Long-Span Steel Arch Bridge Over Niagara Gorge for Michigan Central R. R.

Well-Known Cantilever Bridge Will Be Replaced After 40 Years' Service by Two-Hinged Spandrel-Braced Arch of 640-Ft. Span, Carrying a Solid-Floor Independent Deck—Advanced Methods of Design

A LONG-SPAN double-track arch bridge will be built over the Niagara River gorge, just below the city of Niagara Falls, N. Y., to replace the old cantilever bridge of the Michigan Central R.R. The location of the new crossing resulted from an unusually thorough study of the local topography and other physical conditions, while the design of the structure itself represents a noteworthy application of advanced bridge engineering principles. At the present time the design and all preparations for construction are completed, and contract for the construction of the steelwork has been let to the American Bridge Co., work to begin this spring. The abutments and other items of substructure work will be built under a separate contract. A review of the design is presented herewith, on the basis of data supplied by J. F. Deimling, chief engineer, and H. Ibsen, special bridge engineer of the railway.

The new bridge is required because the cantilever bridge, now forty years old, has for a long time been much overloaded, and has been costly and troublesome to maintain, in addition to necessitating speed and load restrictions which have become more and more objectionable. The planning of a new bridge began in 1916 and has been much delayed as a result of the war. The old cantilever bridge controlled the location, both because of the narrowness of the gorge at this point and because of the existing track and yard facilities. Arch construction was early decided upon as best suited to the case, and a complete design was prepared for an arch to be built around the cantilever—the old alignment being retained, in spite of the great difficulty of such construction, because the charter for the old bridge narrowly restricted its location. Subsequent explorations of the subsoil to reveal the topography of rock surface made it evident, however, that on the line of the cantilever the span between suitable points for arch abutments would be much longer than on a slightly different line, 50 to 100 ft. downstream. The revised location afforded satisfactory abutments for an arch of 640 ft. span, as determined from rock contours and profiles obtained by means of some sixty vertical and inclined borings in the side of the gorge, while on the line of the cantilever a span of 700 ft. would be necessary. As indicated in the map Fig. 1 the new structure will be close alongside the old one at the west or Canadian end, thence diverging to the north as the American side is approached.

