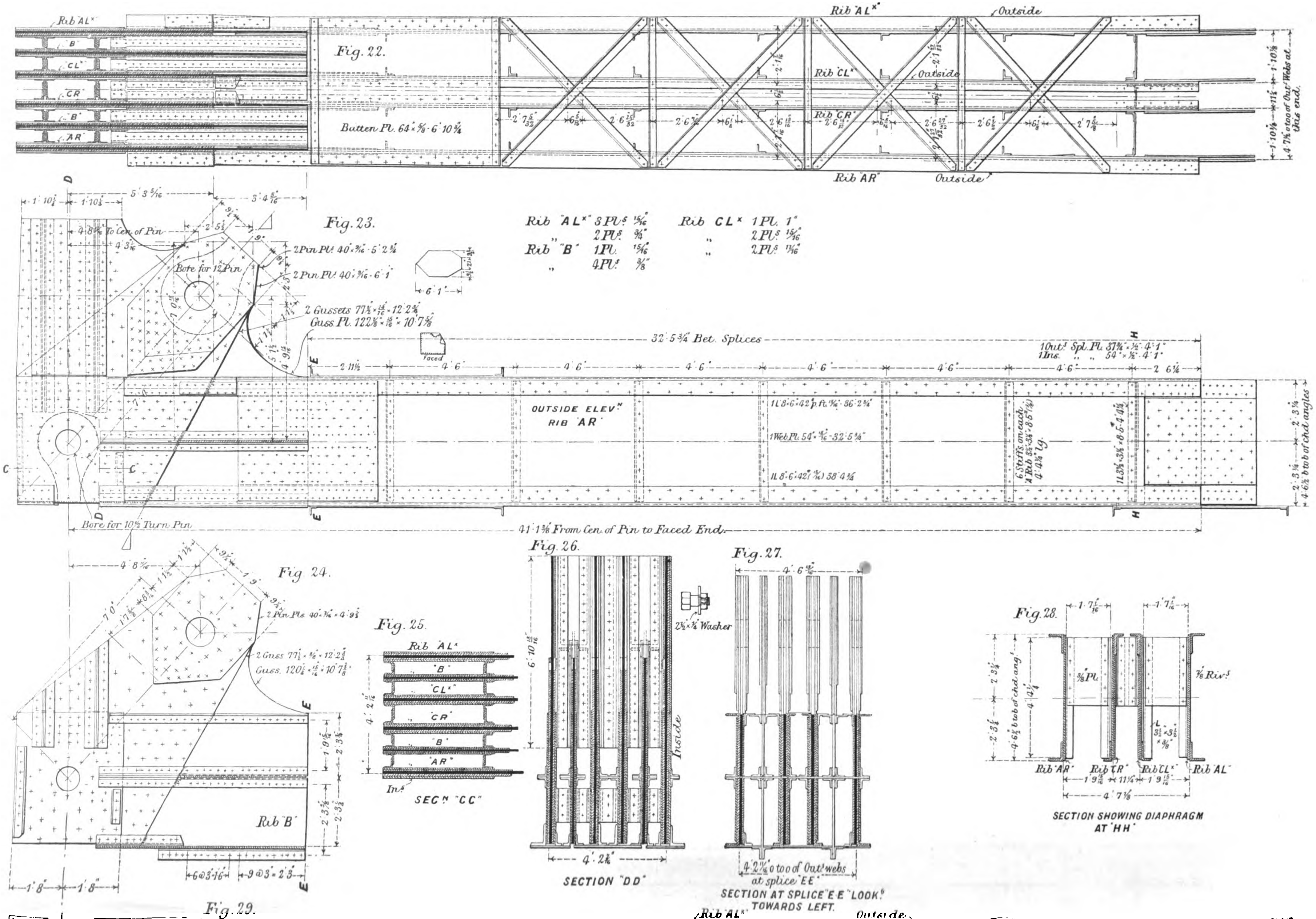
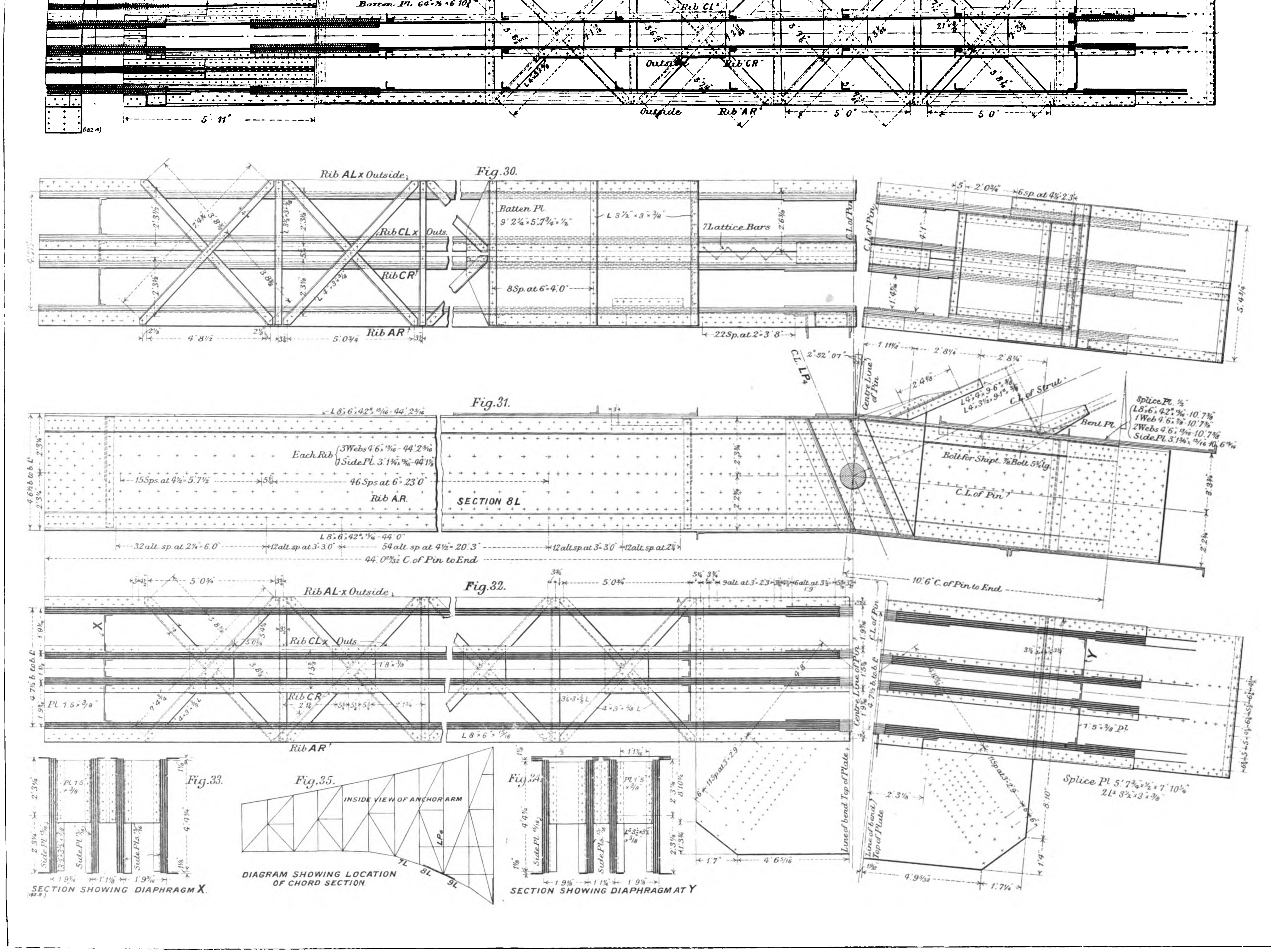
THE CANTILEVER BRIDGE AT QUEBEC; DETAILS OF LOWER CHORDS.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(For Description, see Page 388.)





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THE COLLAPSE OF THE QUEBEC BRIDGE.

(For Description, see Page 391.)

Fig. 3. Broken Steel-Work piled over and around the River Pier.

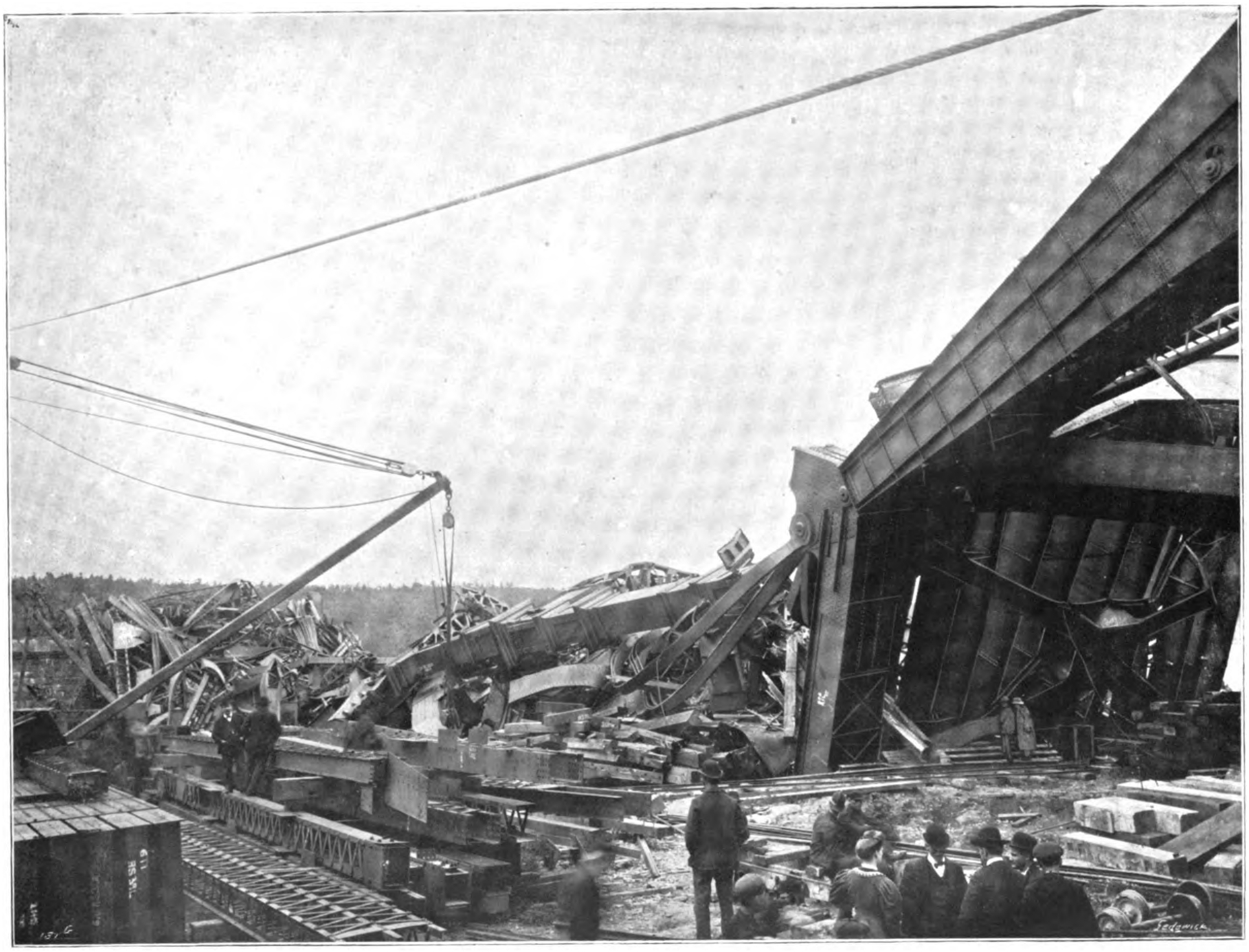

Fig. 4. The Wreck near the Anchor Pier.

are altogether less powerful engines. A dynamometer car has been put into service by the locomotive department, but the results obtained are, it seems, not as yet all that might be desired. It is, however, quite a pleasure to know that the handling of traffic has been brought to such a pitch of refinement that a car of this description can be put to practical use. The works of the locomotive department have been busy during the year on new rolling stock, and in addition to the building of coaches, &c., for their own system, work on the first instalment of a set of corridor vehicles has been started for the Central South African Railways. The report states that in all twenty-four vehicles are under construction for the sister administration, comprising four dining-cars, twelve first-class cars, and eight second-class. Fourteen new vehicles of first and second-class types have also been built for their own lines, besides other new stock, while the conversion of a number of low-sided wagons of 16 tons capacity to high-sided wagons of 22 tons capacity has also been undertaken.

For the lighting of trains by electricity, the Natal railways are using Stone's system, and also a system employing accumulators in a van, the coaches being simply wired. Only a small proportion of the passenger vehicles are now without electric light. There are in use 380 vehicles wired for the van system, 32 wired for both the van and for Stone's system, and 42 for Stone's system only. There are altogether 73 Stone vehicles, and 50 vans, brakes, or composites fitted with accumulators. According to the report the cost per carriage trip for the Stone system works out at 4s. The cost of lighting on the van system comes out at 7s. 2d. per train-trip, which includes cost of current, charging the van-accumulators, stores, &c.

The report is too detailed to deal with exhaustively, and so many matters worthy of attention are noticed in its pages that a complete *résumé* of the situation is scarcely possible in the space at our disposal. We have endeavoured to select those which appear to have had the greater bearing on the broader questions, going, however, more into detail on a few points of special interest to our readers. In summing up, then, it may be said that though the year has been less prosperous than 1905, in 1906 the locomotive expenses, traffic expenses, and maintenance of way and works per open mile, were all less than in the previous year; and had the internal state of affairs in Natal and in the neighbouring colonies been of a brighter character, the railway results would have been altogether satisfactory, there having resulted to the colony as it is a net profit of 62,368*l*.

PUMPING WITH OIL-ENGINES IN INDIA.

IN a country like India, where long periods of drought are often experienced and where, consequently, irrigation has to be resorted to, if the natural fertility of the soil is to be utilised to the best advantage, it becomes necessary in many cases to employ mechanical power for raising water from the wells that are the common source of supply. In many parts of the country the holdings, farms, or estates, whichever they may be called, are mostly in private hands, and are not of very great extent. In such cases, therefore, each may have its own well or wells and its own installation of pumping plant; and the question of importance is, what form of motive power shall be employed.

In certain parts of the United States of America small wind-engines appear to be popular for this purpose, both in the Prairie States and in those bordering the Pacific Coast; although these wind-engines are perhaps as much or more used for watering stock as for irrigation purposes. In India, however, the wind-engine does not seem to be regarded with quite so much favour, and other sources of power are employed, one of which—the oil-engine—appears to be making considerable headway. Indeed, so much importance is attached to it that, with the object of obtaining more reliable knowledge on the subject, the Government of Madras has for some two or three years been carrying out a series of experiments in which oil-engines have been used. The results of these experiments for the year 1905-6 are embodied in a report by Mr. Alfred Chatterton, Assoc. M. Inst. C.E., of the Madras College of Engineering. The objects for which the experiments were started were —(1) to ascertain the actual cost of raising water under different conditions; (2) to ascertain the duty of water under varying conditions as regards crop, soil, and distribution and rainfall; (3) to demonstrate that with a system of intense cultivation it is profitable to employ small oil-engines and pumps for the irrigation of comparatively small areas of land; (4) to ascertain the quantity of water available for lift irrigation throughout the year from various sources of supply, such as wells, open sewers, tanks, and natural lakes; and (5) to study the distribution of the underground water in various parts of the Presidency, and to devise methods whereby it may be much more largely made use of than at present. We will confine ourselves to the first of these. Before the experiments were definitely commenced, a number of different oil-engines were tried; but the one selected was the Hornsby-Ackroyd engine, the main reasons for the selection being the simple character of the mechanism, the fact that it would work satisfactorily on the residue left after the fractional distillation of crude petroleum oil, and that, after once being started, the engine would work without any external lamp. At the time the choice was made there were apparently no other engines in the market (with the exception of the Diesel engine), that would fulfil the last two conditions. Now there are, of course, several makers who manufacture this class of engine, with very satisfactory results. The oil used was imported from the Borneo oil-fields, and could be obtained at the tanks of the importers in Madras for about As. 2 (2d.) per gallon. In practical work it was found to be fairly safe to assume that 1 pint of oil developed 1 brake-horse-power per hour. This oil appears to have the disadvantage that when it is used it is necessary periodically to open the vaporiser in order to remove the deposit of carbon, which should be done about every twelve hours when the engine is working.

When we realise what a nuisance this constant cleaning must be we naturally look for something in the form of a set-off to compensate for it. We find it in the price of the oil. It is easy to understand, when oil of the kind we have named is sold in Madras for about As. 2 (2d.) per gallon, and can be bought at the pumping station (including carriage) for about 3d. per gallon, why a certain amount of trouble and inconvenience in using it is put up with, particularly when the price of kerosine ranges from 5d. to 8d. per gallon, depending on the quality of the oil.

As to the running and maintenance expenses of these small engines, the conditions, of course, vary somewhat in different districts, and are, on the whole, very different from those in this country, labour in India being much lower than here, and maintenance charges much higher. There is really, however, no reason why the maintenance charges in connection with these oil-engines should be high, so far as the engine itself is concerned, at any rate when it is well looked after. But herein lies the difficulty; it is not easy to get engines well looked after, for the native attendant appears to have but crude ideas on the subject. He may be cheap as far as actual wages is concerned; he is often not cheap when his efforts are devoted to remedy any little defect the engine may develop, and it is probable that in many cases this cheap labour has resulted in an increased outlay on repairs not compensated for by the low attendance-bill. Still, this is perhaps not to be avoided under the circumstances. The high cost of running some of the stations has, however, not been altogether due to bad drivers, but also to overload.

It has been found that the most vulnerable part of the engine has been the vaporiser, for the practice of taking the cover off, if carelessly done, results in the nuts being stripped. The hole through which the oil is sprayed into the cylinder also suffers from careless cleaning, and, becoming too large; new spraying-plates and nipples are then required. It is not that the plates in themselves are of much value; it is the time required for drilling the very fine hole necessary that makes them expensive. It is considered, however, that 5 per cent. on the capital outlay on the engine ought to be sufficient to keep them in perfect working order, and this does not seem to us a particularly heavy charge for maintenance if the carelessness of the native attendants really amounts to what it is said to do.

In addition to the above charge for repairs, &c., before we can arrive at a fair estimate of the cost of pumping at these Indian stations, we may mention that the Madras Government lends money under the Agricultural Improvement Loans Act at 5 per cent. interest for the establishment of these pumping installations, to which must be added charges for depreciation, which are taken at 5 per cent.

We may now take a typical example of pumping water with an oil-engine. It is assumed that the installation consists of a 7½ horse-power oil-engine, and a 4-in. pump to be used with a maximum lift of 25 ft. The quantity of water raised will be about 18,000 gallons per hour, and it may be taken, for the sake of our example, that the water supply will be available for 12 hours' running per day. According to Mr. Chatterton such an engine and pump will cost in Madras Rs.2000, to which another Rs.1000 may be added to cover the cost of carriage, fixing, and other charges, making a total cost of Rs.3000. Interest and depreciation are taken at 10 per cent., or Rs.300 per annum, maintenance and repairs 5 per cent. or Rs.150 per annum, the two being equal to a charge of R.1 4a. per day. Three-fourths of a gallon per hour is what the engines will require per hour, or 9 gallons per day. The estimated cost will then be as below:—

	Rs.	a.
Fuel, 9 gallons at 3 annas per gallon ...	1	11
Driver at Rs.15 per month	0	8
Lamp and lubricating oil, waste, &c. ...	0	8
Interest and depreciation 10 per cent., maintenance and repairs 5 per cent. ...	1	4
Total	3	15

About 5s. 8d. per day.

In even figures this may be taken at Rs.4 (6s.) per day, if the plant is run every day in the year. Of course, if the plant does not run every day in the year the cost per day will increase. If we reckon that it works on 200 days in the year, the daily cost works out at Rs.5 10a. (8s. 4d.). The quantity of water lifted 25 ft. per hour is 18,000 gallons, equivalent to 864,000 cubic feet 1 ft. in the course of the day. Comparing this with some lengthy experiments that were carried out some years ago on a farm in Madras, cattle being used as the motive power, it is found that the cost of lifting 4000 cubic feet per hour 1 ft. high was 1 anna, but the rates have since then much increased, so that now it is estimated that only 3000 cubic feet can be raised 1 ft. high for 1 anna. Therefore, according to the figures we have previously given for the cost of pumping with oil-engines, it will be seen that the cost by the latter method will be about one-fourth of what it is when cattle are used, assuming the engine to work every day in the year. If, however, the engine works on only 200 days in the year, 9600 cubic feet would be lifted 1 ft. high for 1a., or a cost of about one-third that of cattle power. Put in another way, or that of the acre foot lifted 1 ft. high, the cost then works out at As.3—3p. (3½d.) per acre foot when the pump is worked every day, and As.4—6p. when it is run on 200 days only.

Other stations gave varying results, which ranged from about 8d. to 2s. 5d. per acre foot lifted 1 ft., according to circumstances. On the whole, the experiments seem to show that there is a greater field for the employment of oil-engines than was anticipated, but up to the present it does not seem possible to do more than indicate localities where prospects appear brightest.

After all, the use of oil-engines in these districts does not appear so much to depend on any difference there may be between the cost of this power and that derived from cattle as on whether it is possible to utilise with profit the water when pumped; and it seems that oil-engines are now doing work that has never before been attempted with cattle, and they make possible the use of water for irrigation purposes in a way not before known. The future ought therefore to see considerable advances made in the use of these motors.

THE QUEBEC BRIDGE DISASTER.

WE publish on page 391 an article by Mr. Frank W. Skinner giving the results of his personal observations at the site of the Quebec disaster.* The data collected by him, the general accuracy of which is further confirmed by an excellent article in the last issue received of the *Engineering News*, leave no doubt that the immediate cause of the disaster was, as surmised in our first article, the failure of a lower chord member. Curiously enough, however, it appears that the member that failed formed part of the shore-arm of the bridge, although

* Mr. Skinner's detailed description of the Quebec Bridge is also continued on page 388.

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the sections of the chord of this span were some 6 ft. to 7 ft. shorter than the corresponding members of the river cantilever. The most conclusive argument in favour of the view that it was the compression member of the shore-arm which first failed is, no doubt, the now established fact that chord member No. 9 of this arm (see Fig. 9 of Plate XLVII. of our last issue) was noticed to be $1\frac{1}{2}$ in. to 2 in. out of line three days before the accident, and this, as we shall show, was in itself quite sufficient to account for the collapse.

In view of this circumstance, it is impossible to understand how it was that work on the bridge was not stopped, immediately on the discovery of the dangerous condition of this strut, until some satisfactory explanation was forthcoming as to its evident weakness. It appears that on receiving notice of the observation of the defect, Mr. Cooper telegraphed to the Phœnix Bridge Company, at Phœnixville, to cease work pending investigation, but this telegram was not received till the very afternoon of the disaster. It is, however, extraordinary that the engineer in charge of the erection, or the resident engineer, did not cease work on his own responsibility pending orders from his principals. They can hardly have been ignorant of the danger, since it appears that the signs of weakness were developed under an actual stress (assuming the stated conditions of loading to be correct) of about 8 tons per square inch, whilst, as shown by the strain-sheet published in Plate XLVII. of our last issue, the member was intended to carry a maximum working stress of something like 13 to 14 tons per square inch. In bridge-erection it may sometimes occur that a member is temporarily overloaded to a stress much in excess of that which it is intended to carry permanently, and if under such conditions signs of incipient failure become evident, it would not necessarily follow that the structure was badly designed. When, however, a strut shows signs of crippling under a load equal to only some 60 per cent. of what it has been built to carry, it is clear that either material, or design, or both, are seriously at fault, and instant and thorough investigation is absolutely necessary before proceeding further with the work of erection. It would seem, therefore, that in the case of the Quebec disaster a very heavy responsibility rests upon the local engineering staff. It is possible that had they acted promptly in lightening the bridge, the loss of life might have been averted.

Mr. Skinner's article fully establishes the fact that bad design, and not bad material, was responsible for the catastrophe. The conditions at the time of the collapse were, he states, everywhere normal or favourable, and the material and workmanship were both excellent of their kind; whilst the substructure, even after the accident, was found in practically perfect condition. Obviously, therefore, the fault must have lain in the general or detailed design of the superstructure. There are no doubt a number of points in which this general design is open to criticism, but the really fatal error appears to lie in the system of bracing adopted for the lower chord members.

As we indicated in our short editorial note last week, the bracing or latticing of these compression members was absurdly light according to English ideas. Full details of these members, for panels Nos. 1 and 8 of the shore cantilever-arm, will be found on Plate XLIX., which we publish this week, and we understand that the bracing of panel No. 9 was identically the same. As mentioned above, the immediate cause of the disaster appears to have been the crippling of this latter strut.

Taking the lighter strut represented in Figs. 22 to 29, Plate XLIX., first into consideration, few engineers on this side of the Atlantic would care to try and hold such thin and deep plates in line by merely latticing them across the top and bottom. The addition of a solid diaphragm-plate running from end to end at mid-depth would undoubtedly have made a better job. Presumably the reason why some such arrangement was not adopted lay in the supposed difficulty of transferring across to the pins the load carried by this plate; and hence the adoption of the design criticised. Although unmechanical, we are not, however, prepared to say that it was actually dangerous in this instance; but there are good grounds for believing that chord member No. 8, and, *a fortiori*, Nos. 9 and 10, were, on the other hand, in a somewhat parlous state. The bracing here was, in short, incapable of developing the full bending strength of the web-plates. If those of member No. 8 were laid flat side by side, over supports 55 ft. apart, they could sustain a uniformly-distributed load of about 53 tons, with a stress in the metal of some 10 tons per square inch. Treating the complete strut in the same way, a load of 40 tons would cause a stress of about $10\frac{3}{4}$ tons per square inch in the bracing members. Each 4-in. by 3-in. angle would then, it is true, take a load of only about $7\frac{1}{2}$ tons on an area of 2.48 square inches; but as this is applied eccentrically through one web only, the actual stress in the metal would reach the figure stated. As a matter of fact, the rivets might, indeed, go first, since these are of $\frac{7}{8}$ in. diameter only. The bending of these bracing bars under their eccentrically applied load would, moreover, allow the webs to get out of line, thus accentuating the effect of their light scantlings.

Unless the load on a strut is applied absolutely centrally, a bending moment is developed, causing an increase in the stress on the metal. In the case of the chord members, this additional stress on the plating is not in itself serious, since the length of the member is only about ten times its width. The same cannot, however, be said as to the stresses due to shear, which are simultaneously developed on the bracing by an eccentrically-applied load.