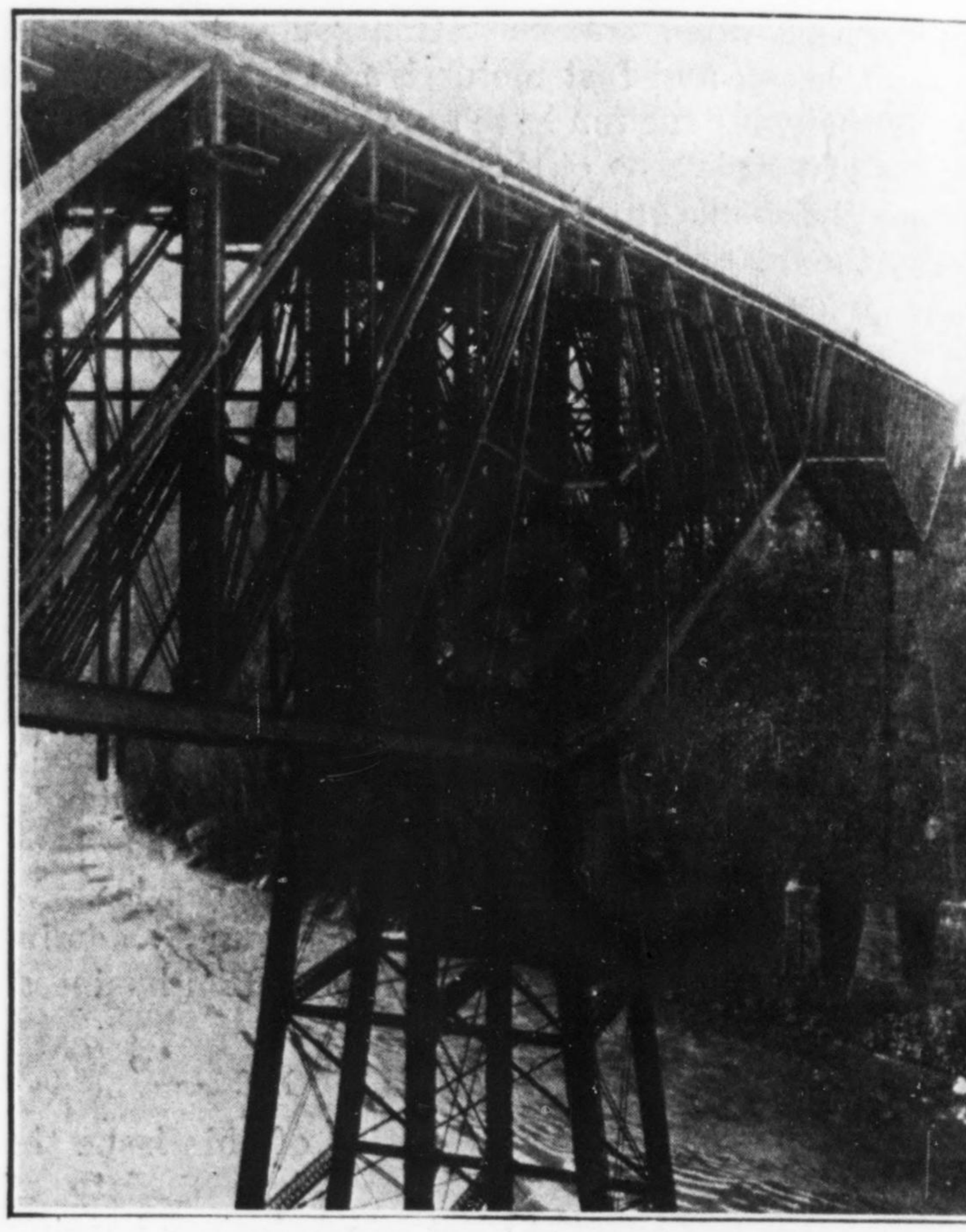


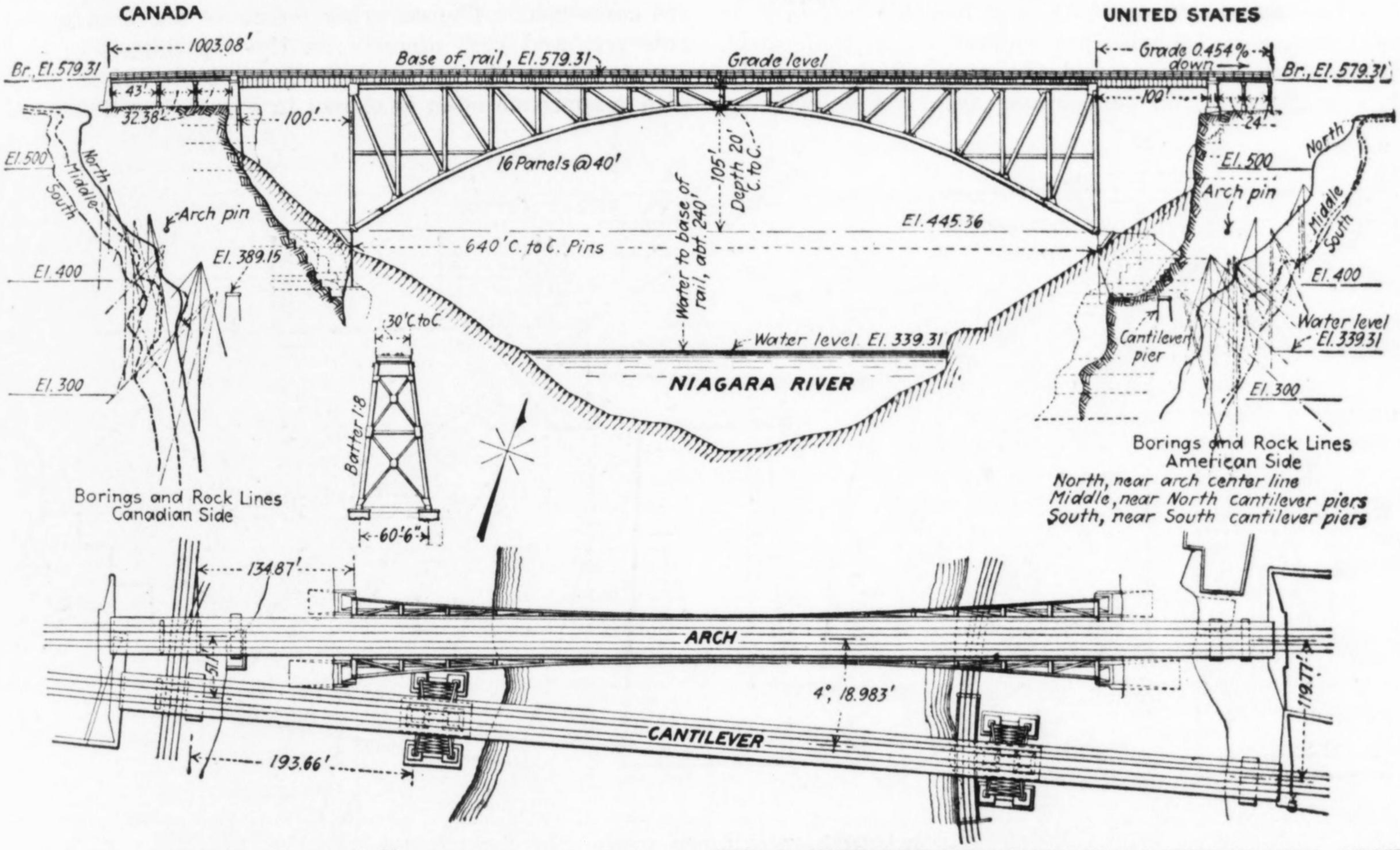
FIG. 2—NIAGARA GORGE AT SITE OF NEW BRIDGE
Present cantilever bridge of Michigan Central R.R. seen looking from American toward Canadian bank; the new arch will be alongside and downstream, about on the line of the right-hand edge of the picture, where the shortest span between hard rock strata is obtained.