In support of this view we may take chord member No. 8 when under its intended full dead load, live load, and wind load. From Fig. 9 on Plate XLVII. of our last issue it will be seen that these are as follow:—

	lb.	
Live load	4,347,000	
Dead load	10,732,000	
Wind load	6,870,000	
Total	21,949,000	= 9800 tons

The total area provided is 756.6 square inches, so that the maximum possible stress would amount to close on 13 tons per square inch. The radius of gyration of the strut in the direction of its width appears to be about 19.5 in.

When a strut is eccentrically loaded, the bending moment at any point of its length at a distance x from its mid-point is given by the relation

$$M = \frac{P e}{\cos \frac{l}{r}\left(\frac{P}{\Omega E}\right)^{\frac{1}{2}}} \times \cos \frac{x}{r}\left(\frac{P}{\Omega E}\right)^{\frac{1}{2}}$$

where P denotes the total load, e the eccentricity of its point of application, l the half length of the column, r the radius of gyration, and Ω the area of the section, whilst E is, of course, Young's modulus.

Since the curvature is slight, we have for the shear

$$S = \frac{d M}{d x} = -\frac{1}{r}\left(\frac{P}{\Omega E}\right)^{\frac{1}{2}} \frac{P e \sin \frac{x}{r}\left(\frac{P}{E \Omega}\right)^{\frac{1}{2}}}{\cos \frac{l}{r}\left(\frac{P}{E \Omega}\right)^{\frac{1}{2}}},$$

which will be greatest when $x = l$, in which case

$$S = -\left(\frac{P}{\Omega E}\right)^{\frac{1}{2}} P \frac{e}{r} \tan \frac{l}{r}\left(\frac{P}{E \Omega}\right)^{\frac{1}{2}}.$$

Taking

$$P = 9800,\ \Omega = 756.6,\ E = 13{,}000,\ r = 19.5'',\ \text{and}\ l = 330'',$$

we find that

$$\tan \frac{l}{r}\left(\frac{P}{E \Omega}\right)^{\frac{1}{2}} = \tan 61.2^{\circ} = 1.819,$$

whilst

$$\left(\frac{P}{\Omega E}\right)^{\frac{1}{2}} = 0.0316,$$

whence

$$S = 562 \frac{e}{r}.$$

It has been shown above that a total shear of 20 tons would strain the latticing to nearly 11 tons per square inch. Hence for such a stress the limiting value of $\frac{e}{r}$ is $\frac{20}{562}$; that is to say, the eccentricity of the line of thrust must not exceed $\frac{7}{10}$ in. From Mr. James Christie's experiments on the compressive strength of steel ⊥'s, Professor Claxton Fidler has deduced that the eccentricity of the loading due to non-homogeneity of the metal, and to unavoidable errors in applying the thrust, reached 0.4 of the least radius of gyration. The latter is, however, small in comparison with the over-all dimensions of the struts under test, and nothing like this degree of eccentricity was to be expected in the chord members of the Quebec Bridge. Still, as shown above, a very much smaller displacement of the line of thrust would perilously overstrain the bracing actually used, which was quite incapable of developing the full compressive strength of the struts. This will be evident when it is remembered that the stress on the bracing is directly proportional to e, whilst the direct stresses on the plate-webs increase much less rapidly when e is increased. Thus with zero eccentricity of the thrust the direct stress in the case of chord member No. 8 is about 13 tons under the combined action of dead load, live load, and wind load. With e = 1.5 in. the direct stress would only be increased by some 25 per cent., making a total of, say, $16\frac{1}{2}$ tons per square inch.

The bracing, on the other hand, with the same eccentricity, would be subject to a stress of 24 to 25 tons per square inch—that is to say, it would be strained up to practically its ultimate strength, assuming it was constructed out of 26-ton steel, as we believe was the case.

Chord member No. 9 is longer than No. 8, and the load on it is greater, so that a lesser eccentricity would probably be fatal to it, were it ever called upon to take its intended maximum load. At the time of the disaster the stress here is said to have been only about 8 tons per square inch, and a greater eccentricity than above calculated might then be needed to endanger the bracing. At the best, however, the margin of safety appears very small, and the construction adopted is certainly unmechanical. In the river-arm the struts are still longer, and the bracing has here, accordingly, a still smaller margin of safety. It is to be presumed that the Commission which has been appointed to report on the failure will test some large models of these struts. It is to be hoped that such tests will not only include experiments on the models centrally loaded, but that the investigation will be extended to cover their behaviour under eccentrically applied thrusts. We are convinced in that case that the bracing will prove wholly inadequate to develop the full strength of the specimens. This view receives additional confirmation in Mr. Skinner's article, where he notes the circumstance that in the case of the struts wrecked by the fall the bracing gave way before the struts crippled.

A different hypothesis as to the immediate origin of the catastrophe is suggested by Mr. Skinner on page 393. He notes here that the lower chord members near the pier did not merely abut on the main pin, but were continuous across the latter, thus introducing the bugbear of so many American bridge engineers—viz., indeterminate stresses. To English ears the suggestion that a mechanically designed detail such as this was responsible for the accident sounds somewhat strange, since it is perfectly well known that riveted structures make by far the best bridges, in spite of a theoretic possibility of uncalculated secondary stresses. It is impossible, for instance, to calculate with accuracy the stresses arising in a common plate-girder, yet every railway engineer knows that such girders give less trouble under traffic than any other type of small-span metal bridge.

This insistence that all structures shall be statically determinate seems to be the special fad of one particular school of American bridge engineers. It was not, however, held by the late Mr. G. S. Morison; and Mr. Buck, another eminent American bridge engineer, has also erected a number of statically indeterminate bridges. The idea rests on a theoretical basis only, and has no support from the teachings of experience. We know of no case in which a structure has failed merely through being statically indeterminate, whilst the rigidity gained has great practical advantages; and where such structures have failed through defective material or overloading, the wreck has not been practically instantaneous, as at Quebec, but has given time for the majority of those endangered to get clear.

Further, it should be noted that in actual practice American pin bridges are no more free from secondary stresses than properly-designed riveted trusses. Bearing in mind the frictional resistances involved, it is inconceivable that the pins actually rotate in the eyes of the links at each change of load on the structure. In fact, if they did, serious wear might be expected, and in the absence of such rotation the bars coupled up are to all intents and purposes as rigidly connected as if riveted, and suffer from the same secondary stresses. As to the objection taken to rigidity of the joint between the chords across the main pin at Quebec, it would seem that it might be equally well raised against every other joint of the lower chord. Indeed, in the old days, pin-jointed compression chords were somewhat in favour, until experience proved

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decisively the superior advantages of constructing such chords in continuous lengths. Again, it seems well established that the strut which failed buckled sideways, and not vertically. If secondary stresses due to the rigidity of the joint past the main pin had anything to do with this, the member should have yielded in a vertical plane, and the contrary fact would *per se* suffice to confute the suggestion, were it not already sufficiently discounted by the teachings of experience.

NOTES.

The Proposed Trelleborg-Stockholm Express Railway.

ENGINEERING, some time ago, contained a paragraph about a proposed express train for international traffic between Trelleborg and Stockholm, to be run at a speed of from 100 to 125 miles an hour. The plan was originated by Professor Richert, and he asked that the direction, the cost, and the financial prospects of such an electric railway might be investigated and reported upon. The Department for State Railways has now reported upon the proposal, and the report is possessed of some interest. It is first pointed out that, with the present degree of development, a train can only be run at 100 to 125 miles an hour by electric traction. The only experiments which have so far been carried out bearing on this point are those which took place in the years 1901 and 1903, on the section Marienfelde-Zossen, close to Berlin, a distance of about 15 miles, under the auspices of the "Studiengesellschaft für Elektrische Schnellbahnen," which company is subsidised by the Prussian State. During the first year the speed attained here was close upon 100 miles; and the second year, when the line had been strengthened, a speed of 130 miles an hour was reached, though only over a very short distance. It was ascertained that, in order to run a train safely at this tremendous speed, a singularly solid and expensive line was necessary; in addition to which the line, above all, must be a straight one; the minimum permissible curve radius being 2000 metres. Level crossings are, of course, entirely out of the question. It is evident that a railway of this description is bound to entail a very heavy cost, in addition to which it must necessarily be a double line, as the speed otherwise will be too much interfered with. It must also be borne in mind that a railway of this description cannot with advantage be used for all kinds of traffic. For all these reasons it has not—even in Germany, with her huge population and many big cities—been thought possible to make such expensive railways pay their way, however desirable a great speed may be considered. The Swedish Railway Department is quite alive to the importance it would have for Sweden to transform part of the country passenger traffic, in accordance with Professor Richert's plan; but in view of the German experiments, the Board sees no chance of paying interest of the tremendous cost such railways would entail, for an indefinite future.

U-Leather Friction in Hydraulic Presses.

In referring to the work done at the Materialprüfungsamt, near Berlin, we have mentioned that Professor Martens makes a wide use of his hydraulic diaphragm testing-machines, which have proved very convenient for many measurements, and the reliability of which has been established by many series of tests. Professor Martens now finds that the ordinary hydraulic press may, in combination with a pressure-gauge, well be employed for testing purposes, and that with due care an accuracy of ± 5 per cent., and even ± 2.5 per cent., may easily be secured without having recourse to special appliances for reducing the friction. The point that required investigation was the friction of the U-leathers. For this purpose he connected three Pohlmeyer and two Martens testing-machines with hydraulic presses. The pistons of the presses for the latter machines were fitted with tail-rods, and required two U-leathers; the pistons of the presses for the Pohlmeyer machines had only one gland and only one leather. The leathers were put in very stiff, lest a slight leakage of water should occur. The indications of the testing-machines in question were checked by direct weights. It was found that the friction during the no-load tests might rise to 16 per cent. in the case of the Pohlmeyer combinations, where one U-leather was used, and to 43 per cent. in the case of the Martens combinations, where two leathers were applied. These values are very high. But nothing had been done to keep the friction low, and the figures were concordant. In the actual tests, which could then be undertaken, moreover, the friction percentage almost vanished. With pressures of 50 atmospheres, the total friction percentage was diminished to 5 per cent., and when high hydraulic pressures were applied, the loss due to friction did not exceed 1 per cent. The deviations from the means did not exceed ± 0.5 atmosphere in all the cases, although the frictions of the several machines differed considerably. It would thus appear that hydraulic presses may be used for practical testing in the workshop and on the spot, where testing-machines are out of the question. From this point of view the research deserves attention; more testing would be done if convenient devices were available. As long as the leathers are not replaced, changes need not be feared. Faults in the piston would soon be recognised. Of all the parts of the machinery, the pressure-gauge is, perhaps, most likely to go wrong; this trouble may be obviated by the use of two pressure-gauges. Professor Martens has communicated a preliminary account of these researches to the *Verein Deutscher Ingenieure*; the full particulars will be published in the *Mitteilungen über Forschungsarbeiten*.

The Viscosity of Mixtures of Oil and Water.

As the viscosity of oil is very much greater than that of water, we should expect that any contamination of water with oil should noticeably raise the viscosity of the water. The subject has been little investigated. But according to experiments described by K. Beck, of Leipzig, in the *Zeitschrift für Physikalische Chemie*, vol. lviii., the viscosity of water is scarcely increased when it is stirred with as much as 10 per cent. of castor oil. From the behaviour of colloids it is, on the other hand, evident that the viscosity of suspensions is much greater than that of the clear liquid, and this is confirmed by tests on the electrical conductivity of electrolytes, in which inert particles, like kieselguhr, are suspended; the resistance is augmented by the addition of these particles, which probably impede the ions in their migration. It would thus appear that the particles must be very near one another, and very small, if they are to affect the viscosity. Beck has conducted his experiments, in conjunction with K. Ebbinghaus, on castor oil, which possesses a high viscosity. Taking the viscosity constant of water as unit, that of castor oil is 1583 at 8 deg. Cent., 361 at 25 deg. Cent., and 124 at 40.5 deg. Cent. For his investigation Beck made use of Ostwald tubes, determining the number of seconds which a certain volume of the liquid took in passing through the tube. He prepared his suspension, first, by shaking the oil for half-an-hour with the water by machinery. He found it necessary to add a little cholesterol to prevent rapid settling. But even with suspensions containing 10 per cent. of oil, the rate of flow was hardly influenced, the time rising, *e.g.*, from 152 seconds for pure water to 154 seconds for the mixture. More concentrated mixtures could not be experimented with, as they did not remain homogeneous. Beck, therefore, prepared emulsions in the way the pharmaceutical chemist does, by rubbing the oil and some gum arabic together in a mortar, and then adding the water. The addition of the gum was indispensable; but the gum itself increased the viscosity, of course, and this effect had to be allowed for. In this manner he found that 5 parts of oil, 4 parts of gum, and 91 parts of water gave a viscosity constant of 2.72, while the gum alone gave 2.6. With 10 parts of oil to 4 parts of gum the constant rose to 3.2, this value being 20 per cent. higher than the constant of the gum emulsion alone; with 20 parts of oil to 8 parts of gum and 72 parts of water, the constant was 12, being 40 per cent. higher than the viscosity increase due to the gum. With higher percentages of oil, the experiments became too difficult. So far as the experiments go, the effect of the addition of oil to water on the internal friction would therefore appear to be small. Beck also made some experiments on alcohol and chloroform, we may add, with another object. When liquids of low viscosity are discharged through long capillary tubes, the calculation by Poiseuille's law gives correct results for the internal friction only when the liquid issues at a slow rate. In order to ensure this condition, a counter-pressure has to be applied; that is to say, the liquid is made to emerge under a certain head of water. Beck has studied these counter-pressures for tubes of different diameters.

Financial Conditions in Japan.

The developments which are taking place in Japan are naturally attracting attention in all countries in the world, and the fear is sometimes expressed that she is spending more than her resources justify, and, therefore, that there may soon be a crisis which will set things back. Mr. Wakatsuki, the Imperial Japanese Special Commissioner in London, and who holds the position of Vice-Minister of Finance in Japan, has made a statement in reply to the criticisms of Japanese finance which have been made in various quarters. It has been often said that there is a deficit of 18,000,000*l.* in the Budget, this being based on the fact that the regular income only amounts to 42,000,000*l.*, while the expenditure is 60,000,000*l.*, and it is suggested that Japan might experience a difficulty in meeting this deficit. The fact is, however, that this sum of 42,000,000*l.* is only the ordinary part of the whole revenue, while the sum of 60,000,000*l.* as expenditure represents the total expenditure, including both the ordinary and the extraordinary. The ordinary revenue, which, to be more exact, is about 43,000,000*l.*, is more than enough to cover the ordinary expenditure, which totals about 42,000,000*l.* The extraordinary revenue is only a trifle less than suffices to meet the extraordinary expenditure. It is thus clear that the whole revenue, inclusive of both ordinary and extraordinary, just balances the whole expenditure. The extraordinary part of the revenue cannot be left out of consideration merely on account of its being of a temporary nature, for the extraordinary part of the expenditure is also temporary, and will cease in the course of a few years. There is no reason to apprehend a deficit even in the future. In the case of the present year the Japanese Government has provided an extraordinary revenue to meet the extraordinary expenditure. For next year, and for succeeding years too, it has a plan of supply for the same purpose. The Budget scheme must necessarily shape itself according to the circumstances it has to meet. Another misconception consists in the supposition that an appreciable part of the Japanese expenditure is caused by Government subsidies and grants to industries that are directly competing with British trades. Many of these subsidies are of the nature of a State contribution to municipalities for water works, sewage construction, harbour improvements, and so forth, all of which rather tend to encourage British trade. Subsidies to shipping seem to have specially excited uneasiness, but the policy of encouraging shipping was in operation long before the war, and it was only natural that an island Empire like Japan should develop its means of communication. Even these subsidies are going to be withdrawn, for the Government has repeatedly told the Imperial Diet that its policy lies in the direction of reducing subsidies year by year. Even the apparent increase in the expenditure on the army and navy is of a temporary nature, and it is caused by the decision to repair the waste caused by the war with Russia. Mr. Wakatsuki says it is a gross mistake to suppose that Japan has adopted an aggressive policy after the war, and for this purpose has suddenly increased the extraordinary expenditure for the army and navy.

Turpentine from Waste Wood.

According to a paper recently read by Dr. J. E. Teeple before the Boston (United States) branch of the Society of Chemical Industry, the turpentine distillation business does not appear to be in a satisfactory condition. The name turpentine is believed to be of Persian origin, and was first applied to the resinous exudation from trees of the sumac family. The commercial spirits and resin of turpentine are, however, all derived from certain pines occurring in the Southern United States, Russia, and South-Western France. The old American practice was to "hack" the trees every week during the summer months, and to collect the resin running out, which represents partly the natural sap of the tree and partly a pathological exudation of the wounded tree. As the hacking is continued for years, the 10 ft. or 15 ft. of the lower part of the tree which are thus treated become more and more charged with the pathological product, which collects in ducts, and which is less rich in spirits and richer in resin than the sap. Such wood is

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side of the valve. The countershaft at the base has fast-and-loose pulleys, and the belt is shifted by the operator's foot, and it drives both the spindle and a crank-disc by which the intermittent trip motion is actuated. When a cylinder is placed on the table the attendant engages the screw-driver end in the valve top, and starts the spindle running. The result is a continuous rotary grinding motion, with periodical lifts of the valve, to prevent scoring, just as is done in hand-grinding. The wooden handle mentioned enables the operator to give the requisite pressure. The countershaft runs at 350 revolutions per minute.

We must defer till another week a fuller notice of the exhibits of Alfred Herbert, Limited, whose stand is a veritable model workshop of turret, milling, and other machine-tools. At present we may note, however, two interesting gear-hobbing machines—namely, Wallwork's patent gear-hobbing and worm-wheel generating machines. The latter is illustrated in Fig. 13, page 428. Its chief feature is, that instead of using a hob, single cutters are employed gripped in a bar. The section of the cutters is that of an involute rack-tooth with 71 deg. of angle, and a single set of cutters will generate wheels of all numbers of teeth of the same pitch. Their setting in the bar can be varied to suit the angle of thread of different worm wheels. The cutter-bar and the vertical mandrel on which the blank is carried are rotated by the two sets of change-wheels at opposite ends of the machine. The driving-cone has five steps, and is back geared; those with a two-speed counter give twenty speeds.

THE QUEBEC BRIDGE.*

By FRANK W. SKINNER, M. Am. Soc. C.E.

(*Continued from page* 391.)

DETAILS OF MEMBERS AND CONNECTIONS.

Wind Anchorage.—Wind anchorage is provided at the shore ends of the anchor-arms and at the main piers. At the piers it is taken directly through fixed connections to the pedestals, bolsters, and masonry. At the shore it is taken through expansion connections to a massive transverse girder connecting the tops of the tower-posts, and through them to the anchor-pier masonry. At the anchor-piers provision is made for a longitudinal motion of 8 in. of the superstructure on the substructure, and very novel and ingenious provisions have been made to allow for this and maintain constant transverse bearing for the heavy lateral stresses, and transmit them continuously to the pier masonry.

The horizontal stresses of the top, bottom, and floor system laterals are combined at the floor-level of the shore ends of the anchor-arm spans, where they are received at the ends of the floor-beam, which is proportioned for a maximum transverse horizontal stress of 1,010,000 lb. in either direction. A large vertical tenon (see Fig. 73, annexed) projects below the bottom flange of the floor-beam at its centre point, and engages with the top strut of the fixed tower-bent, where it has a roller bearing, maintaining contact whatever the relative displacement between the two members, and thus permitting longitudinal motion and preventing any displacement vertically or transversely. The transverse horizontal stress is transmitted through the top strut to the vertical and diagonal members of the tower bent, developing in them maximum stresses of 596,500 lb. and 783,500 lb. respectively, which are absorbed by the masonry.

The end floor-beam is a rectangular box-like member, about 10 ft. deep, with two $\frac{7}{16}$-in. webs 33 in. apart, and four pairs of 8-in. by 6-in. by ¾-in. flange angles with 13-in. by ⅝-in. cover-plates. The top and bottom flanges are connected by shop-riveted batten-plates and lattice-bars and by short 36-in. by ½-in. cover-plates, one of which, in the centre of the top flange, is field-riveted to the end of the vertical tenon girder, 15½ ft. long, with its 5½-ft. solid web parallel to the double webs of the end floor-beam, and on the centre line between them. The flanges of this girder take bearing on the faced reinforced ends of cover-plates on the lower flanges of the double floor-beam, and are also bolted to connections with vertical transverse diaphragms connecting the floor-beam webs, both sides of the tenon-girder.

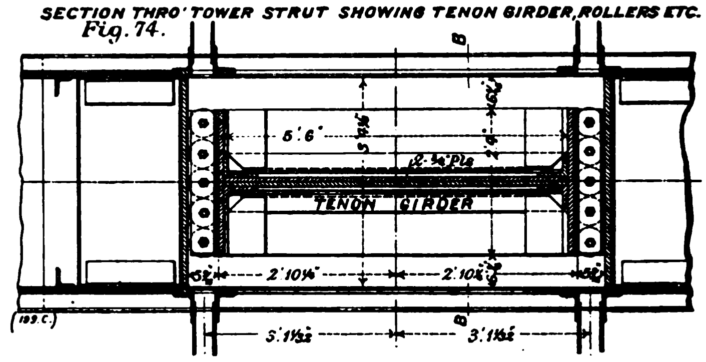
This arrangement really combines the end floor-beam and tenon-girder in a single rigid T-shaped girder, with the vertical member engaging between the double webs of the top strut of the tower. This is a box-girder, in the same vertical plane as the floor-beam, and has two 60-in. by ½-in. webs, four 7-in. by 3½-in. flange angles, and two 49-in. by ½-in. flange cover-plates, cut to clear the tenon-girder. Each side of the tenon-girder the webs are connected by five horizontal diaphragms, with the centre ends faced to bearing on vertical transverse plates parallel with the bridge axis. These have horizontal guide-tongues planed on them to correspond with tongues on the flange cover-plates of the tenon-girder. Both sets of tongues engage grooves in nests of vertical rollers (see Figs. 74 to 76, annexed) forming bearings between the tenon-girder and strut. The flanges of the tenon-girder are extended to make a bearing 28 in. wide, reinforced by six pairs of horizontal diaphragms on the tenon-girder web. The rollers are free to move back and forth, parallel to the bridge axis, with temperature expansion and contraction of the trusses; and in order to prevent cumulative displacement by successive movements in the same direction, they are provided with an automatic attachment to maintain them constantly centered with the strut when not in actual operation under wind stress. Counterweights suspended from the tower strut exert constant horizontal pressure in opposite directions on each side of the roller-nests, forcing the latter into a balanced position on the centre line of the strut as soon as the wind pressure is relaxed.

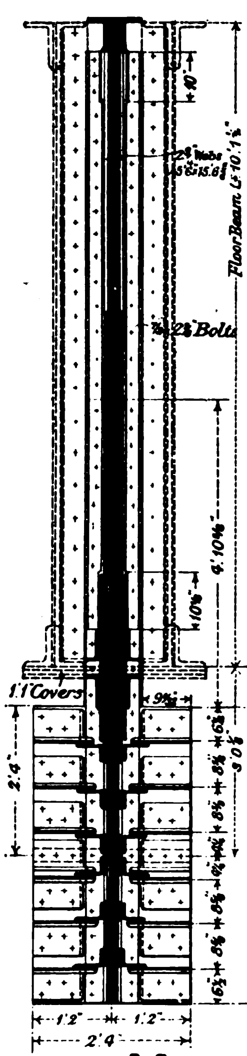
The Floor System.—The floor platform, about 120 ft. above the top of the pier masonry, is 72 ft. wide over all, and slopes up from both anchor-piers to the centre of the bridge on a 1 per cent. grade. It provides in the centre for two railroad tracks, enclosed by vertical screens about 7½ ft. high, outside of which, on both sides, there is a floor of 4-in. diagonal planks on wooden transverse joists. In each roadway there is a trolley track, and on each side of it a 5 ft. side-walk extending through the trusses and protected on the outside by a heavy lattice-girder hand-rail 4 ft. high. There are twelve lines of plate-girder stringers, which in the anchor-arms are uniformly 50 ft. long, and web-connected to the floor-beams, with their top flanges just clearing those of the floor-beams. They have depths of 68 in. and 51 in. under the railroad tracks and under the roadway respectively, and are all braced with vertical transverse sway-frames and top and bottom flange zigzag angles.

At the shore ends of the anchor-arms, where the bottom chords are at floor-level, their bottom flanges are connected to those of the floor-beams by wide lateral plates, and the floor-beams are notched to clear the chords, and are web-connected in the usual way to the vertical posts. Typical floor-beams of this kind have 120-in. by $\frac{7}{16}$-in. shop-spliced webs and flanges made with four 8-in. by 6-in. by $\frac{11}{16}$-in. full-length angles, and four 22-in. cover-plates of varying lengths, and a combined thickness of 2⅞ in., to provide for the maximum bending moment of 9,500,000 foot-pounds. The ends are reinforced to a total thickness of 4½ in. to resist the maximum shear of 402,000 lb.

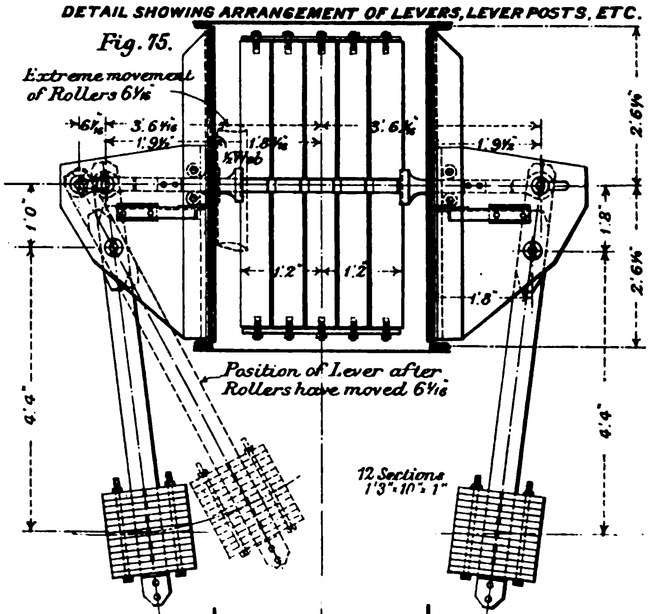
The track-stringers have web and flange connection angles to the floor-beams, and those under

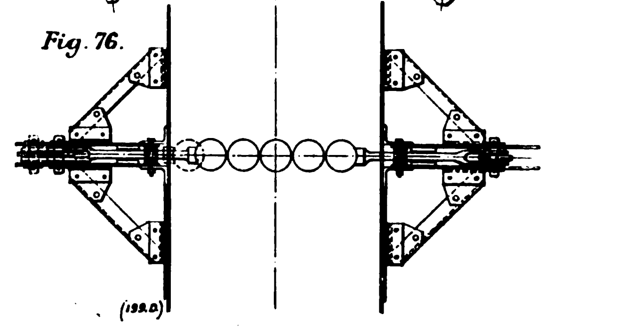
SECTION THRO' TOWER STRUT SHOWING TENON GIRDER, ROLLERS ETC.