FIG. 1—NEW RAILWAY ARCH BRIDGE TO CROSS NIAGARA GORGE BETWEEN EXISTING CANTILEVER AND ARCH BRIDGES

Old Bridge—The cantilever bridge, built in 1883, from designs of C. C. Schneider, has a main span of 495 ft. c. to c. towers. It was originally proportioned for a loading of two 66-ton locomotives followed by 2,000 lb. per foot of track, corresponding roughly to E23. By 1900 the cantilever was no longer of sufficient capacity, and a third (middle) truss was put in, with independent tower posts and pedestals for them; the stringers were doubled at the same time, and the capacity was thus brought up to E35. Subsequently various other repairs and improvements were made in the structure. The webs of the floorbeams developed cracks at their ends, from being bent transverse to their plane, caused partly by the restraining action of the stringers and of a separate wind chord added in 1891, that prevented the floorbeams, which are riveted to the truss post, from following the truss motion under live-load strain, and partly by the pull of the lateral bracing, which is attached to the floorbeam web a short distance inward from the floorbeam connection. The connections of the floorbeams to the trusses were also poorly detailed. The floorbeam connections and the webs were repaired, and the lacing of the posts, which showed itself inadequate, was doubled. The hip pins in the anchor arms had worn into the web metal of their posts, and this wear was repaired by adjustable bars looped around the pin and supported on brackets riveted to the post. Other troubles could not readily be corrected, such as the imperfect working together of the separate tension and compression members of the top chord, which was made up of eyebars inside a normal compression section, the former to take tension and the latter compression. The structure has required more or less constant attention. In the meantime traffic loads steadily increased; the maximum loading now permitted on the bridge is the equivalent of E45, operated at limited speed.

Gorge Conditions—Borings made alongside the tower pedestals of the cantilever bridge in 1900 showed that solid rock is about 200 ft. below water, and that the filling of the gorge bottom is a talus of large rock on which the cantilever bridge foundations rest. The river channel is grooved out in this talus mass, and according to two very careful series of soundings, the first made in 1900 and the second in 1918, carried out with 400- and 600-lb. cast-iron sounding weights from the deck of the cantilever bridge, is about 85 ft. deep at the middle, with general outline about as shown in Fig. 3. The walls of the gorge present horizontal strata of firm limestones, shales, and sandstones. An arch sprung between the gorge walls was the most natural and suitable solution of the problem of a heavy-capacity bridge under these circumstances.

At the top of the gorge walls there is found a 50-ft. thickness of Niagara limestone, under which is a bed of shale of about the same thickness. Directly below this shale about 20 ft. of Clinton limestone occurs, resting in turn on Medina shales and sandstones. Of the series, the Clinton, a hard and durable stratum, was the most suitable as support for the bridge abutments. Much of this stratum, however, is masked by a talus slope which covers the lower part of the gorge walls. The true outlines of the undisturbed rock surface were not known until the results of an exhaustive series of borings were available. From these results the shortest span between the Clinton limestone outcrops could be determined, and the present location is the result. In Fig. 3 is included a small-scale reproduction of the rock profile and the locations of the borings.



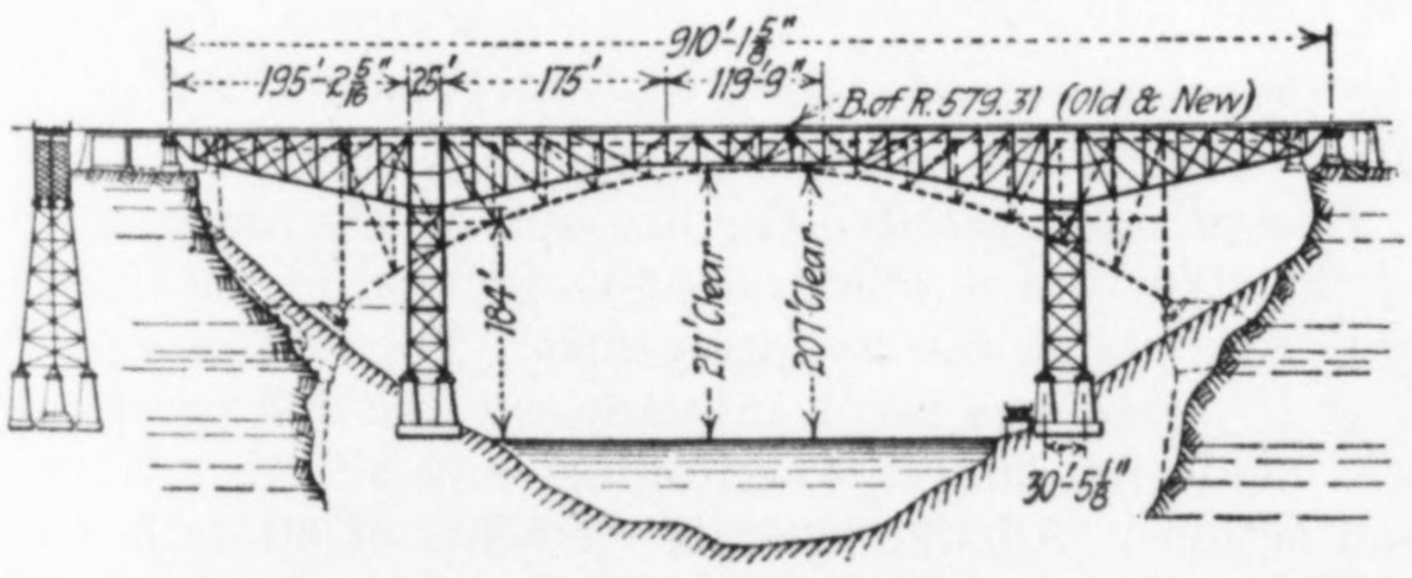


FIG. 3—NEW BRIDGE DESIGNED AS TWO-HINGED ARCH OF 640-FT. SPAN WITH TRACK 240 FT. ABOVE WATER

The plans for the new crossing required rail level to be at the same elevation as the present. However, as there are several grade crossings on both sides of the river, which may at some future time require a change of grade, the floor construction was worked out in such a way as to permit of raising the floor without affecting the main carrying structure.

General Design—As represented in Fig. 3, the arch designed to suit these conditions is of the two-hinged spandrel-braced type, in which respect it is similar to the 550-ft. arch close by. Its span is 640 ft. center to center of end pins, its bottom-chord rise 105 ft., and the crown depth 20 ft. The web system comprises sixteen panels of 40 ft., but the floor panels are only one-third as long. The two arch trusses are spaced 30 ft. apart in the line of the top chord, and thence downward have an outward batter of 1 : 8.