DETAIL SHOWING ARRANGEMENT OF LEVERS, LEVER POSTS, ETC.





* [This article was, of course, written prior to the occurrence of the grave disaster with which we had occasion to deal in our last two issues. As a complete account of a most important work, however, it has lost none of its interest, and a study of the information which it contains will add much to the understanding of the inquiry now being made into the causes of the failure.—Ed. E.]

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the railroad tracks have thirty field-rivets in each connection. There are 106 field-rivets in six vertical rows in the end connection of each floor-beam, the holes for which are reamed through thick iron templates. Each floor-beam weighs about 30 tons, and was finished complete at the shop and handled in erection by swivelling attachments which enabled it to be revolved horizontally 90 deg. while suspended from the traveller tackles.

Where the bottom chord is just clear of the floor-beam, the latter has a full depth connection to the vertical posts, with four vertical rows of field-rivets through the flanges of a pair of 6-in. by 6-in. by 3-in. connection angles. Here the relative position of the stringers and floor-beam changes, and the latter are seated on the top flanges of the former.

On both sides of the main piers, where the bottom chords are far below the floor-level, the floor-beams are virtually triangular trusses with their bottom chords formed by the special sway-brace diagonals. The top chord is a box girder with two 48-in. by $\frac{7}{16}$-in. full-length webs, four 6-in. by 4-in. by $\frac{1}{2}$-in. flange-angles, one 26-in. by $\frac{1}{2}$-in. top flange, and two 7-in. by $\frac{1}{2}$-in. bottom flange full-length cover-plates. Connections to the vertical posts and hangers are made with field-rivets through the ends of the web-plates and the post flanges and through pairs of vertical connection-angles, $6\frac{1}{2}$ ft. long, on the faces of the posts. The lower chords of the floor-beam trusses have I-shape cross-sections made with one $14\frac{1}{2}$-in. by $\frac{3}{8}$-in. web-plate, two pairs of 7-in. by $3\frac{1}{2}$-in. by $\frac{5}{16}$-in. angles, and two 18-in. by $\frac{7}{16}$-in. flange cover-plates, and are connected to the top chord by pairs of 66 in. by $\frac{1}{2}$-in. jaw-plates riveted to the webs of the latter, and projecting below their lower flanges.

The Anchor-Pier Tower.—The dead-load moments of the trusses are such that under all conditions of loading there is an uplift at the shore end of the anchor-arm which reaches a maximum of 1,959,000 lb. for each truss, and is provided for by a set of six short forged links connecting the bottom chord end pin with a chain of twenty 10-in. by $1\frac{11}{16}$-in. eye-bars, two panels of about 50 ft. long, with a total cross-sectional area of 337 5 square inches. The upper panels, about 42 ft. long, are free to oscillate about their lower pin, above the top of the pier masonry; but the lower set are built and concreted solid into the masonry, where they are rigidly fixed and connected at their lower ends to the reaction platform built into the foot of the pier (see page 329 *ante*).

The arc of the circle through which the upper end of the eye-bars swings is so flat that it is virtually horizontal, and thus provides for longitudinal movement of several inches without any material vertical or transverse displacement. The tower posts under the end-panel points can therefore receive no compressive stress from the weight of the bridge or its load, and although they serve as cases for the anchorage bars, care is taken to prevent them from receiving any of their stress, and they act structurally only to help to transmit the horizontal stresses from the superstructure to the substructure.

Pier-Towers.—Each pier-tower is a simple transverse bent consisting of two vertical posts, a top and a bottom horizontal transverse strut, and X-braces between them. The vertical posts, about $84\frac{1}{2}$ ft. high, have a maximum stress of 666,000 lb. each, and have an open rectangular cross-section, made with two solid webs, parallel to the bridge axis, latticed on their flanges. The webs are made throughout of $\frac{7}{16}$-in. plates, and taper from 4 ft. to 17 ft. in width, and from $6\frac{1}{2}$ ft. to $9\frac{1}{2}$ ft. apart, top and bottom, thus forming opposite faces of a truncated pyramid. Each web has a pair of 7-in. by 5-in. flange-angles on each edge, giving it an I-shaped cross-section, and has besides two more intermediate pairs of vertical angles of the same size, 38 in. apart over all, with a separate $\frac{1}{2}$-in. section of web between them.

The webs are stiffened with 4-in. by 3-in. horizontal angles, about $3\frac{1}{2}$ ft. apart, and were shop-riveted in convenient lengths for shipment. The webs are connected by horizontal angle iron frames, in two panels each, with 3-ft. clearances between them for the anchorage eye-bars. The top of each post has a horizontal cap-plate, slotted to receive the eye-bars, and the bottom is stiffened by two vertical diaphragms, about $7\frac{1}{2}$ ft. high and 38 in. apart, that are transverse to the bridge axis, and afford connections for three longitudinal diaphragms with intermediate bearings for the 12-in. eye-bar pin, which also engages the outside post-webs at the same points where are connected to it the tower diagonals, with a maximum stress of 783,500 lb. in each. This stress, with the vertical wind stress, is thus transmitted through the lower panel of eye-bars to the reaction platform independently of the upper panel of eye-bars, which have a lower pin, that, although passing through the post-webs, has so much clearance there that it cannot take bearing on them.

The foot of each post takes a bearing on the pier masonry with a hollow rectangular base, having plates under the post-webs and an open centre space. The base is connected to the post by bent plates, with their horizontal and vertical flanges united by a curve of $3\frac{1}{4}$ in. radius, with sufficient elasticity to provide for a vertical movement of about $\frac{1}{4}$ in., to correspond with the elongation of the anchorage bars between minimum and maximum stresses. The horizontal and diagonal tower-braces have closed rectangular cross-sections, and have gusset-plate connections to the posts.

Cantilevers.—The design of the cantilever-arm trusses is substantially the same as that already described for the anchor-arm trusses, the members are similar, and the details and connections nearly like them except at the river ends of the top and bottom chords, and the two together comprise most of the novel and interesting features of design and erection for the entire bridge, which are fully exemplified by the completion, in the fall of 1906, of the erection of both anchor and cantilever arms on the south side of the river.

The 675-ft. centre span weighs about 6,000,000 lb. exclusive of floor, and has trusses 98 ft. deep at the ends and 130 ft. deep at the centre. It is much longer than any free span before built, and its design corresponds to advanced American practice for long-span railroad bridges. The end lower chord-pins, 12 in. in diameter, engage slotted holes, providing for temperature movements.

Fabrication of Members.—All of the steel-work has been completely finished in the shops of the Phœnix Bridge Company, about 30 miles from Philadelphia, Pa., which have a yearly capacity of 40,000 tons of railroad bridges, and form part of a large steel plant, owning and operating iron and coal-mines, rolling-mills, blast-furnaces and steel furnaces, and having an annual output of 150,000 tons. The bridge-shops cover an area of about 125,000 square feet, besides 375,000 square feet of yards, and employ an average force of 800 men, who turned out as much as 3000 tons of Quebec steel-work, besides 2000 tons of other regular bridge-work per month.

All of the Quebec bridge-work, including the heaviest members, was manufactured and handled by the regular plant and machine-tools previously installed for general contracts, augmented only by a very few items needed in the regular equipment, and including one double rotary planer with 64-in. heads, and a 10-ft. by 10-ft. by 25-ft. horizontal planer. So many of the Quebec Bridge members had to be stored at the shops that the shipping yard was extended and equipped with an additional 75-ton girder crane of 90-ft. span.

The clearances and dimensions of all members and connections are so thoroughly checked, and the accuracy of the drawings and shop-work is so great, that, as in other first-class American bridge shops, the erection of trusses or fitting together of members or assembling of connections is never done there, different members of the trusses being, in fact, often shipped to the site directly from plants hundreds of miles apart. The Quebec Bridge was no exception to this rule; but, on account of its magnitude, the complication of some of the connections and the large number of similar connections, a full-sized wooden model, about 20 ft. high, of the connections at one lower chord panel-point, showing every separate piece and rivet with exact dimensions and clearances, was built at a cost of about 500 dols., and provided a very interesting and graphic demonstration of the magnitude and general character of the work, which proved very interesting to visitors, and valuable to illustrate the relation of parts to the workmen.

Compression Members.—The bottom chords and many of the vertical and inclined posts have four webs built up with several thicknesses of wide plates and heavy flange angles, connected by diaphragms, and unusual dimensions, attaining a maximum of 5 ft. width in the seven sections of the 5-ft. by 10-ft. 350-ton centre vertical posts, which are made in seven sections each, the heaviest section weighing 112 tons.

The different sets of web-plates and flange-angles for a member are assembled separately, reamed and riveted, and are assembled together with special provision to keep them straight, parallel, and out of wind, spacing blocks being fitted between them and secured by heavy clamps and diagonal hook-rods adjusted by turnbuckles. Most of the rivets are driven by movable pneumatic machines of the yoke type, but some of them are driven by pneumatic hammers. Great care is taken to secure accuracy in all holes for field-rivets, which are drilled through wrought-iron templates 2 in. thick, numbered and marked for their respective positions, and adjusted to line and position and tightly clamped—precautions which have resulted in perfect holes when the members and splices are assembled for the first time in the field, enabling them to be fitted up quickly and easily without reaming.

The very wide multiple webs or diaphragms for the top chord pins and intermediate pins have such large dimensions that special precautions are necessary to locate and check the pin centres, which are laid out on both sides of the member. Discs an inch or two smaller than the pin-holes are cut out of the separate plates by small punches, and the holes are counterbored in vertical machines in which the members are levelled, aligned by a transit, clamped to the tables, and bored to $\frac{3}{64}$ in. larger than the pins. The inspector measures to pin centres by the aid of 2-in. discs provided with three set-screws, enabling them to be set quickly and accurately in the centre of the holes. Up to 10-ft. distances between pin-holes are measured by a trammel; longer distances are measured by standardised steel tapes under 12-lb. spring tension, and maintained horizontal on wooden supports 4 ft. apart when necessary. Pin centre distances are thus checked to $\frac{1}{64}$ in., and the diameters of the pin-holes are tested by cast-iron gauges 6 ft. long. In the bridge-shops 100-ton travelling-cranes load the finished members on standard gauge flat cars that deliver them to 100-ton 90-ft. travelling-cranes on 400-ft. storage-yard tracks, between which they are piled for shipment, as required at the site. All members are painted as soon as finished, and, if necessary, are re-painted in the storage-yards.

Eye-Bars.—There are in all about 3300 eye-bars up to 16 in. by $2\frac{1}{4}$ in. by 76 ft. long, with heads 36 in. wide, having an excess of 40 to 45 per cent. of metal over the body, and bored for 12 in. and 14-in. pins. They are proportioned for a working unit stress of about 22,000 lb., and are made from open-hearth steel of an average tensile strength of about 63,000 lb. per square inch, as demonstrated by tests of three tension and bending specimens cut from bars in each heat. After careful surface inspection the bars are upset in three heats by an hydraulic machine, and die-forged in three more heats under a 12,000-lb. steam-hammer at the rate of 24 heads in 10 hours. The heads are hammered to a black heat and finished with great uniformity to a thickness which does not vary more than $\frac{1}{16}$ in. from that required.

The lateral dimensions of the heads are not allowed to vary more than $\frac{1}{8}$ in. from the required dimensions, and are checked by right-angle and diagonal measurements, and tested for dishing by a straight-edge. The pin-holes are hot-punched under the steam-hammer to reduced diameter. The bars are annealed 48 hours in a gas-furnace, heated at first to cherry-red, and allowed to cool in their own atmosphere. They are then straightened and supported continuously in a horizontal position, and the centres of the pin-holes located by standardised 100-ft. steel tapes under uniform 12-lb. tension. Care is taken to set them in the vertical boring-machine in exactly the same relative position as in the bridge, so as to bore the hole at the same angle as the pin will have, on account of the slight divergence of bars in packing, and the hole is completed, counterbored with two rough cuts and one finishing cut. The holes are bored $\frac{3}{64}$ in. larger than the diameter of the pins, and tested by a cast-iron disc gauge that fits tightly, but will drop through if the hole is $\frac{1}{1000}$ in. too large. The pins will drop through the holes in both ends simultaneously when all the eye-bars in the same panel are piled up together horizontally.

(*To be continued.*)

COAL IN BELGIUM.—An adjudication has just taken place of locomotive coal required for the Belgian State Railways. Some English tenders were received, but the English firms required 10s. 2d. per ton delivered at Antwerp, while Belgian offers were made at 8s. 4d. per ton. The British tenders were considered inadmissible.

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one direction is for going ahead, the reverse for moving backward, whilst the mid position throws open a by-pass and affords a free-wheel motion; when moving ahead, a slight backward drive has the effect of a brake. This hydraulic drive is silent, smooth, and the whole apparatus is said to compare favourably, in respect of weight and efficiency, with the systems of mechanical gearing which it is designed to replace. The address of the Pittler Rotary Machine Syndicate is 81, High Holborn, London.

NOTES FROM THE UNITED STATES.

PHILADELPHIA, September 17.

THE past week has developed further weakness in the demand for iron and steel. In a few localities there is a partial shutting down. A number of furnaces in the Shenango Valley will blow out because of the exhaustion of the linings. The production of iron ore has been curtailed by lack of labour and by storms, which interfered with the operation of steam-shovels. Foundry irons have been much inquired for this week, but prices are not low enough to suit the views of buyers. Consumers are looking for a further and a decided drop, when it is expected buyers will make large purchases. Bessemer and basic pig are on active demand, with very little sign of weakness. One influence that will operate against lower pig-iron prices next year will be high-priced ore. Vigorous efforts are being made to ascertain the locality of hidden deposits, and encouraging developments are in progress. Pig-iron production has latterly been increasing faster than ore output. Another influence is the blowing out of furnaces for repairing, but this is temporary. Some large consumers of pig iron are considering the wisdom of heavy buying for 1908. Others are not taking account of remote conditions. The bottom conditions at present are of a financial nature. There is a general slowing down in all activities, and the tendency is for further restriction of enterprise. But there are factors which cannot be controlled by wisdom, such as unexpected demand and unavoidable expansion of requirements.

The coke situation is satisfactory in the sense that the larger consumers have their requirements covered by contract for six months to come. Foundry coke is cleared up, and in the absence of stocks prices are strong. Tinplate continues active in all markets, but the larger consumers are not buying beyond pressing necessities.

The bar and sheet mills are working to full capacity this week, and stocks are light. It is the intention to accumulate supplies wherever possible. The merchant steel and merchant pipe mills are all selling their output promptly, and the pressure for boiler tubes is equal to the maximum output. The pivotal point in the American iron and steel industry is the magnitude of the demand that may be created by the new enterprises to be launched during the next six or nine months.

NORTHAMPTON POLYTECHNIC INSTITUTE.—The winter session of the Northampton Polytechnic Institute, St. John-street, E.C., has just opened, day and evening classes being arranged for in mechanical and electrical engineering, horology, technical optics, artistic crafts, &c., while additional evening courses are given in certain other subjects. Several of the courses have been remodelled so as to bring them into conformity with the revised syllabus recently issued by the City and Guilds of London Institute. In the engineering day courses fresh subjects have been introduced for advanced work of the third and fourth year students, and in other departments the sphere of work has been considerably enlarged. In technical chemistry the course covers the study of rare earths, their production, extraction, &c., in connection with incandescent lighting, both by electricity and gas. The work is throughout of the practical nature which characterises the efforts of this Institution, and as the fees are extremely small, it is to be hoped that its enterprise will meet with due encouragement and support.

MINING MACHINERY.—The exports of mining machinery from the United Kingdom have shown considerable languor this year. The amount of these exports in August was only 2324 tons, as compared with 1282 tons in August, 1906, and 2289 tons in August, 1905. In these totals South Africa figured for 475 tons, 426 tons, and 599 tons respectively. In the eight months ending August 31 mining machinery was exported to an aggregate extent of 16,054 tons, as compared with 13,985 tons in the corresponding eight months of 1906, and 18,909 tons in the corresponding eight months of 1905. The colonial demand was represented in these totals by the following amounts:—

Colonial Group.	1907.	1906.	1905.
	tons	tons	tons
British South Africa ..	3972	4070	5353
British India	1180	1191	696
Australasia	1740	1824	2319

The value of the mining machinery exported to August 31 this year was 568,020*l.*, as compared with 473,201*l.* in the corresponding period of 1906, and 552,389*l.* in the corresponding period of 1905.

THE QUEBEC BRIDGE FAILURE.

TO THE EDITOR OF ENGINEERING.

SIR,—Commenting on the Quebec Bridge disaster, you draw, with good reason, the attention of your readers to the dangerously light bracing of some compression members. As early as 1893 I pointed out the crude and unsatisfactory methods obtaining for the design of such bracings; and, in a pamphlet entitled "Les Treillis Raidisseurs des Pièces Chargées debout," I proposed a very simple formula, which has since extensively been used here. Afterwards I modified it slightly, in consequence of the results obtained by the late Professor Tetmajer in his world-known experiments. It may be demonstrated as follows:—

If all things could be perfect, a centrally loaded strut would not bend at all; but, in fact, some flexure always occurs owing to several possible causes of eccentricity; hence the necessity of determining the practical ultimate strength of a given strut and of giving it but a fraction of the corresponding load. Implicitly we admit, of course, that by using this factor of safety against collapse, the unit stress will not be higher than the safe compressive stress; otherwise the structure could not be considered as safe.

Now, let P be the safe load, Ω the area of the section, r the radius of gyration, a the distance between the edge of the strut and its axis, R′ the safe compressive stress, e the initial eccentricity, and f the greatest deformation caused by flexure; we must have for safety

$$\frac{P}{\Omega}\left\{1+\frac{(f+e)\,a}{r^2}\right\} \leq R',$$

or

$$f+e \leq \left(\frac{R'\,\Omega}{P}-1\right)\frac{r^2}{a}.$$

In fact, the eccentricity e is totally unknown, and all practical formulæ for struts suppose it to be a vanishing quantity; if such be admitted, we get

$$f \leq \left(\frac{R'\,\Omega}{P}-1\right)\frac{r^2}{a}.$$

On the other hand, if e is exceedingly small, we know that the elastic curve of the strut, freely hinged at both ends, will tend towards a sinusoid

$$y = f \sin \pi \frac{x}{L},$$

the origin being placed at one extremity, and L being the full length of the piece. Thence

$$M = P\,y,$$

$$S = \frac{d\,M}{d\,x} = P\frac{d\,y}{d\,x} = \pi P \frac{f}{L} \cos \pi \frac{x}{L}.$$

If we substitute for f its limit, we get

$$S = \frac{\pi\, r^2}{a\,L}(R'\,\Omega - P) \cos \pi \frac{x}{L}.$$

The maximum takes place at both ends, and

$$S_{max.} = \frac{\pi\, r^2}{a\,L}(R'\,\Omega - P).$$

This is the greatest shear to be feared in a well-designed and not over-strained strut, and if the bracing is safe against it, it will have the same factor of safety as the ribs themselves.

The formula can be used as it is, but it may be much simplified if a straight-line formula be used for calculating the strut. I take always, for all iron and steel compression members—too short for permitting the use of Euler's formula—

$$P = R'\,\Omega\left(1 - \alpha\frac{L}{r}\right),$$

the coefficient α being = 0.0043 for iron and = 0.0037 for steel (accordingly to Tetmajer's experiments). If we put this value of P in the above formula, we get

$$S_{max.} = R'\,\Omega\,\alpha\,\pi\frac{r}{a},$$

which is generally the practical formula.

Let us apply it to the strut which collapsed at the Quebec Bridge; here a = 33.75 in., the other elements can be found on page 402 of ENGINEERING.

$$S_{max.} = 13 \times 756.6 \times 0.0037 \times \frac{19.5}{33.75} = 66 \text{ tons.}$$

As the production of such a shear must be admitted as a possibility, it becomes self-evident that the bracing must be regarded as most unduly light, as you stated.

It may be remarked by the way that the stresses considered as safe by American bridge designers seem to be recklessly high. Certainly no sane Continental—and, I fancy, English—engineer, would think of loading the strut in question—even if properly braced—with anything more than 5500 tons. It costs, of course, money to be so cautious, but is it really wasted? If one of the poor fellows who lost their lives in the disaster could answer, his reply could easily be anticipated.

Hoping my "Continental" English will not offend too much the readers of ENGINEERING,

I am, Sir, yours truly,
F. KEELHOFF,
Professor at Ghent University.

Ghent, September 24, 1907.

TO THE EDITOR OF ENGINEERING.

SIR,—In his "General Specifications for Steel Railroad Bridges and Viaducts" (new and revised edition, 1906), Mr. Theodore Cooper lays down the following rules for proportioning compression members constructed of medium steel.

$$P = 10{,}000 - 45\frac{l}{r} \text{ for live load;}$$

$$P = 20{,}000 - 90\frac{l}{r} \text{ for dead load;}$$

in which P is the allowable compressive stress per square inch on the section, and l and r are respectively the lengths of the chord and least radius of gyration expressed in the same units.

These formulæ are essentially different in form from others with which we are familiar; but if the chord which failed in the bridge had been proportioned in accordance with these rules, it would appear from the following figures that it would now be safely bearing the load. Taking 55 ft. as l, and 20 in. as the assumed least radius of gyration, l/r = 33, and therefore we have

P (for live load) = 10,000 − 45 × 33 = 8,515 lb. per sq. in.
P (for dead load) = 20,000 − 90 × 33 = 17,030 ,, ,,

The total calculated compressive stress on the chord with the wind contributing is given on the strain sheets as 22,638,000 lb. With the above unit stresses, assuming the stress to be either all live load or all dead load, we would have sections of 2660 square inches, approximately, or 1330 square inches, approximately, which are enormously greater than the actual. The weight of the chords would, of course, rise in proportion as the sections were increased, which would thus have increased the dead load, so that probably a compromise would have been the best. The writer does not suggest that these rules should have been applied for proportioning these sections, but the actual sections employed indicate a considerable departure from them.

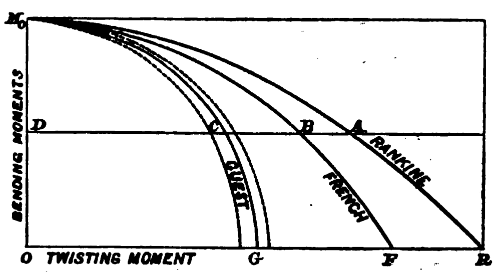
Yours faithfully,
September 25, 1907. A.M. INST. C.E.

THE STRENGTH OF SHAFTS.

TO THE EDITOR OF ENGINEERING.

SIR,—In your issue of the 13th inst., Mr. Walter A. Scoble draws attention to the fact that last year Assistant Professor A. L. Hancock, of Purdue University, in America, and he himself in this country, afforded us some evidence which can be accepted as a confirmation of the truth of Guest's law, and hence of Guest's formula, $M_0 = \sqrt{M^2 + T^2}$. I had previously seen and carefully studied the paper of Assistant Professor Hancock. I regret that it leads one, however reluctantly, to come to the conclusion that it contains sufficient evidence to cause one to hesitate to accept hastily the results stated therein. Mr. Walter A. Scoble's researches, apparently, extend to a dozen tests only, but they afford an independent confirmation of Guest's work, and we may now accept the corresponding formula with some degree of confidence.

It is worth while looking at a graphic representation of the various formulæ. In the diagram the ordinates are taken to represent bending moments, and the abscissæ the combined twisting moments. If O M_0 represent the permissible bending moment, then the curves M_0 A R, M_0 B F, M_0 C G, which are, respectively, a parabola, an ellipse, and a circle, represent Rankine's, the French, and Guest's formula. Note that O R = 2 O G. The position of F depends on the value of Poisson's ratio.



M_0CG is a circle, equation $M^2 + T^2 = M_0^2$
M_0BF is an ellipse, equation $25\,T^2 + 16\,M^2 + 48\,M_0M = 64\,M_0^2$
M_0AR is a parabola, equation $T^2 + 4\,M_0M = 4\,M_0^2$
(806)

The diagram shows the twisting moments D A, D B, and D C, which, according to the various formulæ, can be applied to a shaft already loaded with a bending moment O D—*e.g.*, if O D has the value ½ O M_0, then D A, D B, and D C are in the ratios 1.63 : 1.32 : 1—a surprising variation.

On the right side of the Guest formula curve I have sketched a broken line curve which shows the *actual curve* plotted from the average of Guest's experimental results; on the left side of Guest's formula curve I have shown a dotted curve corresponding to Mr. Scoble's results. The nature of his experimental method tends to produce slight errors which would throw the curve to the left. In this connection those interested should refer to Sections 55 and 56 of Guest's paper (*Philosophical Magazine*, 1900).

These are, however, minor points, involving about 4 per cent. differences. As in the usual proportion of crankshafts the approximate assumption involved in the deduction of the formula, from the law, almost certainly implies larger errors, the Guest formula seems to be as accurate as we are likely to obtain. It is, fortunately, very simple. Further experimental work is now in progress, and seems desirable.

I am, Sir, yours faithfully,
C. A. SMITH.
Engineering Laboratory, East London College.

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the work. The blow-pipe used contains special interior arrangements to ensure the intimate mixture of the oxygen and acetylene before their ignition. Four sizes of blow-pipes are made for different thicknesses of metal to be welded. There is also a special blow-pipe designed for cutting off small bars, sheets, and tubes. Such a plant is very convenient, especially for outdoor repair work which cannot be taken to a large electric plant for welding, or to shears and cold saws for severance. The alternative to the latter would be the tedious use of the cold sett. The "Warden-Simplex" system, as it is termed, occupies a maximum space of barely 12 ft. in length by 4 ft. 6 in. in height. Iron and steel plates up to nearly 1 in. in thickness can be welded or severed.

The Mork Patent Pulley-Block Company show a number of their self-sustaining worm-geared pulley-blocks. In these the load is lifted by the worm-gear; but in order to draw the unloaded hook down (or up) instantly, a lever is pulled by a cord, which elevates the worm-wheel, the lever of which is locked by a catch and spring, and leaves the pulley loose to be readily pulled round for raising, or by the gravity of the hook for lowering. The hook can be stopped at any desired height by throwing the worm-wheel into gear by means of a pull at the hand-chain. A brake is provided for sustaining the load in any position. It comprises a friction-disc, running loosely on the worm-spindle. There are two shoulders on the disc and two claws on the spindle, which make a claw-clutch arrangement. When a load is being raised, the claws engage in the shoulders of the friction-plate, and cause it to revolve freely. When lowering is being done, the weight of the load tends to turn the spindle in the opposite direction. Then the claws on the spindle engage the shoulders on the disc in such a way that it is pressed against a thrust bearing furnished with a leather washer. At the same stand are specimens of the firm's older pulley-block with self-sustaining brake, of which over 100,000 have been made. The worm-gear cannot be thrown out of action in this case. The brake is on the worm-shaft. There are also some travelling blocks or trolleys, and an overhead track is rigged with turn-table curves and switches.

There are several gear-hobbing machines in the Exhibition. An excellent tool (see Fig. 11, page 448), and very powerful, is shown by Messrs. Humpage, Thompson, and Hardy, Bristol, who now make a speciality of this class of machine-tool, manufacturing it in 32-in., 22-in., and 16-in. capacity, while other sizes up to 52 in. are being designed. The machines are built very stiffly, to enable them to cut large pitches even on the smaller machines. Some samples of their slogging capabilities are shown at the stand. Thus, a cast-iron wheel of 19 teeth, 3 diametral pitch, and 6½ face, was finished in 23 minutes, using a high-speed hob. One of mild steel, with 15 teeth, of 2½ diametral pitch, and 3¼ face, was finished in 49 minutes, with a hob of carbon steel, and other examples in similar rapid time.

The machines can be driven by electric motor or by belt; in either case the driving speed is constant, and no countershaft is used. Thence a gear-box provides four changes of speed. The feeds are graduated by cones while the machine is running. This is shown clearly by the little photograph, Fig. 12, in which the cones and gears are exposed. There is no slip of the cone-belt, which has very little work to do, because the power of the belt is multiplied greatly by the epicyclic gears seen to the right. A great advantage is claimed for this over machines in which the feeds are obtained through change-gears.

The worm which drives the work-table is double-threaded, which is an advantage when cutting worm-wheels of multiple threads. Tables are furnished giving the correct cutting speeds and feeds for different metals. These gear-hobbing machines are good high-speed tools, and their parts are made to templets and jigs.

PIG IN GERMANY.—The production of pig in Germany and the Luxembourg in August beat the record for any one month, having amounted to 1,117.545 tons, as compared with 1,064,957 tons in August, 1906. The total of 1.117,545 tons was made up as follows:—Casting pig, 194,465 tons; Bessemer pig, 41,447 tons; Thomas pig, 733 047 tons; steel and spiegel pig, 82,724 tons; and puddling pig, 65,862 tons. The aggregate production of pig for the eight months of this year was 8,587,464 tons, as compared with 12,478,607 tons in the whole of 1906, 10,987,623 tons in the whole of 1905, and 10,103,941 tons in the whole of 1904.

THE QUEBEC BRIDGE.*

By FRANK W. SKINNER, M. Am. Soc. C.E.

(*Continued from page* 419.)

THE CONNECTION OF CANTILEVER ARMS TO SUSPENDED SPAN.

THE ends of the top chords of the suspended span are connected to the top chords and end vertical posts of the cantilever arms, and the horizontal shear passes from panel to panel of the regular top lateral systems of the respective spans through sliding connections, allowing for temperature expansions at these points. The horizontal shear in the bottom lateral system is transmitted from the centre span to the cantilever arms through square vertical pins (see Fig. 87, Plate LV., and Figs. 90 and 91, page 453) in a sliding connection between pairs of special floor-beams at both of these points. The stringers have overlapping cantilever connections.

All connections at one end of the suspended span are duplicates of the corresponding connections at the opposite end, and provision is made for a maximum longitudinal displacement of 24 in., which, however, cannot all take place at one point, but beyond certain limits must be divided between the opposite ends of the suspended span. Theoretically it is assumed that the centre point of the suspended span is fixed, and that both ends expand equally, and about half as much as the expansion at the end of the cantilever arm; practically the location of the expansion movement is indeterminate except as governed by the arrangement and dimensions of the expansion provisions.

Stringer Expansion Connections.—At each end of the suspended span there are two special floor-beams (Figs. 84 to 88, Plate LV.), normally 7 ft. 3 in. apart on centres, one of them rigidly attached to the suspended span, and the other to the cantilever arm. From the adjacent sides of the webs of these floor-beams there project, from each, twelve cantilever brackets, 4 ft. 7¼ in. long, each with its centre line on the centre of one of the regular stringer lines. The overlapping ends of these cantilevers have their flanges cut to clear, while their webs are nearly in contact on opposite sides of the centre lines. The length of the overlap is greater than the maximum expansion at this point, so that the pairs of cantilevers together always provide continuous support from one span to the other. The cantilevers connected to the suspended-span floor-beam are X-braced together vertically and horizontally in alternate panels, and those of the cantilever arms are similarly braced in the intermediate panels, so that all are thoroughly stiffened.

Top Lateral Expansion Connections.—The single transverse top lateral struts, at the connections of the suspended span and the cantilever arms, are rigidly connected to the oscillating vertical end-posts of the cantilever arm, and, with them, move several inches longitudinally in the flat arcs of circles described about the lower pins of the posts as centres. The struts are fixed relative to the suspended span, except that they have a slight angular displacement around a horizontal pivot due to the oscillation of the end vertical posts. Each strut transmits the transverse component of the stress in the end diagonals of the suspended span to the diagonals in the end panel of the cantilever arm, thus making the top lateral system continuous. The suspended-span end top lateral diagonals are riveted to the top chords near the points where the latter have fixed connections to the transverse struts. The diagonals are directly connected to the struts by means of top and bottom horizontal flange-plates reinforced to take solid bearings on their edges between pairs of narrow jaw-plates projecting from the side of the strut, with clearance enough to permit the slight angular movement between the two members and maintain transverse bearing. The extremities of the top lateral diagonals in the end panels of the cantilever arms have wide projecting top and bottom flange cover-plates, which, on their longitudinal edges, are riveted to the cantilever-arm top-chord struts, the ends of which slide longitudinally in the oscillating connection links at the ends of the suspended span. The outer transverse edges of these plates form tenons projecting, between guides, inside the lateral struts, in which they slide with constant bearing, back and forth, according to expansion.

Top-Chord Connection.—The end of the top chord of the centre span is made special with three heavily reinforced longitudinal webs, and is field-riveted to the regular chord section (Fig. 97, page 454, and Fig. 100, page 455). It is approximately triangular in longitudinal elevation, about 15 ft. long and about 9 ft. deep, with separate holes for the 12-in. pins through the tops of the vertical and diagonal bars, making up the web members of the end panel of the truss. The 9-ft. longitudinal extension of the upper part of this connection-piece beyond the centre of the pin, through the top of the vertical bars (virtually the end top-chord pin), has a horizontal slotted hole, 36½ in. long, for the 12-in. pin through the top-chord eye-bar in the end panel of the cantilever arm. The pin through the centre hole engages both the eye-bars at the end of the centre span and the vertical end-post of the cantilever span, which are inclined slightly from each other, so that their feet have a short clearance between them, allowing the ends of the bottom chords in the span and arm to be separate and disconnected (see Fig. 98, page 454). The centre top-chord pin thus acts as a rocker, about which the nominally vertical eye-bars can oscillate through a short arc, like a pendulum.

Bottom-Chord Connections.—The end bottom chords of the centre span are riveted members, with a rectangular cross-section about 5½ ft. wide and 4 ft. deep, corresponding to the cantilever-arm bottom chord, but, unlike that, having only three vertical longitudinal webs. These are 25 in. apart on centres (Figs. 93 to 96, page 454), and at the outer ends are reinforced to total thicknesses of 6 in., and connected by six horizontal transverse diaphragms, with their ends faced to bearing on vertical transverse plates, which temporarily take the erection reactions of the adjustment jacks and shims (see key diagram, Fig. 92, page 454). This construction is comprised in the end section, 9 ft. 3 in. long, which is field-riveted to the main part of the bottom chord, and has the two outside web-plates extended about 8 ft. 10 in. above the top flange-angles, to form gusset connections, or jaws, to receive the field-riveted connection for the inclined end-post and the pin-connection for the vertical suspension bars. A view of this connecting-piece complete is shown in Fig. 99, page 455.

The end-panel bottom chords of the cantilever arms are special only in that all four webs are extended above the top flanges at the outer end (see Fig. 92, page 454), and heavily reinforced to form jaw-plates bored for the 12-in. pins through the lower ends of the end inclined eye-bars and vertical post. These extended webs are connected, above the top flanges of the chord, by two vertical transverse diaphragms, and between the top and bottom flanges by top and bottom flange cover-plates, and by ten horizontal transverse diaphragms between the pairs of webs, all of them being faced at the outer end for bearing on the thick vertical transverse end-plate which temporarily receives the reactions of the erection adjustment jacks and shims.

Lower Lateral Expansion Connection.—The lower lateral diagonals in the end panels of the suspended span are connected to the ends of special floor-beams, field-riveted to the gusset-plates at the ends of the special lower chords. These floor-beams (Fig. 84, Plate LV.) are plate-girders about 9½ ft. deep, with the lower parts of their webs stiffened by a pair of fish-bellied girders, with horizontal webs 5 ft. 9 in. deep at the centre, field-riveted to horizontal angles, and shop-riveted to the outside of the floor-beam web. The horizontal webs, in the planes of the top and bottom flanges of the lateral diagonal struts, are about 3 ft. apart vertically. They are connected by longitudinal vertical diaphragms made with X-brace and horizontal angles, and their outer flanges are latticed together. On the centre line of the bridge the webs have rectangular longitudinal slots $10\frac{1}{16}$ in. wide and 33 in. long, with the edges reinforced by cover-plates and stiffened with transverse angles riveted to them almost continuously.

The end floor-beam of the cantilever arm is similar, except that the horizontal stiffening-webs are about 4 ft. 5 in. apart vertically, thus having clearance outside of the webs of the suspended span floor-beam, which enter between them, and the rectangular holes in them are $10\frac{1}{32}$ in. by

* [This article was, of course, written prior to the occurrence of the grave disaster with which we had occasion to deal in our issues of September 6 and 13. As a complete account of a most important work, however, it has lost none of its interest, and a study of the information which it contains will add much to the understanding of the inquiry now being made into the causes of the failure.—ED. E.]

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the successive stages of work of greater magnitude and difficulty than had ever been executed. Stress-sheets were prepared, weights, moments, resistances, deflections, and displacements were accurately computed for known and assumed conditions, and elaborate analyses were made of stresses throughout the incompleted structure in many stages of construction. The superstructure was typical of American practice in that all portions of it were completely finished in the bridge-shops, requiring no field-work, except the assembling together of the members that form truss units, thus securing machine-work and great accuracy throughout, and minimising the time and expense of field operations.

The most important elements of the erection were: the very elaborate preliminary investigations; complete provision for probable and possible contingencies; the modification of panel-point truss-connections by the use of multiple pins, facilitating assembling in the field, and diminishing the pin stresses; extreme accuracy of construction, which eliminated preliminary assembling of truss members in the shops; special transportation; special storage and handling at site; use of steel false-work for anchor arms; use of independent wooden false-work for service tracks; type of main erection traveller; extensive use of power for all erection operations; elimination of steam power at site, and the installation of complete electric plant there; and the development of numerous special appliances for the details of the field-work.

Shipment.—Erection was considered to commence with the delivery of finished members to the railroad at the bridge-shops at Phœnixville, about 50 miles from Philadelphia, Pa. Here the members were stored in yards commanded by electric travelling girder-cranes, which loaded them as required on cars, by which they were transported about 500 miles, over five different lines of railroad, to the site, a trip that occupied from six to thirty days, averaging ten days for a train of twenty cars. As some of the members were 100 ft. long, and weighed over 100 tons, it was necessary to load them on three cars, the middle car serving merely as a spacer, and not receiving any part of the weight of the long member, which was concentrated on the front and rear cars, and applied so that they were free to swivel and slide longitudinally, to provide for movement on curves, which were encountered to a maximum of 14 deg. A number of special steel cars, 35 ft. long, with a capacity of 180,000 lb., were provided by the railroad company, which designed them in accordance with suggestions from the Bridge Company's engineers, and used them exclusively for this service, returning them to the shops after each delivery at the bridge site. The 11-ft. by 13-ft. by 8-ft. main pedestals weighed about 156,000 lb. each, and were carried on a special steel car with a heavy girder floor framework, through which the pedestal, supported on a box-girder passing through its 24-in. pin-hole, was suspended just clear of the road-bed between the rails, the only position in which it had clearance for the bridges and tunnels.

Storage.—A special standard-gauge track, about one mile long, was built by the contractors from the trunk-line to the south end of the bridge site, and involved considerable heavy cut-and-fill work and costly bridge construction. A similar branch, 8 miles long, was required for the north side of the river, but its construction was delayed. The south-side tracks terminated in a storage yard commanded by two 60-ton electric girder-cranes of 68 ft. span and a clear height of 30 ft. above the ground.

The crane runway, 750 ft. long, was made with 3-ft. plate-girders 60 ft. long, bolted together end to end, and supported on double-trestle bents, each made with a vertical and a battered 12-in. by 12-in. wooden post, seated on a 15-ft. horizontal sill. The girders were not braced laterally, but had their top flanges stiffened by a 15-in. channel, and were knee-braced to the trestle caps. They were designed to be used for the yard cranes at the Phœnixville shops after the completion of the bridge. Four pairs of trestles for each line of girders were braced together longitudinally, making stable towers; the intermediate bents, 30 ft. apart, were unbraced. Each crane was equipped with three independent electric motors, with maximum speeds of 15 ft., 100 ft., and 200 ft. per minute, for hoisting, traversing, and locomotion respectively.

Between the runway girders, and parallel with them, were two standard-gauge surface tracks, one connecting with the railroad, and the other extended about 3500 ft. to the end of the bridge, and provided with two electric locomotives and fifteen steel flat cars for delivering materials to the traveller. The remainder of the space in the storage yard was occupied with 12-in. by 12-in. transverse sills laid on the levelled surface of the ground and supporting 12-in. by 12-in. longitudinal timbers, 3 ft. apart, with light steel rails spiked to them, on which the bridge members were piled and skidded to the service cars. The yard had a storage capacity for about 15,000 tons, and the steel was piled on its skids as high as could be cleared by the cranes.

Members were classified as much as possible in storage, and careful records were kept of their receipt and location. Before the erectors were ready for the members of any given panel they were taken from the general pile, turned end for end if necessary, seated on temporary skids, thoroughly inspected, fitted with connection-plates, fillers, &c., if necessary; any field-riveting that could be advantageously done before erection was attended to, clearances verified, the member turned in the proper direction for hoisting to position, the lifting devices fitted to it, and it was then held in readiness to be delivered to the service cars and erected without any delay. The superintendents' and inspectors' offices, repair-shops, general storage, machine-shop, smith's shop, and a compressor plant were located near the end of the bridge, and all machinery was operated by electric power, brought by special cable from the Chaudiere Falls plant, two miles away. A similar plant was to be installed on the opposite end of the bridge, and it was estimated that the erection equipment cost over 250,000 dols., only a comparatively small proportion of which would be available for other work after the bridge was completed.

Steel False-Work.—The two 500-ft. shore spans, forming the anchor-arms for the channel cantilevers, weighed about 14,000,000 lb. each. The south anchor-span was erected on steel false-work weighing about 1200 tons, which was then removed from its initial position and taken across the river, to be re-erected there for the construction of the north anchor-arm. In consisted of eighteen 9-ft. by 9-ft. steel towers from 127 ft. to 160 ft. high, one under each of the nine intermediate panel points of each of the 500-ft. trusses, 67 ft. apart.

Each pair of towers was connected by two horizontal struts and two sets of X-bracing to form a transverse bent, and these bents had longitudinal horizontal struts and X-bracing, making three 50-ft. by 71-ft. single towers and one 100-ft. by 71-ft. double tower, with three open panels between them. Each tower was seated on an 18-ft. by 18 ft. grillage made of 12-in. by 12-in. timbers laid close together on a sand cushion on the levelled surface of the hard ground and calculated to distribute over it a maximum uniform pressure of 3000 lb. per square foot. Cross-timbers were bolted to each end of the grillage timbers to receive the tower pedestals, and each footing contains about 8000 ft. (b.m.) of pine timber.

The four vertical posts of each tower had rectangular cross sections made with pairs of 12-in. or 15-in. latticed channels, made in three sections, with faced butt-joints spliced with four outside single cover-plates. The lower ends of all posts were field-bolted between pairs of short channels shop-riveted to the pedestals. The latter were either heavy double-web shoes, connected in pairs by continuous lower flange angles, or they were virtually double-web plate-girders, 27 in. deep and 14 ft. long, with full length top and bottom flange-angles, and each receiving two tower-posts. In both cases they had pin-holes through the webs and through side plates for the diagonal eye-bars.

The inside pair of posts of each tower were proportioned for a maximum load of 250 tons each, and were of varying heights corresponding to the varying elevations of the panel-points of the curved bottom chords. Four 36-in. transverse girders, 10½ ft. long, were bolted across the flanges at the tops of the inside posts and across the flanges of the outside posts. The webs of each pair were connected by four shop-riveted vertical diaphragms, and their top flanges were connected by cover-plates 6 ft. long, on which were seated nine 20-in. 65-lb. I-beams, forming a grillage to receive the camber-blocks.

Two solid crossed tiers of 12-in. by 12-in. timbers, secured between guide-angles on the top flanges of the grillage-beams, formed a cushion for the lower camber-block on top of each tower (see Fig. 101, page 455). The bottom block was a massive riveted rectangular pedestal, about 5½ ft. long, 4 ft. wide, and 2 ft. high, with four equi-distant transverse plate-girders, having their bottom flanges riveted to the ½-in. base-plate and connected by two full-depth centre diaphragms and by a set of six 12-in. I-beams, web-connected between the girder webs each side of the centre, so that their upper flanges formed seats for two 500-ton hydraulic jacks.

The upper camber-blocks were, in general, like the lower ones, reversed, and had corresponding chambers receiving the upper parts of the jacks, which engaged very thick bearing-plates on the transverse beams. Guide-angles on the top flanges of the lower blocks centred the upper blocks, and their lower flanges had a small clearance with the top flanges of the lower blocks, the space being kept filled with steel shim-plates as the jacks were operated. Bevelled filler-plates were riveted to the tops of the upper blocks to provide bearing for the lower chord flanges. Where the bottom chords had a maximum inclination, the tops of the upper camber-blocks were bevelled nearly 45 deg. to correspond, and were secured to them by as many as forty-six bolts to prevent sliding.

The outside tower-posts were proportioned for 200-ton maximum loads, and extended from 10 ft. to 80 ft. above the inside posts to carry the traveller tracks at roadway level. Reversed batter-posts were connected to the outer faces of the outside vertical posts, and were spaced 12 ft. distant from them at the top, where both vertical and inclined posts carried lines of plate-girder stringers 58 in. deep, one under each rail of the 12-ft. gauge traveller track, seated on the post-caps and bolted together through their end vertical web-stiffener angles, and through transverse web-plates riveted to the posts, to provide anchorage for the vertical upward reactions from the traveller.

The outside stringers received wind-stresses only, and were single, the inside line received both wind and live-load stresses, and was double. Fifteen panels of stringers were provided, and were moved forward from rear to front as the work progressed, the traveller never requiring to recede after the trusses were completed. All members of the steel false-work were assembled with turned steel bolts, and when re-erected on the north side of the river would be adjusted to the variations of level of ground there by special lower sections of the posts, the upper parts remaining regular and interchangeable.

Wooden False-Work.—Materials were delivered to the traveller on a wooden trestle between the two lines of steel false-work towers, and entirely independent of them. The main transverse bents were located a few feet eccentric from the panel-points to clear the steel towers and the truss members, and between them, from 22 ft. to 28 ft. apart, were located the intermediate bents, one of each to every 50-ft. panel. The bents were about 167 ft. in maximum height, and were made in five or six stories, with four 12-in. by 14-in., and one centre 10-in. by 12-in. vertical posts in each, with butt-joints spliced with pairs of 4-in. by 12-in. scab planks, having five ¾-in. bolts in each end. There was a single 4 in. by 8-in. horizontal transverse strut bolted on at each story, and the panels between them were X-braced by 4-in. by 12-in. planks.

The posts were spaced 17 ft. and 24 ft. from the bridge axis, and the outer pairs had 12-in. by 12-in. caps and sills 10 ft. long, bearing on full-length caps and sills, the former carrying cribwork or pony bents for the plate-girder track-stringers, and the latter seated on grillages of four 12-in. by 12-in. longitudinal timbers 4 ft. long, carefully bedded with sand cushions on the levelled surface of the ground. The intermediate bents were like the main bents, except that they were made with three vertical posts. All bents were connected by continuous longitudinal struts at each story, and alternate panels between main bents were X-braced, making successive towers and open panels.

The tops of the main bents were at first guyed transversely by steel cables until after the steel false-work towers were erected, after which the guys were replaced by bracing against the steel-work. This false-work was proportioned for a maximum load of 400,000 lb. per bent, which was distributed by the grillages to reduce the load on the ground to about 4000 lb. per square foot, an amount which did not produce any noticeable settlement. As the ends of the timber were all cut square, there was very little framing to be done; but the pieces were cut to length, bored with

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pneumatic augers, and assembled and bolted in single story panels on the ground at the end of the bridge, and erected by a special wooden erection traveller.

The False-Work Traveller.—The traveller, 100 ft. long, 116 ft. wide, and 30 ft. high over all, virtually consisted of a four-bent 46-ft. by 61-ft. rectangular tower, mounted on twelve double-flange wheels running on the two outside rails of the material tracks and on a single centre line. The transverse bents were balanced double cantilever trusses with the horizontal top chords overhanging 30 ft. each side. They were connected by vertical longitudinal bracing, equivalent to trusses, in the planes of the track-rails, 23 ft. apart. These trusses formed cantilevers overhanging the centres of the forward trucks 51 ft.—far enough to command one full panel of the bridge in advance of the completed false-work.

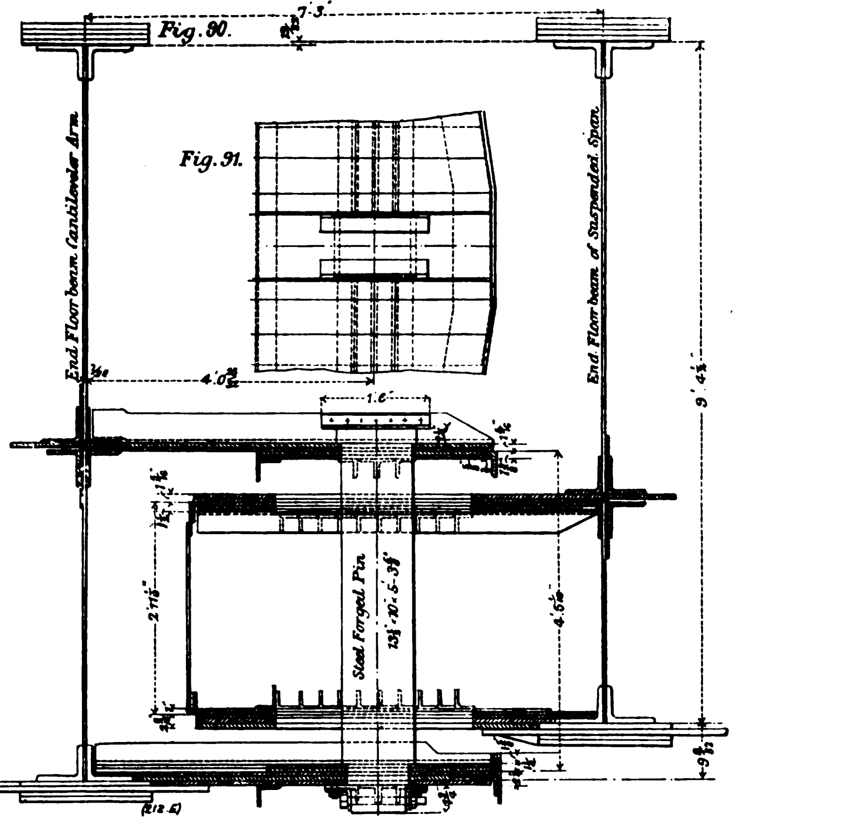
Both longitudinal and transverse trusses were X-braced together, thus securing great rigidity, and wooden jigger beams laid on their horizontal top chords afforded support for Manilla, rope tackles operated by two standard electrically-driven four-spool hoisting engines carried on the traveller floor. The chords and sills were made with pairs of 6-in. by 12-in. timbers, receiving the 6-in., 8-in., and 10-in. square timbers of the other principal members between them. Connections were made with wooden scabs, except where pairs of bolted ⅜-in. steel gusset-plates were used at a few important points. This traveller moved on top of the completed false-work and erected it in advance, panel by panel, very much as a permanent steel viaduct is often erected. After completing the false-work erection, it erected the lower part of the steel traveller.

The Steel Traveller.—The steel traveller, with which all of the superstructure, except the centre suspended span, was to be erected, was of special construction, of the outside gantry type, and constituted the most striking and important part of the erection plant. It combined the principal features of the largest travellers heretofore used for either free or cantilever spans, and had entirely new details, adapted to the novel structure it assembled and conforming to its unprecedented magnitude.

It contained about 2,000,000 lb. of steel, and with its equipment weighed over 2,225,000 lb., an amount which seems almost incredible when it is remembered that it was at first intended to erect the entire superstructure with it and thus develop, when in the centre of the suspended span, a moment of nearly 200,000,000 foot-pounds. The superstructure of the permanent bridge was proportioned to receive this enormous temporary stress at a higher unit intensity, of course, than that adopted for the working loads. But it was finally decided to erect all but the end panels of the suspended span with a special smaller traveller, thus releasing the main traveller, and enabling it to be dismantled and re-erected on the opposite side of the river, to erect the north anchor-arm, while the south half of the suspended span was being erected by the second traveller, thus reducing the maximum moment of the large traveller to about 111,000,000 lb.

The traveller ran on tracks outside the main trusses, and cleared the completed structure, thus enabling all the members of the latter to be assembled and permanently connected as each panel was built out, avoiding the necessity for temporary bracing.

Although the traveller passed over the tops of the trusses 315 ft. deep, its vertical clearance was only 202 ft., since its track was at roadway level over 100 ft. above the lowest points of the parabolic bottom chords. It was designed to run from anchorage to anchorage on tracks about 120 ft. above the main pairs and 160 ft. above water-level, and to assemble the three spans of continuous trusses, 2800 ft. long, handling 100-ton members 100 ft. long at a height of nearly 400 ft. above the water, to swing them in pairs, far beyond the supports of its own base, and maintain them safely and accurately in position until permanently connected and self-supporting. It was of the Z shape type, previously successfully used for the erection of the 812-ft. cantilever span of the Wabash Bridge at Pittsburg and the 700-ft. 6000-ton span of the Mingo Bridge across the Ohio River, but differed from them in that it was an outside traveller, like those of the plain tower type generally used for false-work erection, a feature which had been emphatically pronounced impracticable for cantilever erection on account of the difficulty of providing tracks for it. It consisted essentially of a 54-ft. by 103-ft. tower, with a trussed 54-ft. rear cantilever extension at the base to counterbalance the 66-ft. upper front overhang, from which the erection tackles and working platform were suspended. It was mounted on twenty-four double-flange wheels on two 4-ft. 8½-in. gauge tracks, 79 ft. apart on centres, and on two outside single rails 103 ft. apart.