The spandrel-braced two-hinged type of arch was adopted for superior rigidity and appearance, as against a rib arch. Appearance also dictated the parabolic curve of the bottom chord.

Full riveted construction was decided upon. Subsequently, on computation of the secondary stresses in the structure, it was found that the short web posts in the middle section of the arch would be subjected to excessively heavy bending, and for this reason four posts were detailed with pin connections at their ends. Except for these pins and the pins which form the skewback hinges, all connections are riveted. During erection and until completion of the structure, however, there will be a hinge at the crown of the bottom chord; the top chord is not to be connected until after completed erection, so that the dead-load stresses in the bridge will be those corresponding to three-hinged condition. It is intended to make the connection of the top chord (after setting the closing member) at the mean temperature assumed in the design (50 deg. F.), using in this process a device similar to that originated for the closure of the Hell Gate arch, where the last joint, brought into contact at the proper temperature, was held together by tie rods to prevent its separation until riveted up.

The floor construction, as already indicated, is decidedly novel. The entire floor rests on the top chord, and forms no part of the arch structure. Over each of the two top chords there is a line of longitudinal girders, carried on cast-steel shoes at the main panel points; transverse floorbeams are riveted between the girders at intervals corresponding to one-third the truss panel length, and the deck stringers rest on these floorbeams. Each main-panel section of floor is kept independent of the adjoining one, so as to form an expansion section by itself, and avoid participation of the floor in the arch stresses. The stringers, which are groups of four 15-in. I-beams under each rail, supplemented by edging and footwalk beams, are covered with a [illegible]-in. steel deck plate. The roadbed ballast will be placed on this plate after a layer of waterproofing with its bedding and cover of concrete is applied. At the ends of each expansion section the deck plate is edged by a transverse I-beam which forms a retaining curb for the ballast and is joined to the curb of the next floor section by a cap plate riveted to one of the two curbs.

In connection with the description of this remarkable deck construction (Fig. 7), it should be noted that the floor on the approach spans is closely similar to that on the arch, so far as the deck plate and parts above are concerned. The carrying members are transverse, however, and rest directly on the top flanges of the main girders. A section of the floor of the 100-ft. plate girder approach span is shown in Fig. 7, together with the deep transverse plate girder between the end posts of the arch which carries the approach span.