The tower had two transverse bents 54 ft. apart, each with four vertical posts about 214 ft. long, spaced 39½ ft and 51½ ft. each side of the bridge axis, with their tops connected by a plate-girder 8 ft. deep, providing horizontal and vertical clearances of about 78 ft. and 209 ft. respectively, which allowed 4 ft. net above the highest point of the permanent superstructure.

The outer posts on each side were latticed together with angles, virtually making vertical trusses 12 ft. deep, and the feet of all the posts engaged pairs of jaw-plates on four lines of longitudinal sills, about 110 ft. long over all, which formed the lower chords of the bottom cantilever trusses, and were supported on trucks having four wheels under each inside post, and two wheels under each outside post. The tops of the posts were connected by horizontal longitudinal struts, about 123 ft. long over all, forming the top chords of the upper cantilever trusses, which, like the lower ones, were integral with the towers. The pairs of trusses were X-braced together, and the top of the tower was rigidly braced by transverse and diagonal members in the planes of the top chords. The inside vertical posts were proportioned for maximum live loads of 450 tons each, and the outer ones for maximum wind loads of 170 tons each.

Although the erection was planned to avoid, as much as possible, the support of many unconnected members simultaneously from the traveller, the latter was designed to hoist the principal truss members in pairs simultaneously for both trusses, and was proportioned for a maximum total hoisting load of 500,000 lb., which was partly balanced by the four 20-ton hoisting-engines and other equipment carried on platforms supported on the rear ends of the bottom chords of the lower cantilever trusses. This did not, of course, give stability under maximum loads on the overhang, which was ensured by tying down the traveller to the completed structure. In actual use the weight of the traveller and its load was transferred from its wheels to steel wedges.

Erection of the Steel Traveller.—The erection of the 2,000,000-lb. traveller, 212 ft. high on top of false-work 150 ft. high, was itself an important and unusual operation, and as the structure had to be taken down twice from the extremity of the cantilever arms high over the river channel, and re-erected once on the opposite side of the river, it was doubly important to devise a method applicable to the rapid and economical handling of its members. As soon as the shore-end of the north shore steel false-work was completed, the lower trusses and the lower sections of the vertical posts of the traveller were erected by two stiff-leg boom-derricks and temporarily braced. Double-web steel-plate brackets were bolted to the upper ends of the four inside vertical posts, and received 4-part Manilla-rope tackles operated by the hoisting-engines to lift a skeleton working platform, made with light I-beams and plate-girders, bolted together and carrying boom-derricks.

After the platform was lifted as high as the tackles could hoist it from their first positions, a second set of steel brackets were bolted to the vertical tower-posts, just below it; it was lowered to bearing on them, the tackles were released, and the first set of brackets were removed. The vertical posts and other members of the tower were then built up as far as they could be commanded by the derricks, the brackets were bolted to the tops of the vertical posts, the tackles connected to them and to the corners of the working platform again, and the platform raised as before and seated on the lower set of brackets removed from the first position, and so on. The tower was thus erected in successive stages until completed, temporary transverse struts giving it stability until the permanent upper girders and lateral bracing could be assembled.

The Traveller Equipment.—The traveller was equipped with a working platform suspended from the upper cantilever at roadway level, to permit the extension of the two service tracks beyond the completed portions of the trusses; but this was not, of course, put in position until after the completion of the anchor-arm. It also had 36 main tackles, besides numerous small tackles and whip lines, operated by the four electric hoisting-engines, carried on cantilever side platforms clear of the service tracks and supported on the lower chords of the lower cantilever trusses. There were in all about 60 sets of lines, aggregating about 7½ miles of ⅞ in. plough-steel rope, 15 miles of 1¼-in. Manilla rope, 3 miles of 2-in. and 6 miles of 1¾-in. Manilla rope, besides about an equal amount carried at the site in storage.

The eight 55-ton steel-rope tackles were each suspended from a pair of 11-in. by 1-in. vertical steel-plate links inserted between the double webs of the overhead girders, and connected to them by 5$\frac{13}{16}$-in. pins through their upper ends. The lower ends of these links projected below the lower flanges of the girder and engaged 4-in. pins, which received U-shaped bent plate-hangers bored for 3$\frac{1}{16}$-in. pins

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THE QUEBEC BRIDGE; CONNECTIONS OF SUSPENDED SPAN.

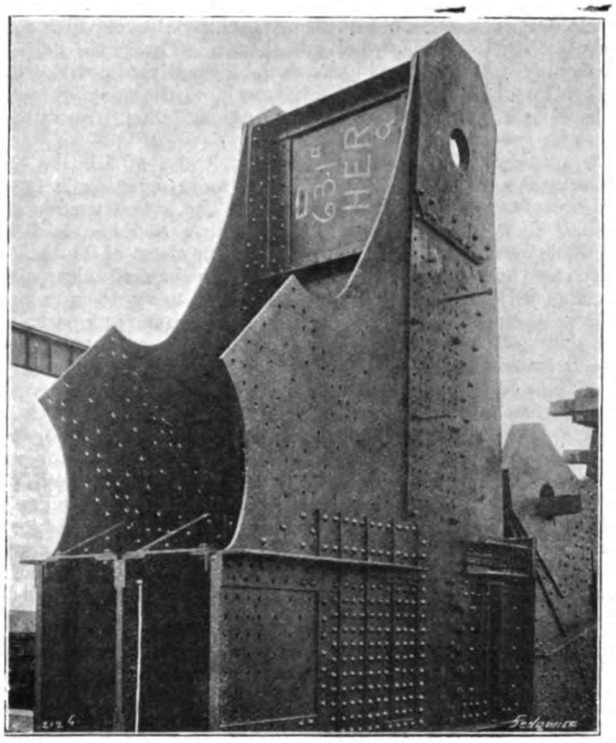

Fig. 99. Bottom Chord Connection; Central Span.

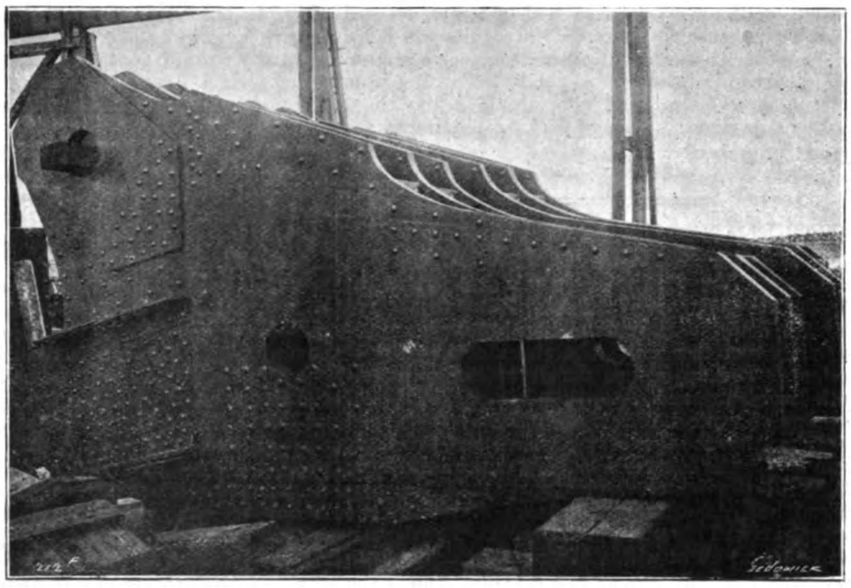

Fig. 100. Top Chord Connection; Central Span.

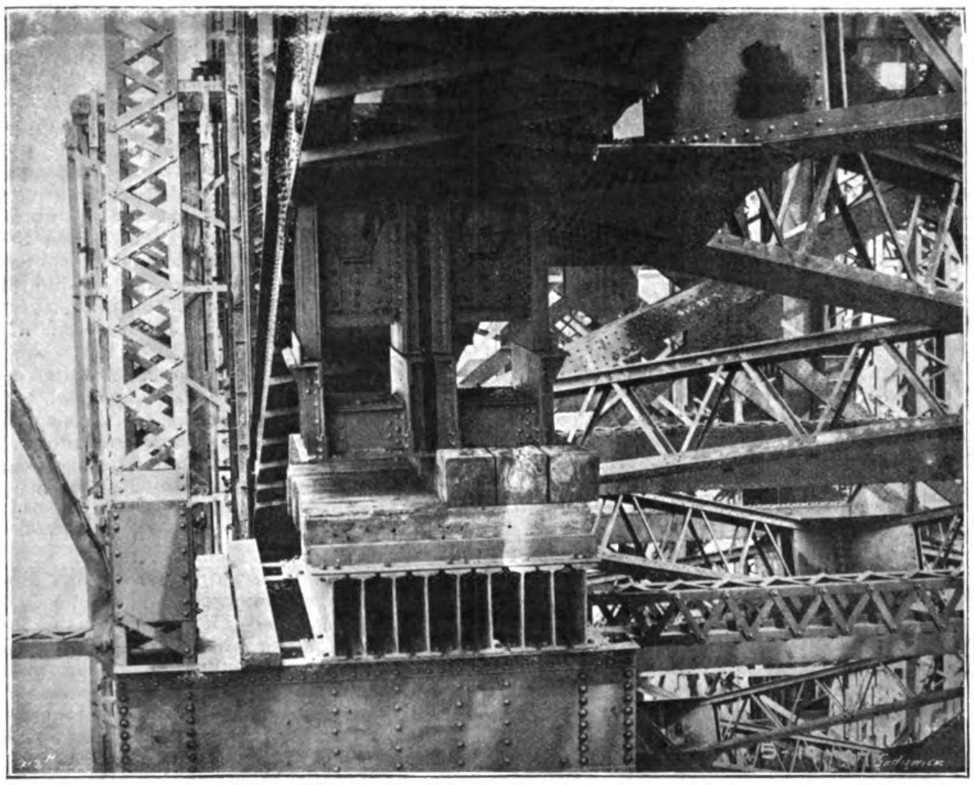

Fig. 101. Camber Blocks.

tapping his head. Great care was taken not to lower more than one member in either truss simultaneously from the traveller overhang, a precaution which was considered very important, to reduce the danger of subjecting the traveller to large indefinite loads whilst making the connections.

Hoisting-Tackles.—The weights of the different sizes of tackle, with lines, blocks, hooks, shackles, and overhauling weights complete, were about 542 lb. for a 5-ton, 940 lb. for an 8-ton, 1544 lb. for a 10-ton, 2974 lb. to 3810 lb. for a 22-ton Manilla tackle, and from 7600 lb. to 12,000 lb. for a 55-ton steel cable tackle, according to the attachments provided for its blocks. Each principal tackle required about 1½ miles of line, and the larger ones were of special construction.

The 10-ton tackles had 260-lb. triple wooden blocks with 11-in. sheaves and self-lubricating axles, and were rove with 1¼-in. Manilla line. The 17-ton tackles had 400-lb. quadruple blocks, 12½-in. sheaves, rove with 1¾-in. rope, and clevises with forked eyes engaging the 2-in. axles. The 22-ton tackles were similar, with 585-lb. blocks, 14-in. sheaves, and 2-in. lines. The 55-ton tackles had blocks made with steel web-plates, cast-steel sheaves and cast-iron separator-plates, and shackles 3 in. in diameter, with forked eyes engaging the $3\frac{9}{16}$-in. pins. The sheaves had self-lubricating bushings of gun-metal bronze, fitted with soft metalline plugs, designed to diminish the friction for a high velocity, continued 15 minutes under a pressure limited to 5000 lb. per square inch, or about half that of ordinary blocks.

Under specially-prepared tests, when a pressure of 8000 lb. per square inch was maintained, while the sheave was revolved thirty-eight times per minute for thirty minutes, the bearing was uninjured, although it became very hot and smoky. These sheaves weighed about 140 lb., and the upper blocks about 2140 lb. each. The lower blocks had the separator blocks extended to increase their weight to about 3550 lb. each. They were about 4 ft. long and 30 in. wide, and, notwithstanding their great weight, would not overhaul unaided. It was calculated that there was a loss of 55 per cent. of efficiency due to friction, and the resistance to bending of the ropes, and it was necessary, on at least one occasion, to attach another tackle to the lower block to pull it down, notwithstanding the fact that it was already loaded with a 23-ton load.

Hoisting Attachments.—The great weight, up to 105 tons, the dimensions, up to 100 ft. in length, or 10 ft. in width, of the members, and the necessity of maintaining them accurately in the required positions and hoisting them through varying angles, made the problem a most important one of securing them rapidly and safely to the hoisting tackles, and in such a way as to allow necessary freedom of movement, to avoid twisting, and provide for distribution of load between several tackles on the same member. Very careful study was given to the design of special appliances for all the different bridge-members, and thousands of dollars were spent in the construction of clamps, yokes, frames, and other devices for handling all the principal members in the storage yards and at the site. The devices varied greatly for the widely-different members, but most of the heavier ones included a heavy plate, or pair of plates, with three or four large pin-holes bored in it to receive shackles connecting it to the member on one side, and to three or four tackles on the other side, thus equalising the hoisting stresses, and providing for the attachment of the fleeting or drifting tackles.

Vertical Posts.—The intermediate vertical posts were erected in two or more sections, with splices field-riveted after the posts were assembled in the truss. In order to make the hoisting attachments clear the permanent connection, and allow the post to hang in the proper position in the tackles, the lower section was, while in a horizontal position on the service-car, fitted with a pair of 15-in. channels, about 5 ft. long, bolted to the open field rivet-holes in the main connection jaw-plates. The outer ends of these channels were bored for an 8-in. horizontal pin, nearly 4 ft. long, which engaged at the centre a 12-in. by ¾-in. U-plate, with its upper ends engaging a pin through the lower part of the tackle-plates.

For the upper ends of the lower sections of the posts the similar attachment had a pair of channels bolted transversely to the post-webs through the field rivet-holes in the splice of the latter, and projecting far enough beyond the post-flanges to provide an offset sufficient for the tackles to have clearance to revolve into a vertical position parallel to the post as the latter swung into its required position. The U-plate engaged one pin in its loop, and was bored for another upper pin, at right angles to the first, which received a second U-plate, with

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THE QUEBEC BRIDGE; CONNECTIONS OF SUSPENDED SPAN.

Fig. 92.

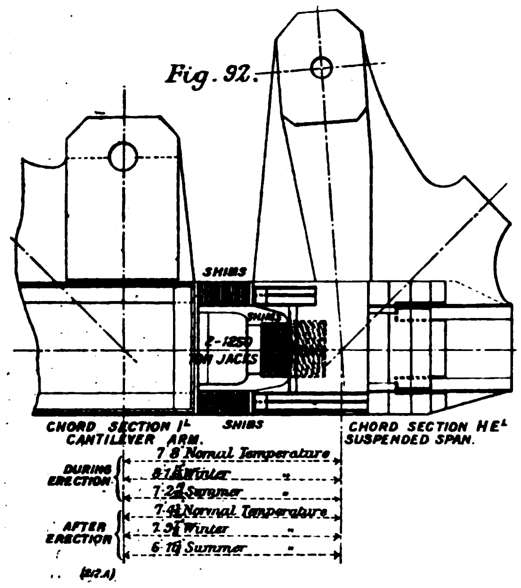
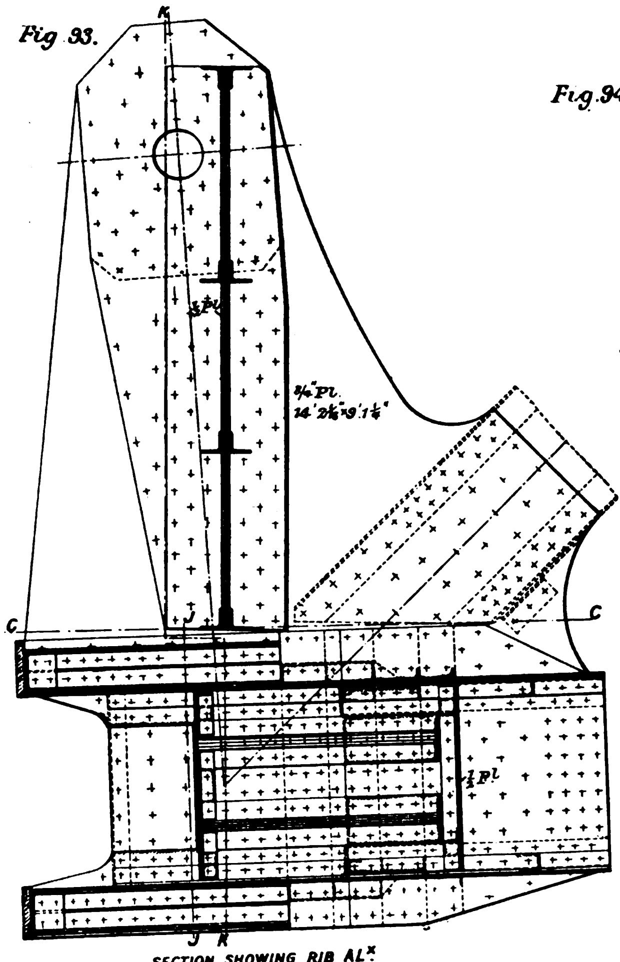
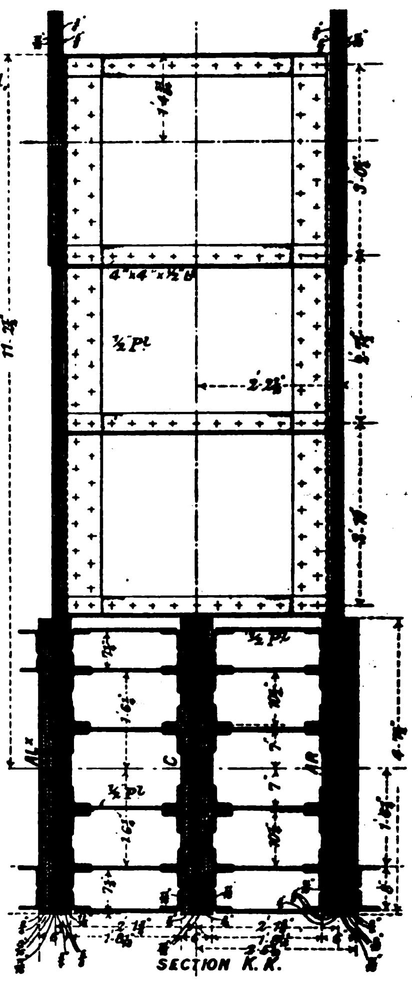


Fig. 93.



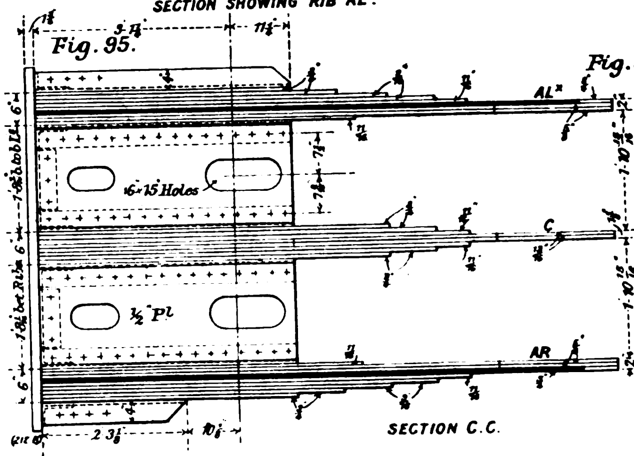
SECTION SHOWING RIB AL^x

Fig. 94.



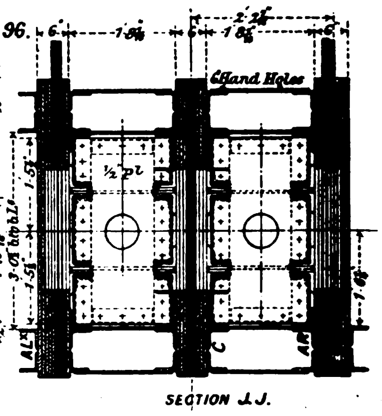
SECTION K.K.

Fig. 95.



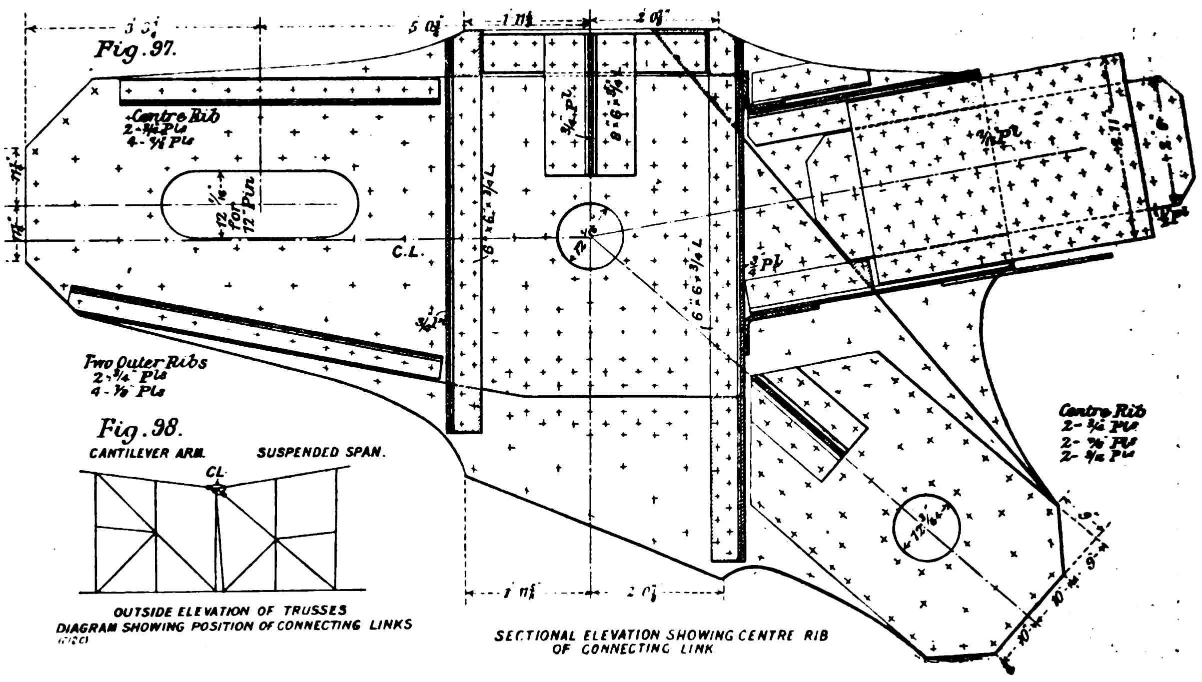
SECTION C.C.

Fig. 96.



SECTION J.J.

Fig. 97.



SECTIONAL ELEVATION SHOWING CENTRE RIB OF CONNECTING LINK

Fig. 98.

CANTILEVER ARM. SUSPENDED SPAN.

OUTSIDE ELEVATION OF TRUSSES
DIAGRAM SHOWING POSITION OF CONNECTING LINKS

transverse to the link-pins, and engaging the upper ends of a pair of 10-in. by $1\frac{1}{8}$-in. by 25-in. plate-links bored at the lower ends for $3\frac{5}{16}$-in. pins, with cast-iron saddles serving as bushings for the shackles attached to the tackle-blocks.

The hoisting-lines from these tackles ran over cast-steel guide-sheaves between the double webs of the girders. Great pains were taken to lead these and all other hoist-ropes, in the top of the traveller, to points between the outer and inner tower posts, where they could be run down to the hoisting-engines clear of the service tracks and the cars on them. They were therefore guided by snatch-blocks, and supported on small horizontal steel rollers with phosphor-bronze metalline bushings, set on plates adapted to be bolted on the under-sides of the stringers and beams in the top of the traveller. The heavy tackles were invariably supported directly from the steel girders, but those of a capacity of 22 tons or less were supported by vertical bars passing through holes bored through short beams seated on pairs of short 8-in. by 16-in. wooden beams, laid longitudinally, or transversely, across the top flanges of the steel beams or girders.

The special hoisting-engines, 14 ft. by $12\frac{1}{2}$ ft. by 7 ft. high, each weighed about 25 tons, and were driven by 115-horse-power 500-volt electric motors, operated by electricity from the hydraulic plant at Chaudiere Falls. Each of the two shafts had one 18-in. drum, 48 in. long, and two spools, all normally running free on it, and operated by independent clutches. Each drum had a capacity of 4500 ft. of $\frac{7}{8}$-in. steel cable, and was provided with a V band-brake and a safety appliance to prevent accidental disengagement of the clutch.

The operating levers were all grouped together, so as to be controlled by a single man with assistants at the spools; and a solenoid brake was arranged to automatically set and hold the armature shaft and prevent the loads from overhauling if any accidental interruption of the current occurred. The engine had two sets of gears of about 50 to 1 and 16 to 1 respectively, so that it could hoist on a single line either a 33,000-lb. load 100 ft. per minute, or a 15,000-lb. load 150 ft. per minute. Two engines together have lifted a 210,000-lb. load at a speed of 15 ft. per minute. The drums were used exclusively for the 55-ton tackles, and the spools for the lighter tackles. Two sets of eye-bars and their attachments, having a combined weight of about 170 tons, have been simultaneously lifted by the four engines.

All hoisting operations were controlled by the foreman in the lower part of the traveller, who directed the engine-man by gestures of his hands and arms, certain signs being always prefixed to designate which drum or spool was referred to, the drums operating the 55-ton tackle, for instance, being referred to by

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a pin for the tackle-plates, and also engaged a pair of short eye-bars, which provided a swivelling connection for the fleeting-tackle; they had a capacity of 76 tons. The upper ends of the upper sections of the posts were carried by simple riveted yoke-beams, with pins through the regular pin-holes in the posts. They had clearance to revolve around the upper end of the post as the latter swung from horizontal to vertical, and were provided, like those already described, with eye-bars for fleeting-tackle connections.

(*To be continued.*)

MISCELLANEOUS EXHIBITS AT THE ENGINEERING EXHIBITION.

MESSRS. LEE, HOWL, AND CO., LIMITED, of Tipton, Staffs., are exhibiting the Howl and Tranter detartariser and feed-heater for preventing scale and increasing the steaming capacity of boilers. The apparatus, which is made to suit both Lancashire and water-tube boilers, consists of a series of conical dishes arranged in the steam space, so that the feed-water enters and overflows from one to the other before it joins the rest of the water in the boiler. The feed thus becomes heated and deposits its insoluble matter in the dishes, instead of on the boiler walls. The deposit can be blown off at will, and scum is removed from the surface of the water at the same time by a trumpet-shaped pipe, which is also connected to the blow-off. It is generally accepted by engineers that, apart from the prevention of scale, very considerable increase of boiler efficiency is obtained by heating the feed right up to steam temperature, so that the boiler and furnace-plates are doing the work of evaporation only. Messrs. Lee, Howl, and Co., Limited, also show types of boiler-furnaces and two patterns of boiler feed-pump.

The Standard Engineering Company, Limited, of Leicester, show several blowing and exhausting-fans, but more especially a system of heating for factories and large buildings. No air-ducts are used either overhead or underground, but at intervals of about 60 ft. "Stanlock" heating units are placed. These are cylindrical metal casings, containing a nest of steam-pipes, over which air is passed by a small motor or belt-driven fan at the top. The air, which may be taken either from outside or inside the building, is warmed in its downward course through the heater and delivered just above the floor-level. It is claimed that the consumption of steam and power in this system, as well as the first cost, are lower than for any other. Further, the ease of altering the position of the units when desired, or of removing them in summer, is of great advantage.

The machine illustrated in Fig. 1 on page 458 is a hand-shearing machine built and exhibited by Messrs. Allen and Simmons, of Reading. As will be seen from the figure, it can be readily moved about the workshop on its wheels by using, as handles, the feet on which it stands when at rest. It can be used on sheet-metal up to ⅛ in. thick, and is particularly useful for cutting out planished steel lagging, as the cut can follow any curve. The severed metal will pass the framing, so that there is no limit to the length of cut which can be made. The knives are not held by bolts passing through them, but firmly clipped by their backs. Furthermore, special care is taken that the cutting edges are positively held in contact, so that they cannot spring apart when cutting, and bend the metal down between them. This is a very convenient tool to use for shaping thin sheet-metal, and is employed to a considerable extent in locomotive and other works.

The vice illustrated in Fig. 2, page 458, is one of the exhibits of Messrs. Wadkin and Co., of Leicester. As will be seen from the engraving, the jaws are at the end of an arm, which can be turned or twisted to any position by means of a ball-and-socket joint at the base. The arm can be securely locked by the half turn of a handle which closes the socket on the ball. Thus the fitter can instantly get his work into the most convenient position for filing or chipping any part, without having to shift it in the jaws, or to reach over or round it in an awkward manner. For small articles, and especially such as can hardly be gripped in more than one way, the vice should be an extremely serviceable tool, permitting of better and more rapid work than can be done with vices of the ordinary patterns, though possibly it would hardly have the rigidity required for heavy chipping.

A full-sized working example of a marine governor is shown by Andrews Governor Patents, Limited, of 64, Victoria-street, S.W. On one of the engine-room bulkheads, and running fore and aft of the ship, is mounted a steel tube half filled with mercury. The tube is pivoted near its centre, and normally rests in contact with a small valve-spindle, which, when depressed, holds open the throttle of the main engines. If the stern of the boat rises, the mercury rapidly runs up the tube and overbalances it, bringing it into contact with another valve, which closes the throttle. The sensitiveness of this governor may be varied by adjusting the point about which the tube swings. It is claimed that a governor of this type will act before the engines have had time to race, whereas the ordinary marine governor does not come into action until the engines are running above their normal speed. Of course, the difficulty in all governors of this class is to differentiate between the effect of alteration of level due to pitching and the effects of variations in the speed of the vessel on the inertia of the moving mass in the governor, a check due to striking a head sea acting in the same way as a rise in the stern. To make the Andrews governor control the engines when racing is due to some other cause than pitching, such as, for example, the loss of a propeller blade, an inertia attachment is fixed to one of the cross-heads. An increase of speed causes this to press against a pivoted slide on the engine framing, which, when moved, causes the operation of the throttle-controlling cylinder. The movement of this rod by hand gives the engineer an extra means of instantly shutting down the engines in case of emergency.

Messrs. A. Warden and Co., of 60, Great Eastern-street, E.C., show apparatus for welding or cutting metals by the oxy-acetylene process, and examples of work done with the apparatus. The acetylene gas is made from calcium carbide and water in a simple generator, and burnt with compressed oxygen in a special blow-pipe, giving a flame which will raise metals to welding heat very rapidly. For cutting metal, the flame is supplied with pure oxygen from an auxiliary jet. It is stated that a 2-in. square bar of iron can be cut through in less than three minutes by means of the flame, and the cut is only from ⅛ in. to $\frac{3}{16}$ in. wide. Oval and circular holes can be rapidly cut in boiler-plates, and there are many other uses to which the oxy-acetylene flame can be put.

Messrs. Murray, Lotz, and Co. show a form of construction for timber buildings, such as sheds, barns, &c., which is designed to avoid all mortising and to permit of cheap and rapid erection. The foot of every upright is clipped between a pair of channels set back to back and vertically in a concrete foundation block. The uprights are held by two bolts passing through them and the channels, and do not quite touch the ground. It is said that erection is performed by assembling the entire framing of each side of the building on the ground, putting the lower bolts through the feet of the uprights, and swinging the complete side into its vertical position by means of one or more derricks. The framing is joined throughout by cleats or angle pieces. We understand that examples of this type of construction are to be found on the Continent, and that it is advocated for agricultural buildings where skilled labour is difficult to obtain.

Two firms show excellent examples of die-castings—namely, Aerators, Limited, of Upper Edmonton, London, N., and Messrs. Norman and Young, Limited, of 83, Bishopsgate-street Without, E.C., those of the latter firm being of Continental manufacture. These castings are of various alloys of aluminium, according to the purposes and strength required, and are of such accuracy and finish that they may be erected without fitting or machining. Many examples of great intricacy are shown, having internal and external threads, holes, &c., cast where required. Instrument parts and cases, carburettors and such-like small motor-car details, up to 5 lb. or 6 lb. weight, are largely made by this method of casting, and the results appear excellent.

The Adjustable Cover and Boiler Block Company, Limited, of 64, Victoria-street, S.W., have on view specimens of their boiler-seating blocks, and a model illustrating their application. The seating-blocks are made to overlap, so as to prevent gas leakage between the flues; the face in contact with the boiler is as small as possible, and there is no place where soot can lodge. The cover-blocks over the side flues also overlap, and, further, are able to rock to accommodate the expansion and contraction of the boiler. Air-tightness is obtained by packing the expansion joint with slag-wool or asbestos. The whole or any of the cover-blocks can be readily removed without disturbing the brickwork, so that daylight can be admitted when cleaning the flue, inspecting the boiler, &c.

The Lamp Pump Syndicate, of 13, Walbrook, E.C., are exhibiting a most ingenious steam, or rather, vacuum, pump. The steam used is never above atmospheric pressure, and is raised by a paraffin lamp burning under a sort of kettle. The action of the pump could not be made clear without a diagram; but it will suffice to say that, excepting the valves, it contains only two moving parts. It is started by moving a handle up and down two or three times, and thereafter works without any attention so long as the lamp burns. There is no more danger about it than about a tea-kettle, and very little more complication; it wants no special house, foundations, or chimney. The pump shown working weighs complete about 1 cwt., and is said to be capable of lifting continuously 400 gallons per hour against a head of 40 ft. It would appear a very useful machine for agricultural or country-house use.

The Seal Motor Company, of Oil Mill-lane, Hammersmith, have on view several small internal-combustion motors for marine and other purposes. The Seal motors are in appearance very much like small totally-enclosed electric motors. The spherical casing contains an inverted cylinder, the piston being below the crank-shaft. Either paraffin, benzine, or petrol can be used as fuel, and automatic, tube, or electric ignition is provided. The water circulation is automatic, and the jacket can be cleaned out, or the working parts of the engine inspected, without dismantling or disturbing the adjustment of any details. The whole of the valve-gear, oil feed, and ignition parts can also be removed independently of the cylinder, and everything is arranged to make skilled attention unnecessary.

Messrs. Wasserman and Co., of Lausanne, and 11, Queen Victoria-street, E.C., are showing specimens of brazing performed on cast iron by means of their "Castolin" process. It is claimed that by this process cast iron may be brazed in a smith's fire, or gas blow-pipe flame, with strong and reliable results. A special flux, which is presumably some decarburising compound, is rubbed well into the cleaned surfaces of the fracture, which are then clamped together, and the casting heated in the fire. More flux is added as the metal becomes warm, and borax and spelter are then applied in the ordinary way. The joints are said to be at least as strong as the body of the metal, and the work can be done by anyone capable of managing the brazing of wrought iron.

The Forced Lubrication Company, Limited, of 73, Bridge-street, Manchester, are exhibiting pedestals fitted with an ingenious and effective method of forced lubrication. The lubrication of every pedestal is self-contained, and the shaft is supplied with a continuous stream of oil forced underneath it on the line of greatest pressure. The arrangement will be understood from Figs. 3 and 4, page 458, which explain how the system is applied to ordinary pedestals by merely replacing the steps with those of the design shown. The steps are preferably of cast iron, and the lower one is cored out at one end to form an oil-well, in which an extremely simple type of force-pump is fitted. This pump is worked by an eccentric on the shaft, which thus at every revolution forces oil under itself at a pressure only limited by its own weight. The eccentric is made in halves, so that it can be put in place without disturbing the shaft. The inlet-valve to the pump is merely a hole through the side of the plunger, which is closed by the descent of the latter, and the delivery or non-return valve is a steel bicycle ball. There are no pipes or outside connections whatever.

A new pattern of inferential water-meter is exhibited by Mr. Edwin Allen, of 496, Kingsland-road. The meter, the design of which is due to Mr. H. C. Ahrbecker, is represented in section and plan in Figs. 5 and 6, page 458. The water enters the meter at A, and passing up through the opening B, enters the guide-chambers, which are best seen in the plan. The flow, following the direction of the arrows, is directed on to the

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buckets of a turbine wheel C, which it drives round. The water finally escapes from the meter through the opening at D. The wheel actuates a counter by worm-gearing in the usual way. The guide-casing and the turbine wheel are aluminium castings, and are thus not subject to corrosion. A couple of vanes, represented at *e*, *e*, Fig. 5, are fixed to the upper side of the turbine wheel. By turning this so as to present more or less resistance to the motion of the wheel, the latter can be very accurately calibrated. The bearings are of vulcanite, and the spindle of aluminium-bronze.

The Margewood Lock Company, Limited, of 41, Gray's Inn-road, W.C., show types of lever locks made on a new principle, which is distinctly better mechanically than that commonly adopted. In the Margewood locks the tumblers slide with the bolt, which is made from sheet metal, and bent over so that the tumbler-pin is firmly held at both ends, and cannot get bent or loose. The stump or pin with which the tumblers engage is likewise held at both ends, and thus the resistance of the lock to being forced back or broken is many times greater than can be obtained with locks made in the ordinary fashion. It will be seen that the lock in question is, so to speak, an inversion of the common tumbler lock, so far as the internal parts are concerned, the security of the tumbler principle being retained, but incorporated with a really mechanical design. The workmanship of the locks also appeared excellent.

NOTES FROM THE UNITED STATES.

Philadelphia, September 25.

Conditions in the steel industry are not quite so satisfactory as a week ago, particularly in finished products; but for two months past the volume has been up to the average. The selling prices of bar-iron have been the highest for twenty years, as was proven by the adjustment of wages effected last week at Pittsburg between the Republic Iron and Steel Company and the Amalgamated Association of Iron, Steel, and Tin-Workers. According to the selling prices of iron and steel bars during July and August, wages were advanced to 6.62½ dols. for puddling, the average selling price being 1.68 dols. per 100 lb. Quite a number of orders for 500 and 1000-ton lots have been placed recently, and more are in sight. The price of sheet-bars has been reaffirmed at 31 dols. per ton for the fourth quarter by the Carnegie Company's orders for structural material, but most of this week's orders are small—that is, near 1000 tons each. A great deal of public building is being done, and this calls for shapes. The Carnegie Company has bought about 100,000 tons of Bessemer and open-hearth billets since April 1 from outside mills, and has orders out for large additional supplies, due very soon. The pig-iron market continues very quiet in all sections. At the same time a general buying movement is believed to be near at hand. This is because a good many large consumers have been putting off buying until the depressing influences would have had plenty of time to mature. At present a number of furnaces are still idle, and they will remain idle until the situation fully warrants resumption. The furnace interests intend to prevent any slump in pig, and are able to do it. It is barely possible this policy may precipitate an upward tendency in prices. It appears to be a cardinal doctrine among American pig-iron makers to avoid anything like an accumulation of pig iron for emergencies. They prefer to erect sufficient producing capacity to fill demands promptly, and blow out on the first signs of a weakening market.

Tenders Invited.—The *Gaceta de Madrid* recently contained a notice to the effect that tenders are invited for the construction of certain dock works of the Port of Cadiz. Tenders should be sent in by October 26 to the Junta de Obras del puerto de Cadiz, Cadiz. The *Gaceta*, containing a full description of the works, may be seen at the Commercial Intelligence Branch of the Board of Trade, 73, Basinghall-street, E.C.

The late Mr. Joseph Wilkes.—We regret to record the death at the age of eighty-two of Mr. Joseph Wilkes, until recently proprietor of the Pelsall Foundry, Walsall. Mr. Wilkes commenced his career at the works of Messrs. Wright and Co., Goscote Foundry, near Walsall, where his father was foundry manager. In 1853 he commenced business in a small way on his own account, and subsequently turned his attention to the production of railway material and constructional plant. On the development of coal-mining in the Midlands, Mr. Wilkes enlarged the sphere of his work by taking up the manufacture of colliery plant, winding-engines, pumps, &c., and, latterly, still further increased his business by supplying equipment for brick and tile works, quarries, mills, &c. He retired from active business five years ago, the works being handed over to his son, Mr. Ernest Wilkes, J.P., their present proprietor.

THE CROCCO AND RICALDONI HYDROPLANE BOAT.

We illustrate on page 462, in Figs. 1 to 4, and in Fig. 5 below, a new hydroplane motor-boat, designed by MM. A. Crocco and O. Ricaldoni, of the Brigata Specialisti, in Rome. The little vessel has been built in the yard of M. Baglietto, at Varazze, on the Gulf of Genoa, and has been run on the Lake of Bracciano, near Rome.

The length of the boat is 8 metres (26 ft. 3 in.), and she is fitted with a Clément-Bayard 80-100-horse-power motor, having cylinders 180 millimetres by 180 millimetres (7.09 in. by 7.09 in.), and working at a speed of 1200 revolutions per minute. It will be seen from the views we publish that the boat is provided with hydroplanes only at its stem and stern. The planes at the bow are arranged in the manner of a V, while those aft, though similarly disposed, do not join at the inverted apex. These planes, and the principal members of the frames supporting them, are made of steel plating, the remaining parts of the carrier frames being of aluminium. Their arrangement is clearly seen in Fig. 1, which gives a broadside view of the boat, suspended with its stem towards the right hand. Fig. 3 shows an end-on view of the vessel showing the low V-shaped plane, while Fig. 4 shows the arrangement at the stern. Fig. 2 also shows the stern of the boat, and this, with the other views, gives a good idea of the arrangement of the aerial propellers. These propellers are of doubled aluminium plating, and weigh each about 12 kilogrammes (26.4 lb.). Their pitch can be altered while running and they can be reversed. The propellers are mounted on frames of aluminium sheeting, which, together with the shafts, gear, transmission, and controlling devices, &c., weigh 300 kilogrammes (660 lb.). The weight of the motor is also 300 kilogrammes (660 lb.). Including all machinery, fuel, &c., and two men on board, the vessel weighs 1500 kilogrammes (3300 lb.). When running the boat rises, so that the hull is clear of the water, and at the speed of 70 kilometres per hour (43.5 miles per hour) which has been obtained with this novel form of vessel, the hull is about 18 in. out of the water. Fig. 5 shows the boat at full speed, supported solely by the V-shaped planes, the hull being clear of the water as described.

We are informed by the inventors of this novel type of boat that on commencing a run, when a speed of about 10 kilometres (6.2 miles) per hour is attained, the bows begin to lift in the water, and the fore fins slowly emerge as the speed increases. At a speed of 25 kilometres (15.5 miles) per hour the hull is wholly out of the water, only the flat portion near the stern skimming on the surface. At from 30 to 35 kilometres (18.6 to 21.7 miles) the boat is supported solely by the V-shaped planes; and at the highest speeds yet attained the hull is, as we have already stated, 18 in. out of the water. It has been found that waves of a height of 20 centimetres (7.87 in.) do not affect the vessel, as at the high speeds the hull stands quite clear of the tops of waves of this size. Trials of a length of 6 kilometres (3.73 miles) have been run, and sharp turns have been taken while running. After a certain amount of further experimental work, the inventors propose to put the boat through still more exhaustive trials, under as varied conditions as possible.

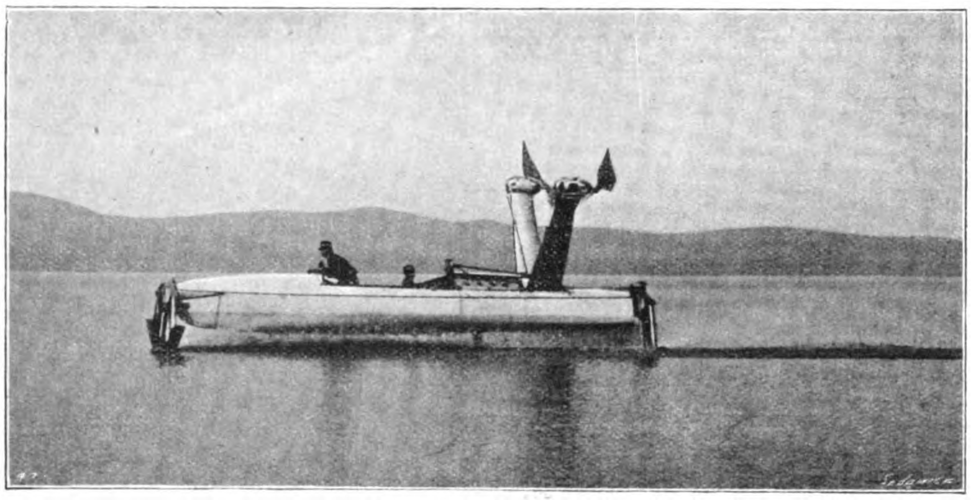

Fig. 5.

THE QUEBEC BRIDGE FAILURE.

To the Editor of Engineering.

Sir,—In your issue of the 6th ult. you suggest that the portion of the Quebec Bridge which fell may have been swayed by a wind blowing in periodic puffs, which synchronised with the period of vibration of the overhanging steel.

Whether the wind ever really blows in such puffs may be doubted, but the following incident shows that a very moderate wind, when acting on an extended area, may exercise the effect which might be expected from such puffs. Rather more than three years ago, during the building of a suspension bridge over the River Mersey, on which I was resident engineer, the cables had been erected, and hung practically free from tower to tower. The span was 1000 ft., and the average height above water about 120 ft., with a dip of 76 ft. The weight of each cable, from tower to tower, was over 90 tons. One day, although only a very moderate wind was blowing, the western cable vibrated violently longitudinally. The amplitude of the vibration at the quarter span was, as near as I could judge, 10 ft., and the period 4 seconds. After some hours the vibrations died down. On another occasion the western cable was still whilst the eastern cable vibrated. These vibrations were only noticeable on these two occasions, although the cables were often subjected to heavier winds. As the diameter of the cables was 10½ in., the area exposed to wind pressure per ton of weight was by no means excessive.

Yours faithfully,

L. H. Chase, M. Inst. C.E.

67, Dale-street, Liverpool.

STEAM AND ELECTRIC TRACTION ON THE SAME TRACKS.

To the Editor of Engineering.

Sir,—Mr. Carlile's letter on this subject, appearing in your issue of September 20, has only just come under my notice, and as no reply has so far appeared, I send a few remarks which may be of interest.

I believe there are several places in the United States and in Germany where steam trains work over tracks equipped with overhead conductors for electric traction; but the best known and most remarkable instance occurs at Baltimore, where electric locomotives are used for hauling steam trains with the locomotive attached through the tunnel sections of the Baltimore and Ohio Railroad. The electric locomotives work for about three miles, and in places the steam locomotives work as well. The traction only occurs in one direction, the steam locomotives working the trains alone on the long down grade the other way, usually without steam. The earlier electric locomotives were large eight-wheeled four-motor engines, weighing about 88 tons; but the later ones consist of two complete units, usually worked in permanently connected pairs. With passenger trains the electric locomotive is coupled in front of the steam locomotive, but with freight trains it generally pushes behind.

The overhead conductor is a reversed iron conduit in which a shuttle-like shoe carried by parallelogram arms runs, but where possible it is placed to one side of the centre of the track. This working has been in force for seven or eight years, but the writer has no information as to the detrimental effect of the smoke and fumes of the steam locomotives upon the conductor rails.

The Stockholm-Rimbo Railway is a small line in Sweden usually worked by electric-motor cars, but on Sundays and holidays the power-station is not, or was not, sufficiently large to supply current for the increased service, and therefore on those occasions several steam-trains were run as well. It is thought, however, that this working is now discontinued, the power-station having been enlarged. In this case the overhead conductor was of the ordinary wire type.

I shall be pleased to give additional particulars of these instances of combined steam and electric working. There are probably many cases where steam locomotives work over sections of overhead-equipped track.

Yours faithfully,

J. F. Gairns.

32, Bromley-street, London, E., September 30, 1907.

The Sheppee Motor Company's Feed-Pump: Erratum.—It should be pointed out that the piston of the feed-pump constructed by the Sheppee Motor Company, York, and described on page 436 of our last week's issue, is fitted with rings, instead of plain grooves, as was then inadvertently stated.

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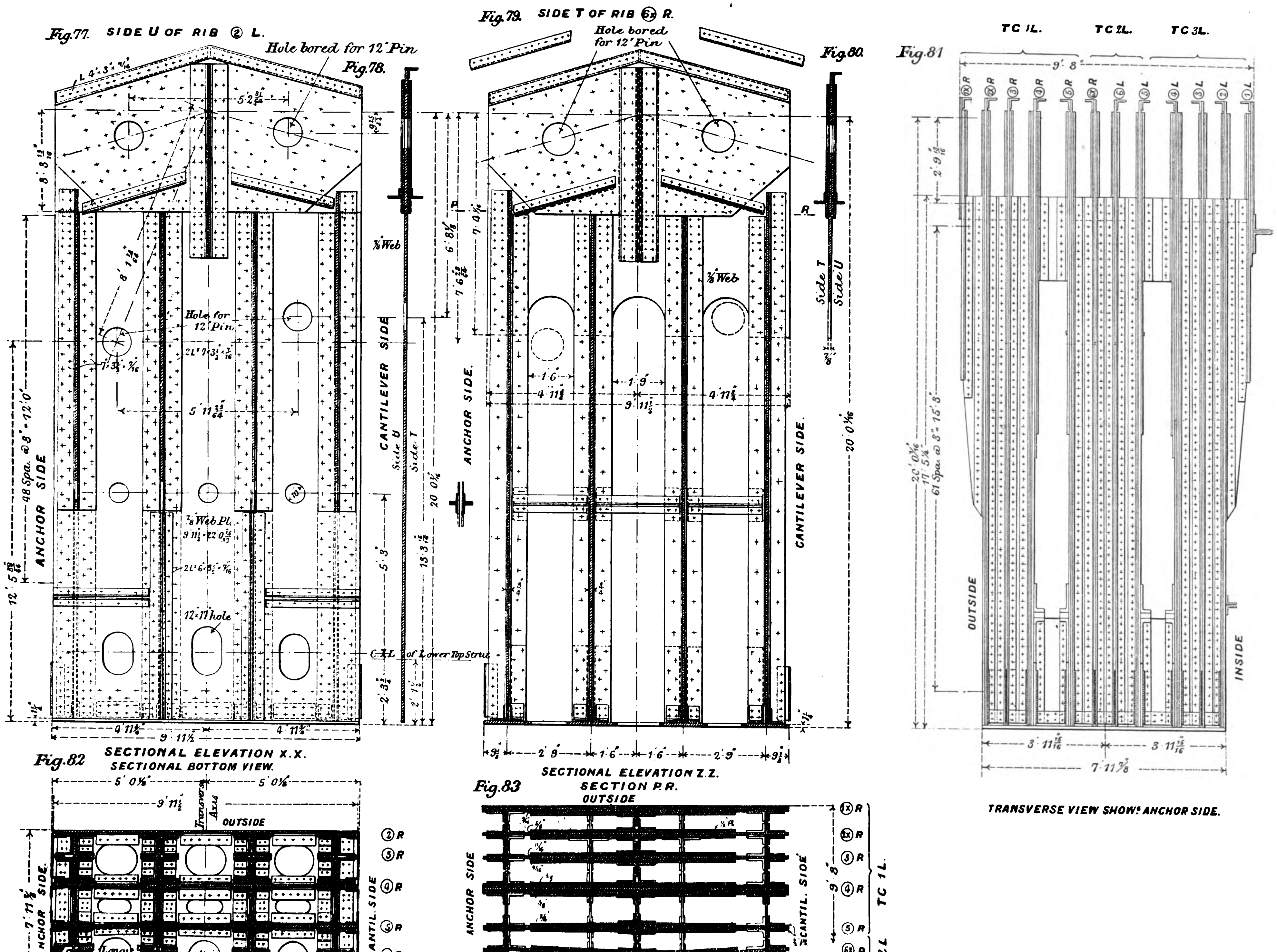
THE CANTILEVER BRIDGE ACROSS THE ST. LAWRENCE AT QUEBEC, CANADA.

THE PHŒNIX BRIDGE COMPANY, PHŒNIXVILLE, PA., U.S.A.

(For Description, see Page 449.)



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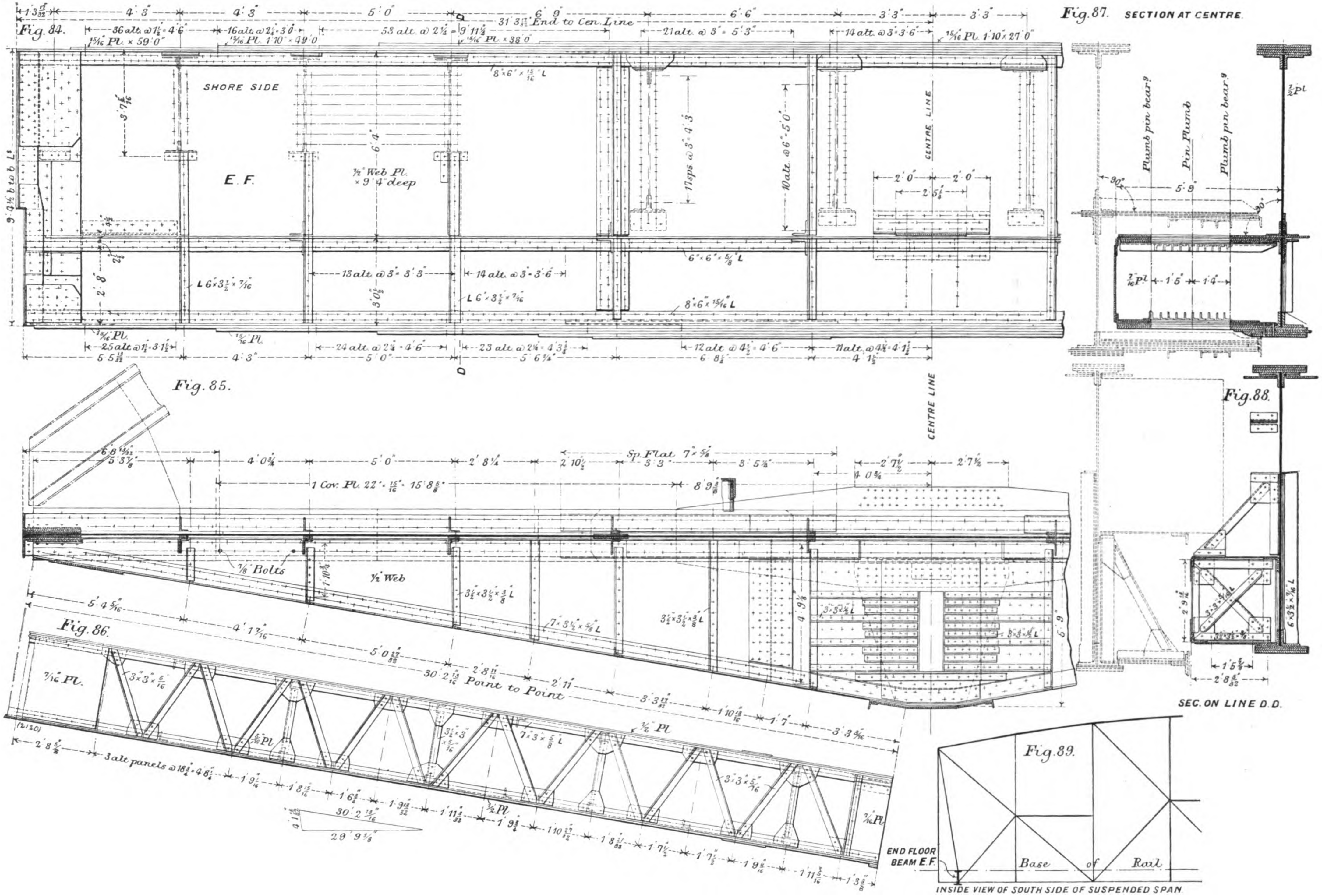
Figs. 77 to 83. Top Sections of the Towers over the Main Piers.



Figs. 84 to 89. End Transverse Girders of Suspended Span.

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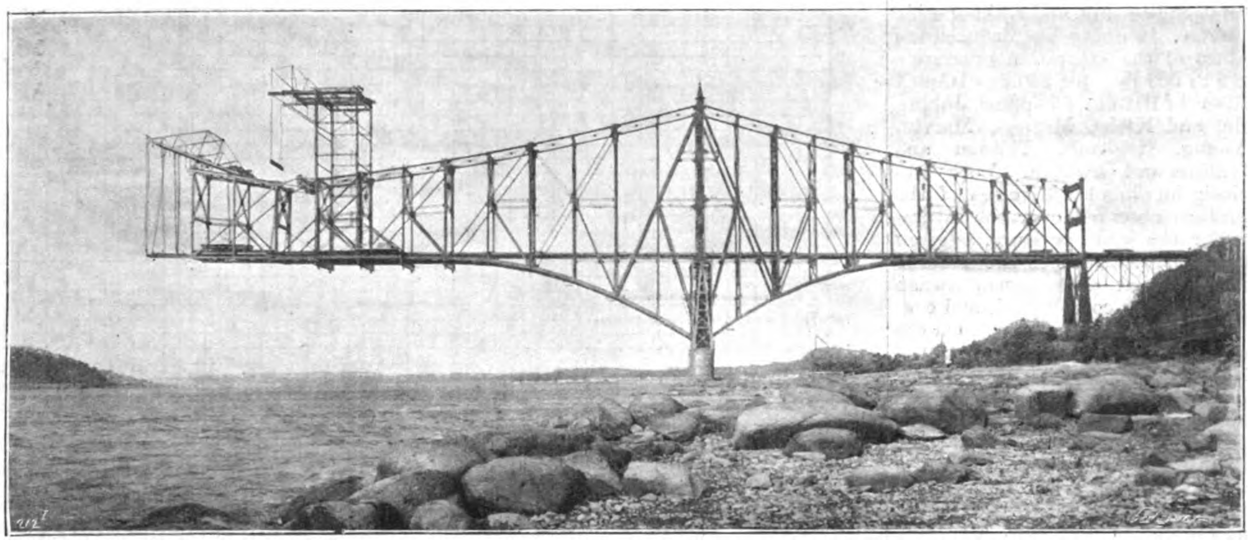
THE QUEBEC BRIDGE; AUGUST 14, 1907.



WE reproduce above a photograph taken of the Quebec Bridge on August 14 last, just fifteen days before the final catastrophe. The view shows the river arm finished, and the completion of the first two bays of the central suspended span. The great 1000-ton traveller is to be seen near the end of the cantilever-arm. This traveller was being dismounted at the moment of the disaster, and was to have been transported and re-erected on the other side of the river, and used to construct there the shore and river cantilevers. A much lighter traveller was being used for erecting the suspended span. This was designed to be supported by the upper chord of the truss in course of erection, and is shown in position at the extremity of the overhanging arm in our illustration.

SCOTCH RAILWAY PROPERTY.

IT cannot be denied that the position of Scotch railway property is less encouraging now than it was two or three years ago. Ordinary stock dividends have, upon the whole, been declining; and in spite of this, it appears to be impossible to check further outlay on capital account. The dividend upon the ordinary stock of the Caledonian Railway, for instance, has declined during the past twelve months from 4 per cent. per annum to 3½ per cent. per annum; while a further expenditure of 232,746*l*. was made on capital account in the six months ended July 31. Of the 232,746*l*. expended, 146,934*l*. related to outlays upon lines previously opened for traffic.

The North British Railway, again, has had to reduce the dividend for the past twelve months upon its ordinary stock to 15*s*. per cent. per annum, as compared with 1*l*. 15*s*. per cent. per annum in 1905-6, while the further expenditure on capital account for the half-year ended July 31 was 386,935*l*. The only redeeming feature in the continued North British Railway expenditure on capital account is the fact that of the 386,935*l*. expended in the past half-year, 217,656*l*. related to additional working stock. The Glasgow and South-Western Railway has had to bring its ordinary-stock dividend down from 4¼ per cent. per annum to 4 per cent. per annum, while its further expenditure on capital account during the six months ended July 31 was 84,031*l*., in which additional working stock figured for 40,493*l*. The Great North of Scotland and the Highland Railways appear to be guarding more strictly against further capital expenditure, and the capital outlay made by the Great North Railway in the past half-year did not exceed 9397*l*., while that of the Highland Railway came out at 20,457*l*. The ordinary-stock dividend of the Great North Railway has been maintained for the last two years at 1 per cent. per annum, and that of the Highland Railway at 1¾ per cent. per annum, a similar distribution having been made for 1905-6.

The five companies own between them practically the whole of the railways of Scotland, and it will be seen that the average return upon their ordinary capital for the twelve months ending July 31 was 2*l*. 4*s*. per cent. per annum, as compared with 2*l*. 11*s*. per cent. per annum in the year ending July 31, 1906. The progress of the capital accounts of the five companies—that is, the increased outlay of capital made by them during the past half-year—was 733,566*l*., in which companies associated with Glasgow figured for 703,712*l*. These Glasgow companies have always exhibited a tendency to competition, and have been continually constructing additional lines. The Great North and the Highland Railway Companies have, on the other hand, practically "buried the hatchet," and have concluded that their best course is to rely upon the tourist-attracting grandeur of Scotch scenery, pure and simple.

It is not surprising to find that stockholders in the Glasgow companies are becoming alarmed at the outlook, their alarm being reflected in something of a disturbance at the recent half-yearly meeting of the Caledonian Railway Company, although the directors, with the help of proxies, were able to maintain themselves in office. But they did not succeed in doing this without the issue of tranquillising circulars, and promises of increased future economy, from which possibly some good may result. No doubt it is impossible to apply to railway business in the West of Scotland rules similar to those which would hold good in the picturesque, but thinly inhabited, northern counties. At the same time, declining dividends must have some effect upon the bolder railway enterprise of Glasgow capitalists. It is right to remark, however, that one great difficulty with which Scotch railway companies have to contend is the marked increase in working expenses. Traffic receipts are by no means stationary, but working expenses appear to be growing more rapidly.

The receipts of the Caledonian Railway expanded from 2,200,017*l*. in the half-year ended July 31, 1906, to 2,251,044*l*. in the half-year ended July 31, 1907; but the working expenses advanced from 1,164,156*l*. to 1,219,507*l*. The revenue of the North British Railway grew from 2,360,651*l*. in the half-year ended July 31, 1906, to 2,440,109*l*. in the half-year ended July 31, 1907; on the other hand, the working expenses expanded from 1,247,528*l*. to 1,313,571*l*. The receipts of the Glasgow and South-Western Railway advanced from 902,515*l*. to 913,545*l*., while the working expenses rose from 517,108*l*. to 533,107*l*. The earnings of the Great North Railway rose from 247,668*l*. to 249,086*l*., while the working expenses grew from 120,390*l*. to 126,450*l*. The income of the Highland Railway in the six months ended July 31, 1906, was 255,231*l*., and in the half-year ended July 31, 1907, 262,808*l*., the working expenses rising from 148,195*l*. to 152,613*l*. The gross receipts of the five companies for the six months ended July 31, 1907, were accordingly 6,116,592*l*., as compared with 5,966,082*l*., while the working expenses were 3,345,248*l*., as compared with 3,197,377*l*. The North British Railway pursued the unusual course of charging its net revenue account with 12,000*l*. for a reserve for the repair and renewal of locomotives. When account is taken of this charge, it will be seen that the working expenses increased 159,871*l*., while the increase in revenue stood at only 150,510*l*. In other words, net profits were scarcely maintained in the past half year, while further capital was expended to the extent of 733,566*l*. At 4 per cent. per annum the additions to capital loaded the five companies, and especially the first three of them, with an additional annual interest charge of about 58,000*l*., while net revenue, as we have just shown, has been scarcely maintained. This, of course, accounts for the decline of 7*s*. per cent. per annum noted in the average ordinary-stock dividend.

The increase in working expenses is largely explained by the increased cost of locomotive power. Upon the Caledonian Railway locomotive power cost in the past six months 392,804*l*., as compared with 343,359*l*.; upon the North British Railway, 386,444*l*., as compared with 350,291*l*.; upon the Glasgow and South-Western Railway, 156,503*l*., as compared with 144,505*l*.; upon the Great North Railway, 43,753*l*., as compared with 39,868*l*.; and upon the Highland Railway, 24,209*l*., as compared with 19,410*l*. It will be seen that the increase in the cost of locomotive power has been universal, and it has been mainly occasioned by the higher cost of coal. Advances in wages also account largely for the growth of Scotch working charges.

THE LATE MR. DAVID CARLAW, GLASGOW.

THE death of Mr. David Carlaw at his summer residence at Lochview, Craigmore, as the result of a paralytic seizure, removes an engineer who did much to raise the standard of precision in work, especially in model and instrument-making. Mr. Carlaw, who was in his 75th year, founded the business of Messrs. David Carlaw and Sons, mechanical engineers and machine-makers, Glasgow, and having natural mechanical ability, with that faculty for taking infinite pains which is akin to genius, he won wide recognition for exceedingly fine small work, and he brought the firm to the first rank in several departments of mechanics. Many improvements in printing and stationery machinery are also due to his inventive genius. He was trained as an optician, serving first with a Glasgow firm, and later with Messrs. Hommersley, of London. His next appointment was with the late Mr. James White, who was for so long associated with Lord Kelvin in the production of compasses and other instruments. Carlaw went to White's works at the time when Professor William Thomson (as he then was) was making his instruments in connection with the laying of the Atlantic cable, and as a mechanic Carlaw did for him very useful service. About the same time also he was employed in making and fitting up the instruments in Kelvin's laboratories at Glasgow University, and when one recalls the great amount of valuable experimental research which has been conducted in this laboratory, with so much accuracy and profit, one recognises the value of Mr. Carlaw's precision work.

Mr. Carlaw commenced business for himself in 1860, making from the first a special feature of instruments corresponding to those which he had constructed for Lord Kelvin—theodolites, surveying instruments, compasses, spirit-levels, &c. His inventive mind found a further outlet in the design of printing and stationery machinery, and amongst his inventions was a paging and a numbering machine, envelope-making machine, &c. These have, from time to time, been dealt with in ENGINEERING in connection with exhibitions, notably with the Glasgow Exhibitions of 1888 and 1901.

In 1860 Mr. Carlaw's growing business necessitated new works, and a few years later further developments took place, and his three sons became partners. The works were still further extended, and at his works at Finnieston-street, Glasgow, he manufactured machinery for the production of bank-notes, cheque-books, numbered tickets, &c. A separate business was founded by him for the manufacture of models, principally of machinery, and many of the instruments constructed by him, notably the models of Brock's quadruple expansion engine and of locomotives, were, as we have already said, sources of attraction at many exhibitions. Mr. Carlaw took no part in public affairs, and found recreation in horticulture. He enjoyed wonderfully good health, and maintained all his genial and kindly characteristics to the end, and found pleasure in his later years in watching the development of his business; but on the 12th ult. he had a paralytic seizure, and although he rallied a little from time to time, he lost consciousness on the 19th and died on September 20, in the seventy-fifth year of his age, leaving a widow and five sons and five daughters, who have the sympathy of many professional friends.

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the headstock spindle to its original position for commencing work on succeeding articles. A thread is cut at one traverse, at the termination of which an automatic trip stops the machine instantly.

The milling-cutters are carried on the cross-slide, which revolves on the saddle, and which is traversed by hand-wheel, rack, and pinion. The direction of rotation of the cutters is reversed by a lever behind the saddle. The machine is driven by fast and loose pulleys at constant speed from the line-shaft, and the shaft which carries the pulleys at one end extends along the rear of the machine, and actuates the headstock spindle. The sequence of operations is as follows :—1. Fix work in chuck. 2. Feed in milling-cutter to the depth required. 3. Throw spindle-clutch into action to revolve the spindle. 4. When the machine has been stopped automatically by means of the trip, the milling-cutter is withdrawn, and the threading-cutter is brought into position by revolving the top of the cross-slide. 5. The nut is thrown into engagement with the hob, and the threading-cutter fed in to the depth required. 6. The spindle-clutch is thrown into gear to revolve the work. 7. After the machine has automatically stopped, the cutter is withdrawn and the work taken from the chuck. A pump and pipes are fitted, and there is a convenient tray for laying work upon.

In an exhibition where high-speed cutting is so much in evidence we naturally look for specimens of high-speed tool-steels. But the exhibits are not as representative as they might have been. A good stand is that of Messrs. Samuel Osborn and Co., Limited, of Sheffield, devoted to Mushet high-speed steel, the capabilities of which have been increased since the advent of the new steels. Adjacent is the stand of Messrs. J. Butler and Co., to which reference has been made (September 27), and where the Mushet steel is seen in operation turning and drilling. Twist-drills of Mushet steel, ⅝ in. in diameter, running at 500 revolutions per minute, will penetrate mild steel at the rate of 8 in. per minute, and cast iron at 17½ in. per minute. Drills 2 in. in diameter, running at 160 revolutions per minute, will penetrate 28-ton steel at 2⅝ in. per minute ; and, drilling at speeds even greater, was being continually shown to visitors on one of Messrs. Butler's machines. The drills, the smaller of which were hand-ground, appeared in no way affected. A relic of interest to metallurgists is a piece of ferro-maganese believed to be a portion of that with which the late Robert F. Mushet experimented in the Bessemer process. It was given to the late Samuel Osborn by E. F. Mushet in 1858.

The capabilities of Mushet steel are seen in a large collection of milling and other cutters, on Messrs. Osborn's stand, as well as the single-edged cutting tools for lathe, planer, &c., which have so long enjoyed a high reputation in the machine-shops and turneries. Always when a piece of especially tough cutting has had to be done, resort has been made to the Mushet tools. The high-speed milling-cutters were, however, unknown a few years ago. Among these is a new design of edge-mill (Anderson's), the feature of which is that it is built up in a series of rings with diagonal teeth, which hit and miss. It is therefore a staggered cutter with a shearing action, but with the advantage that should teeth break, any ring so damaged can be removed and replaced by a new one. Also the rings are built on a mandrel held between bevelled washers, which impart a lateral oscillating motion to the cutters when rotating. It is claimed that three times the amount of material can be removed with these by comparison with an ordinary cutter.

A few massive models of single-edged tools are shown painted to indicate the degrees of orange or red to which they should be heated for hardening. There are also on Messrs. Osborn's stand some interesting specimens of the silky fracture of Mushet high-speed steel. The Mushet high-speed steel is produced on similar lines to the older self-hardening steel of the same name. Turning tools of high-speed Mushet should be hardened by heating the nose to a welding heat, after which they are cooled in a blast of compressed air at a pressure of about 60 lb. per square inch, or a bath of whale-oil may be used. It is advised that tools, after forging, should be left on the hearth for a night to anneal and take out forging stresses. A water-hardening steel, the "Titanic" brand, is shown, with the aspect of fractures at different temperatures, and also in different stages of its manufacture. Steel castings and crucible cast-steel files are also much in evidence.

Another interesting exhibit of high-speed steel is that of Messrs. Sanderson Bros. and Newbould, Limited, whose "Saben" brand is seen cutting on the heaviest high-speed lathe in the building, by Messrs. John Stirk and Sons, which we shall illustrate later. "Saben" steel drills are also operating in a drilling-machine by the same firm. A fine collection of tools is shown, including cold saws with inserted teeth. The range in manufactured qualities is very large, and, as extremes, are shown razors and a punch, as examples of what may be done with the steel. The punch shown has punched thirteen locomotive frame-plates 1$\frac{1}{16}$ in. thick. A cold-sawing machine by Messrs. Clifton and Waddell, of Johnstone, is located at this stand, fitted with a 30-in. saw of "Saben" steel. It cuts at 80 ft. per minute, with a feed of 2 in., and severs a joist 6 in. by 5 in. in 4 minutes. The feeds, of from ½ in. to 2 in., are obtained through the Sellers type of friction-discs.

Without making comparisons between the products of different firms, the visitor cannot avoid noting how manufacturers fall into the practice of working on identical problems. After the phenomenal growth of the spur and spiral gear-wheel hobbers, the most interesting feature is the growing tendency to the abandonment of the countershaft. In more than a score of instances the single-belt pulley drive at constant speed has taken its place, all changes in speeds being made through gears, with or without auxiliary mechanisms. The advantages are great, not merely because belt-shifting is avoided, but because such machines are equally well adapted to be driven by a motor or from a line shaft. In a period of transition like the present this convenience appeals powerfully to many of the older firms, where belt-driving is being changed gradually for electric.

No one can doubt that the days of belt-driving from the countershaft are numbered. One only has to see the ease with which motors at constant or variable speeds are connected and operated, and the convenience of the variable switch, to understand the great advantages which these have over countershafts and belts. These advantages are much in evidence at Olympia.

Another impression is, how much the electric drive and high-speed work have influenced the manufacture of gear-wheels. Cast gears would be an anomaly in these great all-geared headstocks, which are ubiquitous. All are cut, many in cast steel or in forged steel, while gears of raw hide are common on the first-motion shaft from the motor. Further, generating gear-cutting machines far outnumber the form gear-cutters, and the rotary cutter-machines, which are but slightly represented. Spurs and spirals are being generated by hobbing, bevels by planing, and worm-wheels on the Wallwork machine.

THE QUEBEC BRIDGE.*

By Frank W. Skinner, M. Am. Soc. C.E.

(Continued from page 456.)

Main Posts.—Each of the main-pier vertical posts had five 4-ft by 10-ft. sections, with lengths and weights up to 77 ft. and 80 tons respectively, which were handled in erection by a pair of 55-ton tackles, one at each end, suspended from the main girders of the traveller overhang in the planes of the trusses. A box girder, transverse to the axis of the post, was bolted through end-flange angles to the jaw-plates in the splice at the upper end of the post section, and a 12-in. by ⅞-in. U-plate engaged a pin through the centre of the girder webs, parallel to the jaw-plates. The U-plate was bored for a pin, at right angles to the girder pin, which engaged the loop of a bent eye-bar and two straight eye-bars. The bent eye-bar virtually acted as a shackle, providing connection with the tackle-plates, and the straight eye-bars formed a long pivoted link for the shackle of the fleeting-tackle.

The tackle at the lower end of the post section was shackled to a pair of standard triangular tackle-plates engaging a U-plate with an 8-in. pin-connection to the projecting ends of a pair of 15-in. channels, 11½ ft. long, bolted transversely across the post-flanges, and projecting far enough beyond the post to provide an offset pivoted connection enabling the tackles to swing into position as the post revolved from horizontal to vertical, and allow the tackle to assume its final parallel position alongside without fouling.

The 10-ft. by 10-ft. by 20-ft. upper section of the main post was shipped in three pieces, each having one pin-hole through the top and two pin-holes through the bottom. A pin in each hole engaged a pair of long outside links riveted at their upper ends to a double-web transverse horizontal girder, bored at the centre for the pin-connection to the tackle-plates and fleeting-tackles, thus forming a wide yoke free to revolve around the piece, and allow it to swing from a horizontal to a vertical position. The combined weight of the three pieces was 245,000 lb., and they were assembled and field-riveted in position nearly 400 ft. above the river. They are the highest portions of the trusses, and very little more clearance was left between them and the top of the traveller than was necessary for the accommodation of the great tackles when the blocks of the latter were drawn up close together.

The 18-ft. by 20-ft. 160,000 lb. bolster for the main post was handled by two tackles attached to pins through the projecting ends of horizontal double-web beams, 19 ft. long, which passed between their webs and took bearing against the cap-plate. The 156,000-lb. pedestals were handled by tackles attached to vertical channels engaging a small pin through the 24-in. pin-hole.

Bottom Chord.—The 4½-ft. by 5½-ft. bottom chord sections had a maximum length and weight of 68 ft. and 210,000 lb. respectively, and were handled with connections similar to those described for the lower ends of the main posts, having double sets of U-plates at right angles to each other, engaging the regular chord-pins and permitting the members to swing freely in vertical planes to assume their final inclined positions and to be simultaneously drifted transversely to position by the fleeting-tackles.

Floor-Beams.—The 32-ton floor-beams, 10 ft. deep and 67 ft. long, were handled by double pairs of riveted hooks, 5 ft. apart, which engaged the undersides of their top flanges, 22 in. wide. One hook of each pair was forged from a 6-in. by 2-in. bar, and was pivoted between the two 6-in. by 1-in. bars, riveted together with fillers to form the opposite hook. The points of the hooks were grooved to engage the heads of the flange rivets and prevent slipping, and both hooks were pivoted to a 2$\frac{11}{16}$-in. pin, engaging a U-plate connecting it to a transverse pin through one end of the lifting beam, made with a pair of channels back to back. A 4½-in. pin through the centre of the beam-webs received an eye-bolt swivelled to the tackle-plates, so that the floor-beam was free to be revolved about a vertical axis, and could be loaded parallel with the bridge axis and swung to its transverse position without changing the hitch.

Eye-Bars.—Most of the tension members of the trusses were composed of eye-bars up to 16 in. by 2⅛ in. by 76 ft., in all 3302, weighing about 7000 tons. There were as many as twenty-eight bars in a single panel of top chord, and owing to their great length it would have been a very long and difficult process to assemble them singly on the 12-in. pins, 7 or 8 ft. long, some of which received as many as sixty different pieces with less than ⅛ in. clearance for each. Devices were therefore prepared for assembling and hoisting all the eye-bars in one member as a single unit, thus adding much to their rigidity and to the accuracy and rapidity of erection, and making the connections like those of rigid members with multiple interlocking jaws.

A set of eye-bars were assembled together in proper sequence, in vertical planes on horizontal skids in the storage yard, spaced exactly the required distances apart, with wooden fillers, and clamped tightly by horizontal transverse steel plates on opposite edges, connected by screw-rods, the latter being set in the interstices between the bars, and being slightly oblique to the axis of the bars, so that the skew tended to tighten the plates. Numerous holes through the clamp-plates received stud-bolts, which were screwed up against ½-in. bearing-plates on the upper edges of the bars.

Double-web transverse lifting-beams were set on the lower edges of the bars between the clamp-plates and the filler-yokes, with vertical plates parallel to the eye-bars bolted to angle connections on the clamp-plates, to hold the bearing-beams

* [This article was, of course, written prior to the occurrence of the grave disaster with which we had occasion to deal in our issues of September 6 and 13. As a complete account of a most important work, however, it has lost none of its interest, and a study of the information which it contains will add much to the understanding of the inquiry now being made into the causes of the failure.—Ed. E.]

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against slipping longitudinally. The webs of the lifting beams were pin-connected to short vertical eye-bars, with their upper ends engaging a pin through a short riveted box, with two side-plates parallel to the pin, projecting beyond the latter to receive a transverse pin engaging the tackle-plates. Sometimes sets of top-chord eye-bars were provided with pairs of transverse wooden beams on the under side, having vertical U-bolts, which passed between the eye-bars and the beams, with bearing nuts on the latter; 32-ton tackles were attached to the U-bar loops, and set up to take the weight of the eye-bars, thus releasing the 55-ton tackles and allowing the steel lifting-beams to be removed and used for other members after the eye-bars were hoisted to position, and were maintained there until permanently connected.

Erection of the Superstructure.—The short deck approach span at the south end of the bridge was erected on wooden false-work by an overhead wooden traveller, which had a low rectangular tower with trussed horizontal booms in front overhanging two panels, and supporting the hoisting-tackles, which were counterbalanced by ballast on upper platforms at the rear of the tower. After the completion of this span its traveller and false-work were removed, and the false-work traveller was erected on it, with its three lines of wheels engaging a single rail in the axis of the bridge, and one rail of each of the service tracks. The traveller was thus supported entirely by the wooden false-work, which it could erect two bents in advance, and move forward on to before erecting the steel false-work with tackles suspended from its transverse overhangs.

All panel-points and levels were located from the centre of the east pedestal on the south main pier, which was taken as the origin of co-ordinates. A distance of 67 ft. was measured on top of the pier in a line at right angles to the bridge axis, and determined the centre of the opposite pedestal; and 550 ft. measured south from each of these points, in lines parallel with the bridge axis, located the end panel-points of the anchor-arm trusses. These measurements were repeated and the locations checked by direct measurement between the anchor-pier panel-points, all being made with a 100-ft. steel tape, with tension spring and temperature adjustment.

The only points of the superstructure not subject to perceptible displacement during or after erection were the feet of the main posts on the river-piers. The trusses were so designed that if the bottom chord panel-points were properly located during erection, the other panel-points would fall in the right positions for the anchor-arm trusses; and very careful computations were therefore made for their displacement from camber, distortion from stresses in the members, variations of temperature and possible settlement of the false-work; the algebraic sums of the displacements were plotted on a large-scale diagram, and the corrected positions of the lower chord pins, and their final positions in the completed structure, were shown. The cumulative effect was to throw the end lower chord pin $7\frac{1}{32}$ in. south and $1\frac{1}{16}$ in. below its final position on top of the pier-tower, the upper end of the main vertical post being correspondingly deflected about 15 in. south.

Erection of South Anchor-Arm.

The erection of the tower-bent on the south anchor-pier was commenced on July 22, 1905; its top was inclined 4 in. to the south; the end lower-chord pins were accurately located in the calculated positions on it, and the bottom chords were erected from this point to the main pier by the steel traveller. The corresponding pieces for both trusses were carefully checked and inspected in the storage yard, turned to the right position, fitted up, any field rivets driven that could be advantageously driven there, and they were then loaded on flat steel cars, and drawn out on the false-work on two standard-gauge service tracks, one on each side of the bridge axis, between the planes of the trusses. The two chord sections were located as accurately as convenient longitudinally, and were connected at both ends by horizontal Manilla tackles, drawn up snug and made fast.

Connection appliances were bolted on at both ends, and tackles, suspended from the overhang of the traveller, were made fast and operated to lift them clear of the cars, after which the horizontal tackles were slacked off, allowing them to swing out into the planes of the trusses, where they were lowered away to rest on the camber-blocks on the false-work towers. The panel-points were temporarily set above or below their calculated positions to allow for the effect of the increasing dead-loads; but after three panels had been completely assembled, the first point was adjusted to its final position, and the successive points were adjusted to correspond, as the work advanced always two panels ahead.

There was a maximum difference of $5\frac{3}{8}$ in. between the adjusted erection height and the final height of the chord-pins, and the depth of the chords, and the inclination of some of them to the horizontal, caused this difference to produce triangular clearances between their abutting ends at splices. The planed ends of the sections were therefore in contact at the top or bottom, and diverged as much as $\frac{13}{16}$ in. at the opposite flange. All field-rivet holes in the splice-plates were filled with service bolts of diameters small enough to allow them to be inserted in the half-holes.

The bearing of each truss had a two-story 250,000-lb. bolster, which was bedded on the masonry, with a cushion, or filler, made with two thicknesses of heavy canvas, painted with two thick coats of pure linseed oil and red-lead. The bolsters were surmounted by the 13-ft. by 11-ft. by 8-ft. 156,000-lb. riveted pedestal, which engaged the vertical post shoe, and the special bent section connecting the lower chords of the anchor and cantilever-arm trusses across the pier.

After these bearings, each having a total weight of 565,000 lb., were assembled by tackles suspended from the traveller-overhang, the traveller returned to the shore-end of the false-work and commenced the erection of the other members of the trusses, which it completed, panel by panel; the field-spliced joints of the vertical and diagonal posts and horizontal struts being left partly open to allow the parts of the member to deflect enough to correspond with the displacement of the lower-chord panel-points, and enable the members to be connected in the triangular elements of the trusses without producing any initial stresses. In this manner all connections were easily made, and as the permanent-stress condition was gradually approached, the members assumed their final positions and dimensions, and the open joints closed themselves, although the bottom-chord splices were not riveted up until the cantilever arms were erected.

All the principal truss members were handled in pairs, one piece for each truss simultaneously, much as described for the lower-chord pieces, being first lifted vertically from the service cars, and then fleeted transversely by gravity into the planes of the trusses at a height sufficient to clear obstructions. The floor system, and all lateral, transverse, and sway-bracing were completely and permanently assembled before the traveller left the panel, and avoiding the necessity of repeating any of the operations or passing again over the same point after the main connections were made. The use of several auxiliary pins at the same panel-point very much facilitated the convenience and rapidity of erection, which progressed sometimes at the rate of 340 tons per day.

Generally all the eye-bars, up to 30, forming a single member, were clamped together as already described, and with their attachments and tackles made a 70-ton maximum unit, which was hoisted simultaneously with the corresponding one for the opposite truss to a height of about 200 ft. above the track, at a speed of about 12 ft. per minute, the engines being perfectly silent in their operation, and controlled entirely by signals from the foreman standing on the deck of the bridge or the traveller platform, and indicating by special gestures which tackle was to be operated, and how.

In the typical erection of the eye-bars for the second panel of the top chord from the river end of the anchor-arm trusses, the shore ends of the sets were at first permanently connected to the vertical posts, while the river ends were suspended from the traveller overhang by auxiliary tackles which released the 55-ton hoisting-tackles.

The top-chord bars in the river-end panel were divided for convenience in assembling between the post diaphragms, in groups of three, and the centre group of eight bars was first hoisted, and its lower end was inserted between the post diaphragms, and the pin started, but not driven through them.

The upper ends were lowered to rest on the main post diaphragms, and a transverse timber was put through the pin-holes. The outside group was hoisted in the same way, and the pins were driven. The diagonal eye-bars were assembled in the same way as the top-chord bars; and when they were made in two lengths, the intermediate pin was driven before they were hoisted.

Post-Caps.—The main post-caps were made in three sections, weighing 105,000 lb., 35,000 lb., and 105,000 lb. Each of them had four pin-holes, and was provided with three revolving yokes, to which the hoisting-tackles were attached. The inside member of the cap was first hoisted in a horizontal position to the required height, revolved to a vertical position, and temporarily seated on the bracket for the transverse strut, while the middle and outside pieces were hoisted and assembled, after which it was moved back and connected to them, and all were field-riveted together. In order to prevent the development of indeterminate and excessive stresses in the traveller, great care was taken to lower but one main truss member from the traveller overhang at once, all other members that had been previously hoisted being supported either by the truss or by fixed attachments to the traveller.

Driving Pins.—The main-chord pins, weighing up to about 4000 lb. each, were mostly 12 in. and 14 in. in diameter, and were provided with conical shouldered nuts, called "pilots," by which the different members engaging the same pin were readily drawn together, even when the holes did not register accurately, and the pins were easily driven in the usual manner by a battering-ram, which consisted of a steel bar several feet long, suspended horizontally by a long tackle attached to its centre of gravity, and swung back and forth, like a pendulum, by men controlling it by handle-bars projecting from its sides. This sufficed to drive 12-in. pins nearly 12 ft. long with as many as sixty bearings.

The men operating it were supported on an enclosed platform, called a "boat," suspended at a height of sometimes as much as 375 ft. above the water from the traveller overhang. A special hydraulic press was provided for driving the pins, but was not needed. When the principal members were hoisted, groups of men rode up on them, in readiness to guide their engagement with other members at panel-points, and to enter the splice-plates between jaws of riveted members, put in the service bolts, &c. A small force of men was kept constantly on duty on top of the traveller to attend to the lines and tackles there, and other gangs were kept busy aloft, on the lateral and transverse bracing, fitting up for the riveting gangs, building scaffolds, and painting.

Access was usually had to the top of the traveller by climbing up a stairs on the inclined end-post, and thence walking on the edges of the top chord eye-bars, the maximum inclination being so steep that it is difficult to make the ascent without rubber-soled shoes. Sometimes, however, the men climbed up on the lattice-bars of the vertical posts, or rode up on tackles, and often descended by standing on the lower block of a light Manilla tackle and letting it overhaul; or even sliding freely down on a hand line, taking care to wind it once around one leg and grip the loose end between the hollows of the feet. A maximum force of about 200 men was employed on the erection, and until the final catastrophe there were few serious accidents, although the work has been maintained during heavy wind-storms and in severe cold.

The work had been accurately built, and well inspected, so that very little fitting was required in the field, although none of the members were assembled together in the shops before shipment. Practically no drilling or reaming was required, except in a few cases where special holes or rivets were omitted until pieces having special plates were assembled in the storage yards. For such work pneumatic drills and reamers were provided, and operated by compressed air at 100 lb. pressure, which was carried to the yards and along the bridge axis by a 3-in. supply-pipe from the electrically-driven compressor plant, with a capacity of 2000 cubic feet of free air per minute. Each riveting gang comprised one heater and three other men, and drove an average of 250 $\frac{7}{8}$-in. and 1-in. rivets a day. The entire bridge and false-work involved an estimated number of only about 600,000 field-driven rivets.

(*To be continued.*)

More Copper.—Copper ore of the estimated value of 25,000,000*l*. has been discovered in the interior of Tarapaca.

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ments, and for all purposes it is taken that the two are directly proportional.

The results may be summarised as follows for engines working at variable revolutions per minute, with no explosions missed :—

1. No ordinary design of engine draws in a charge equal to the piston displacement when reduced to atmospheric pressure (temperature neglected). If temperature effect is included (an uncertain quantity and largely dependent on design and working of engine), its amount is considerably reduced when referred to atmospheric pressure and temperature. Great losses of power occur from insufficient valve area, or unsuitable valve-setting, but the former does not necessarily affect thermal efficiency upon I.H.-P.

2. The work done may be taken as directly proportional to charge admitted, its ratio being unchanged and point of ignition such as to give best results.

3. Engine friction increases very considerably with revolutions per minute, and, consequently, lowers both mechanical efficiency and thermal efficiency on brake horse-power.

The following deductions may also be made. From No. 1 it is evident that for this class of engine indicated horse-power is not directly proportional to explosions per minute or revolutions per minute alone, and owing to the way in which the explosive charge is steadily reduced at the higher speeds, it eventually reaches a critical maximum value. The maximum value for B.H.-P. occurs at a much lower speed, dependent upon the amount of engine friction. If for this engine the maximum indicated horse-power occurs at 350 revolutions per minute, the velocity of the entering gases is as high as 290 ft. per second, or three times too great. While this points to the necessity of ample valve area, it must be borne in mind that prevention of heating of the charge during admission is equally important. The continued risk of overheating in the case of air-cooled engines of from 3 to 4 horse-power, and their rapid falling off in power thereby, and refusal to work, emphasises this very strongly. No. 3 is important to bear in mind, for if an engine is required to develop its power by a high rate of speed, the losses occasioned by friction may seriously affect both the mechanical and thermal efficiencies.

Objection might be raised to the fact that all the foregoing results are derived from experiments upon one engine only; and while, for this reason, it would be unwise to attempt to generalise for all internal-combustion engines, yet many of the most important points have been noted and remarked upon by other experimenters on different engines. It is actually upon the fact that the work has been carried out on one engine—not, however, without collateral experiments upon others—that the author feels most confident. A few years' close study of one engine under every variety of test in the effort to bring normal and abnormal features strikingly home to young engineers in the making, has drawn to the author's attention an enormous number of points which, in the almost cursory test of an unfamiliar engine, would pass unheeded. Every dimension and every characteristic which seemed to influence results have been noted and investigated, and wherever opportunity has offered the deductions have been applied to other engines, to determine whether the results were confirmed or of a doubtful nature.

THE QUEBEC BRIDGE.*

By Frank W. Skinner, M. Am. Soc. C.E.

(*Concluded from page 485.*)

Erection of the Cantilever Arms.

The cantilever arms, 562½ ft. long and 315 ft. deep, each weighed 15,000,000 lb., exclusive of the floor system, having ten panels 56 ft. 3 in. long, and contained truss members of the maximum dimension of 105 ft. long and 100 tons weight. The south cantilever arm was erected during the summer of 1906, and it was expected that the north anchor arm would be erected during the summer of 1909, in the same manner.

* [This article was, of course, written prior to the occurrence of the grave disaster with which we had occasion to deal in our issues of September 6 and 13. As a complete account of a most important work, however, it has lost none of its interest, and a study of the information which it contains will add much to the understanding of the inquiry now being made into the causes of the failure.—Ed. E.]

The russ members were designed with sectional areas and connections proportioned not only for the maximum working stresses of the finished structure in service, but for the temporary erection stresses, sometimes of reverse quality, which, however, were allowed higher unit values, and were algebraically combined with those of the finished structure. No temporary reinforcements were used, and the reactions were all provided for by the anchor-arm trusses and their anchorage.

Suspended False-Work.

The south cantilever arm was erected by the same great gantry traveller, 212 ft. high, with a 66-ft. forward overhead overhang, which, with its equipment, weighs 2,225,000 lb., and previously erected the anchor-arm, moving, at that time, as already described, on two rows of steel towers, forming separate false-work viaducts outside of each truss, and each supporting a broad-gauge track at permanent floor-level, on which the traveller moved with clearance above and outside the completed structure.

These tracks were continued beyond the shore-line, from the main pier to the end of the cantilever arm, by extending the three lines of longitudinal plate-girders in each with stringers borrowed from the permanent floor of the 675-ft. centre suspended span, and supported on false-work suspended from the completed portions of the cantilever trusses themselves. As only two panels of this false-work were actually in service at any given time, only four transverse bents were provided, and the rear bent was taken down, moved forward, and re-erected in front successively as the work advanced.

Each false-work bent consisted of a lower transverse girder cantilevering beyond both permanent trusses, and carrying outside of them two pairs of vertical columns, 12 ft. apart on centres, to support directly the longitudinal track-girders at floor-level. The transverse girders were duplicates, and only four of them were required, but the lengths of the vertical posts varied for every bent, and enough of them were provided to reach from the main pier to a point where the bottom chord was so near the floor-level that the traveller track-girders could be supported by blocking on the ends of the transverse girders.

Each transverse girder was 106 ft. long, 6 ft. deep, and 42 in. wide over all, and weighed 108,000 lb. It had two solid plate-webs, 38 in. apart, and was suspended from the bottom chord-pins by two pairs of eye-bars about 15 ft. long, which passed between the girder webs and engaged pins through three transverse diaphragms in a double-web bearing girder, 7 ft. long and 4 ft. deep, supporting its lower flange under the centre of each truss. The space between the top of the transverse girder and the inclined lower flanges of the bottom chords was filled by a riveted triangular box and shim-plates, adjusted to give a rigid connection.

As the false-work bents were moved forward from rear to front at the completion of each successive panel of the trusses, the longitudinal girders, with their bracing and track complete for each track, were disconnected, lifted up by the traveller tackles, fleeted inside the trusses and moved back, and temporarily stored in the traveller tower until re-set in the front panels. The pairs of vertical posts were similarly hoisted inside the traveller and loaded on cars and sent back to the storage yard, to wait until required for the corresponding panel of the second cantilever arm.

Cantilever Traveller.

In order to have a receiving platform under the traveller overhang, for the delivery of material cars under the erecting-tackles, a temporary working platform was provided there for an extension of the material tracks brought out to the traveller on the permanent bridge-floor. A light riveted triangular longitudinal truss, 69½ ft. long and 38 ft. deep, with a horizontal bottom chord, was connected to the foot of each forward vertical post of the traveller tower, and the lower chords stiffened by horizontal inclined outriggers braced to them on the outer sides, to make triangular trusses 12 ft. wide.

From the forward ends of the cantilever trusses was suspended a transverse plate-girder about 80 ft. long over all and 6 ft. deep, with four vertical connection angles riveted to the girder web and diaphragms, or brackets, on the girder web and bolted to the chord webs. The top flange of this girder supported the drop-ends of four longitudinal girders or track-stringers, 5 ft. deep and 69½ ft. long, connected at their inner ends to the web of the forward transverse girder in the foot of the traveller tower.

These stringers were braced together in pairs, one for each track, and were easily removed by the traveller tackles to allow the assembling of the members of the permanent bridge-floor. Each track was long enough to receive one and a half flat cars, or six axles, each of which was limited to a live load, exclusive of the car weight, of 12,500 lb., or 29,000 lb., according to whether both tracks, or only one, were loaded. Care was taken to remove the cars as soon as their loads were hoisted clear.

Very complete instructions and diagrams, prepared in advance of the erection, and in conference with the designing engineers, were provided, giving all principal data to the erector, and instructing him in the details and sequence of all operations for handling, hoisting, assembling, and erecting all the principal members of the bridge. Sketches of the members, showing their principal dimensions, weight, and the special attachments required for the connections of the hoisting-tackles, and diagrams of the traveller, with the positions of all the principal tackles indicated by reference letters, were given, with explanatory notes on blue-print sheets for every panel of the cantilever arm. These fully described all operations at each position of the traveller, which was always placed with its forward vertical posts in the plane of the vertical posts of the last completed panel of the truss, and assembled the next panel complete in that position.

The first erection of the first panel beyond the main pier required thirty-eight different operations. The succeeding panels, being lighter and with fewer members, required somewhat fewer operations. In some cases the corresponding members of the same panel in opposite trusses were erected simultaneously; in the case of the top chords these involved the hoisting of combined loads of a total weight of about 160 tons, which were handled with ease and rapidity by the 125-horse-power hoisting-engines. All hoisting operations were directed by signals, each principal tackle being numbered, and the number repeated by the signalman with a corresponding number of motions, combined with some special gesture to indicate which tackle in a set. The work was thus executed in remarkable silence; and the rapid ascent and drifting or fleeting of the huge members, as large as long-span bridge girders, singly or in pairs, with groups of workmen riding on them 300 ft. above the water, ready to enter and bolt the connections, was very interesting and fascinating, and was accomplished with very few accidents; less, in fact, than usually attends erection work employing an equal number of men. Most of the men were professional American bridge-erectors, and were not at all disturbed by the great height.

The total force employed was about 200 men, and until the final catastrophe there had been but few fatalities attendant on the erection, one of them being occasioned by the carelessness of a man descending from the top of the traveller on a single line which was coiled around one leg and slipped between his hands and feet in the manner often adopted by erectors. He was unaware that the lower end of the rope was 50 ft. short of the platform, and it slipped through his hands, and he was fatally hurt by the fall.

A patrol-boat was constantly stationed under the traveller, or near it, to rescue anyone who might fall from it. Strangers were not allowed on the upper part of the bridge, and the writer was the first visitor to the top of the traveller. The ascent was made by climbing the end diagonal of the anchor-arm truss, and then walking on the edges of the top chord eye-bars, which in some panels were so steeply inclined that rubber-soled shoes were necessary to avoid slipping.

At the top of the main post, over 400 ft. above the water, the view of the landscape and the mighty river was superb. The platform on the top of the traveller was seen to be covered by a complicated system of tackle-supports, lead-lines, and guide-sheaves arranged to take all hoisting-ropes down outside the traveller clearance.

The tackles were attended by a special gang of traveller men, who usually brought their dinners up when they came in the morning and only went down at night. They often rode up on the lower blocks of the hoisting tackles, and usually descended by overhauling a Manilla tackle. The traveller was

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equipped with a total of about sixty tackles, including whip-lines, which were rove with about seven miles of steel-wire rope and thirteen miles of Manilla rope $1\frac{1}{2}$ in. and 2 in. in diameter, and in lengths of 2800 ft. The eight thirteen-part 55-ton steel-rope tackles were handled by the two drums on each of the four hoisting-engines, while their capstan heads handled the other tackles and lines.

Great care was taken to fill all field rivet-holes in the connections of the bridge members with service bolts until the field-rivets were driven. No serious vibrations were noted during the heavy storms to which the cantilever arm was exposed last winter, when the top of the traveller had an observed deflection of only 13 in. in the most violent gale encountered.

The 600,000 field-rivets, many of them 1 in. in diameter, were driven by special short pneumatic hammers, which, together with drills and reamers, were operated by air, at about 100 lb. pressure, from electrically-driven compressors. A pneumatic ram was provided for driving the chord pins, but its use was not found necessary. A total weight of 410,000 lb. of steel, more than equal to the weight of a heavy 200-ft. span double-track bridge, was erected in one day on the cantilever arm; and the maximum amount erected in one day was 680,000 lb., when four lower chord sections were erected on the south anchor-arm.

Erection of the Suspended Span.

The suspended span was to have been erected by the cantilever method, in halves, from both ends of the cantilever arms to the centre, the semi-trusses forming, during erection only, continuous extensions of the cantilever trusses, and making cantilevers of 900 ft. maximum length. It was at first intended to erect the suspended span by the large gantry traveller with which the cantilever and anchor arms were erected. It was, however, finally decided to erect the south half of the suspended span while the north anchor arm was being erected, and in order to release the gantry traveller for this work, and for other considerations, it was decided to erect the suspended span by a much simpler and lighter traveller, which virtually consisted of a horizontal platform travelling on the top chords, and having stiff-leg derricks, reaching a panel in advance, and building out the structure ahead of itself in the usual manner.

The Centre Suspended Span Traveller.

As just explained, the 675-ft. span was being erected, cantileverwise, by an overhead steel cantilever traveller, moving on the completed top chord. This traveller had two triangular Fink trusses, 39 ft. deep, 131 ft. long, and 67 ft. apart on centres. The main vertical posts were extended about 14 ft. below the centres of the approximately horizontal bottom chords, and at the lower ends had horizontal pins connecting them to the deep web-plates of trucks, with two double-flange wheels, tandem. The traveller was supported in transit on these wheels, engaging single rails 67 ft. apart, set with wooden cross-ties on the centre lines of the inclined top chords.

When the traveller was in service, steel wedges were driven between the top and bottom castings, engaging the lower end of the main posts and the grillage beams interpolated between track-ties on the finished trusses, and the traveller was lifted from the wheel-bases, and its weight transmitted through the fixed bearings to the forward panel points, the reactions being provided for at the rear end by U-bolts anchoring it to the top chord-pins.

The projecting lower ends of the main vertical posts were braced to the rear ends of the lower chords of the traveller trusses by inclined struts. The main vertical posts were connected by three horizontal struts and two panels of X-bracing in their transverse plane. The lower chords were made with pairs of built channels, latticed, and to them were web-connected eight transverse girders carrying intermediate longitudinal girders X-braced, with horizontal diagonal angles in all panels. Tackles were suspended at fixed points from the longitudinal girders, and their lead lines passed to the hoisting-engines through guide-sheaves set in vertical frames attached to the main-post strut. The hoisting-engines were seated on a solid floor platform at the extreme rear end of the traveller, where their weight was most effective as a counterbalance for the loads suspended from the overhang.

Erection Adjustments.

During erection the spaces between the adjacent ends of the lower chords in the suspended span and cantilever arms were each filled by two 1250-ton horizontal hydraulic jacks, reacting against the heavy end vertical transverse plates. Special sets of eye-bars in the line of the top chords in the end panels of the cantilever arms were connected at one end to the pins in the slotted holes in the end top chords of the suspended span. These eye-bars were made in double lengths, and the middle pins were connected to two bearings, vertically separable, so that the two groups of eye-bars connected to the two pins form an oblique parallelogram, with the diagonals varying according to their relative positions.

The pins were separated by a pair of 500-ton vertical hydraulic jacks, thus forming a powerful toggle. When the jacks were extended the vertical diagonal of the parallelogram was increased, and the horizontal diagonal was shortened, lifting the outer end of the connected half of the suspended span, an operation which was supplemented by extending the jacks between the ends of the lower chords. Reversing both sets of jacks lowered the ends of the trusses.

The lower toggle-pins were relatively fixed in the feet of heavy vertical riveted yokes or rectangular frames, about 23 ft. long and 8 ft. wide over all. The upper pins engaged rectangular blocks sliding in the yokes, with a total travel of 8 ft. The piston of the hydraulic jack engaged a bearing girder travelling between guides, and the variable space between it and the other pin-bearing was filled by a set of shim-plates between guides. A similar set of shims was set on each side of the jack, and the pressure was alternately transferred from the centre to the two outside sets, and back again, as the jacks were released and pumped out again. Similarly, three sets of shims were used with the jacks in each lower-chord connection. When the erection of the suspended span was commenced, the jacks were set so that they would carry the ends of the trusses above their required positions, and it would only have been necessary to slack them off to make the final centre-panel connections.

At each end of the suspended span the yokes, or vertical members, of the toggles were connected from one truss to the other by top and bottom horizontal transverse struts, and the panels between these struts and the cantilever arm top-chord transverse strut were X-braced in two panels, besides which the upper toggle-pins in opposite trusses were connected by a transverse strut. This system afforded a very simple and positive adjustment, the shims provided against any danger from possible failure of the hydraulic jacks, and all the apparatus would have been easily removed after the erection was completed; and the hydraulic jacks—the most costly part of it—would have been available for any other use, and could have been kept for permanent erection plant.

The type and connections of the superstructure were designed with the approval and verification of Mr. E. A. Hoare, chief engineer, Mr. Theodore Cooper, consulting engineer, and Mr. Bernt Berger, chief of staff, for the Quebec Bridge and Railway Company; and of Mr. John Sterling Deans, chief engineer, and Mr. C. L. Szlapka, designing engineer for the contractors, the Phoenix Bridge Company. The superstructure connections were detailed under the direction of Mr. Charles Scheidl; the traveller, by Mr. C. W. Hudson; and the erection, in charge of Superintendent A. B. Milliken, was largely designed by Erecting-Engineer G. A. Tretter, in conference with him and with Messrs. Deans and Szlapka.

Construction at the shop was in charge of General Superintendent R. W. Wright.

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Die Dampfkessel. Lehr- und Handbuch für Studierende Technischer Hochschulen, Schüler Höherer Maschinenbauschulen und Techniken, sowie für Ingenieure und Techniker. Bearbeitet von F. Tetzner, Professor, Oberlehrer an den Kgl. Vereinigten Maschinbauschulen zu Dortmund. Dritte, verbesserte Auflage. Berlin: Julius Springer. [Price 8 marks.]

Technische Untersuchungsmethoden zur Betriebskontrolle, Insbesondere zur Kontrolle des Dampfbetriebes. Zugleich ein Leitfaden für die Arbeiten in den Maschinenbaulaboratorien Technischer Lehranstalten. Von Julius Brand, Ingenieur, Oberlehrer der Königlichen Vereinigten Maschinenbauschulen zu Elberfeld. Zweite, vermehrte und verbesserte Auflage. Berlin: Julius Springer. [Price 8 marks.]

Belgian Blast-Furnaces.—The number of furnaces in blast in Belgium at the commencement of October was 36, as compared with 36 at the commencement of October, 1906. The number of furnaces out of blast was six at both dates. The total of 36 representing the furnaces in blast at the commencement of October was made up as follows:—Charleroi group, 16; Liége group, 14; Luxembourg, 6. The production of pig in Belgium in September was 116,610 tons, as compared with 114,500 tons in September, 1906. The aggregate output for the nine months ending September 30, this year, was 1,067,250 tons, as compared with 1,051,990 tons in the first three quarters of 1906. The total of 1,067,250 tons was made up as follows:—Puddling pig, 167,680 tons; casting pig, 75,390 tons; steel pig, 824,180 tons.

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