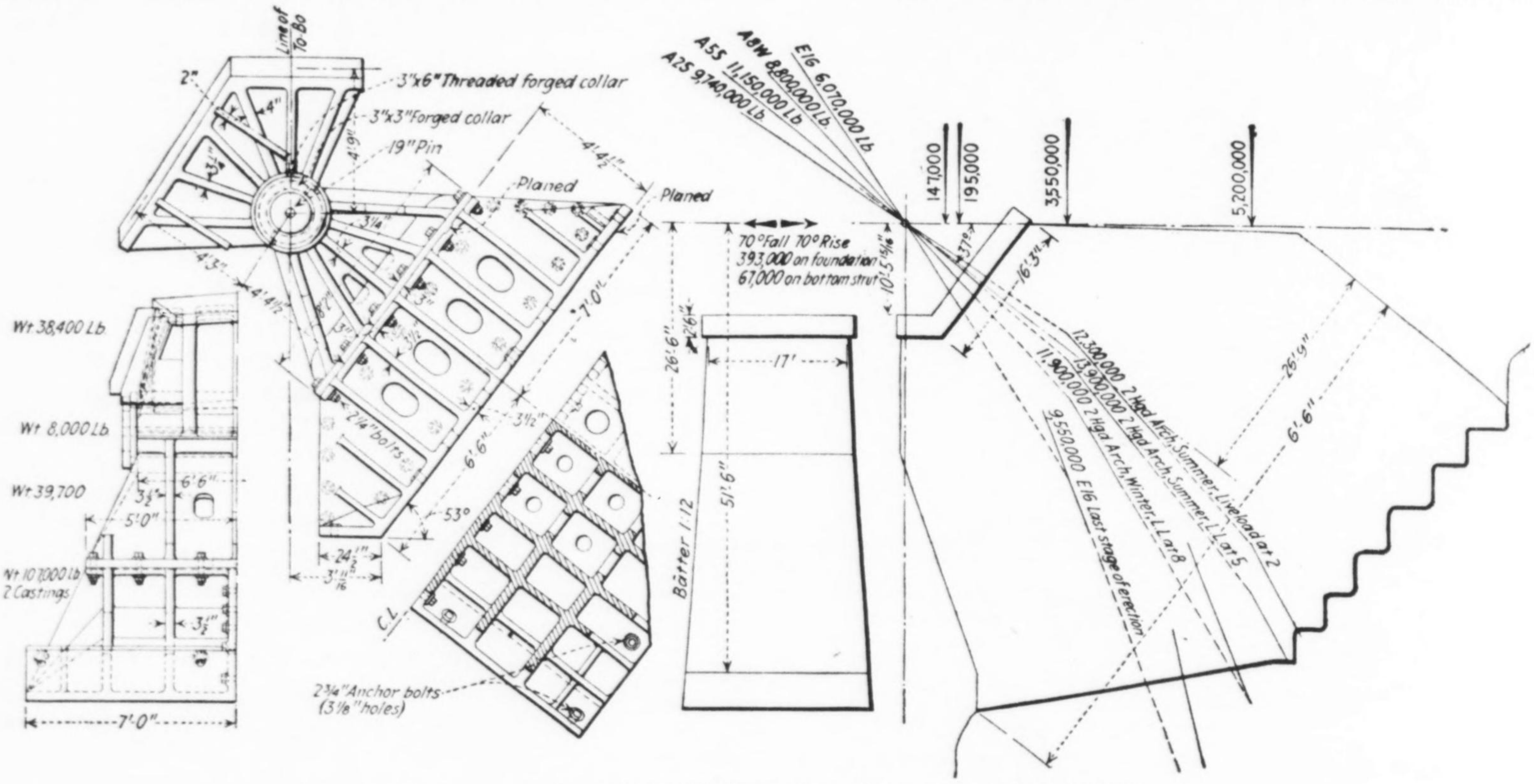


FIG. 4—END SHOE, CONCRETE PIER AND REACTIONS

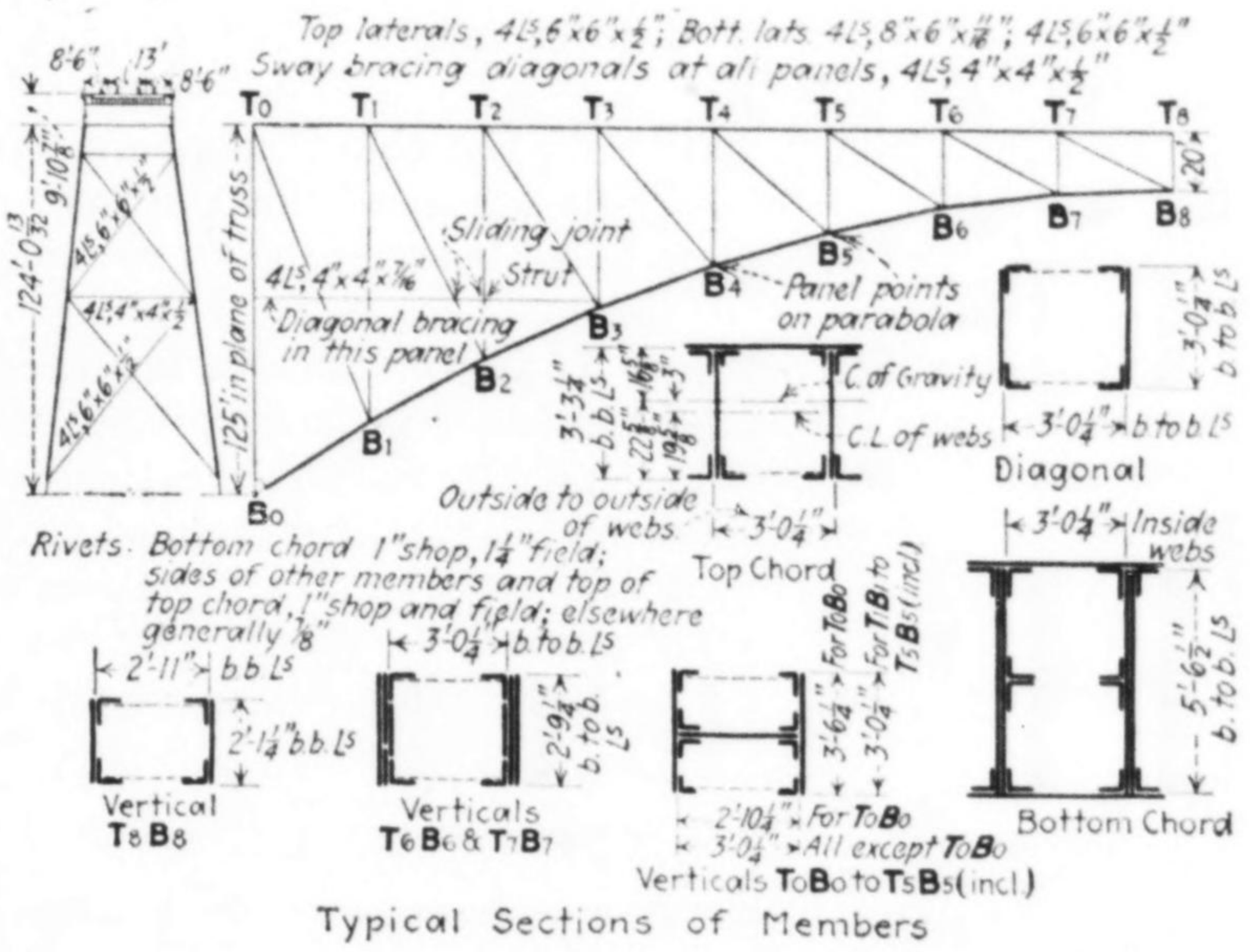
Design and Details—On account of the importance of the structure it was desired to provide for all future requirements as far as foreseeable. For this reason, a high live load was taken into account and full provision was made for lateral, longitudinal and all other forces and actions. All the separate elements of stress amenable to calculation were computed. Accordingly, high unit stresses could be used.

Dead-load stresses were computed for the three-hinged arch condition, this condition being maintained by the erection procedure until after the floor dead load is in place. Live-load stresses were computed for the two-hinged arch condition. Stresses due to wind, a separate transverse force, longitudinal or traction forces, and temperature were separately computed. Secondary stresses due to distortion under load were computed for the middle half of the bridge, outside of which length the members are of such proportions that the secondary stresses will be small. Torsion due to single-track loading and due to wind was computed by a method of approximation; the division of torsional moment at five separate cross-sections of the bridge was computed, and the shears in the laterals and truss webs for the five sections were then averaged, with the general result of assigning to the bottom laterals 65 per cent of the total resistance to wind pressure, to the top laterals 10 per cent, and to each arch 12½ per cent, which figures were used in determining stresses.

All these stresses except lateral and secondary were included in the computation of required cross-section by the normal unit stresses, noted below; secondary stresses were provided for by increase of cross-section only where they exceeded one-third of the axial stresses.

The design of members and details was carried out with a view to obtaining sturdy construction. Simplicity of connections was also sought, and for this reason the joints were made by two gussets, and splices were made without interleaving of webs. The bottom chord was designed as a box-shaped member of uniform depth throughout, composed of two I-shaped ribs and top and bottom cover plates. This member is very heavy, reaching a maximum cross-section of 714.8 sq.in. in the end section, the latter forming a 70-ton erection piece. The field joints in the chord are faced to broken outline each outer third of the depth of the face being backed off 1/32 in. so that the bearing during erection is concentrated in the middle section and the joint can adjust itself to distortion until the splice is riveted up.

The top chord is of normal top-chord section, composed of two channel-shaped side members with cover plates. The posts have channel-shaped side members connected by a continuous longitudinal diaphragm. The diagonals are of similar section without a diaphragm. The open sides of all members are laced with stiff lacing,



TOP CHORD			BOTTOM CHORD		
Member	Resultant Max. and Reversal Stresses	Make-up	Member	Resultant Max. and Reversal Stresses	Make-up
T_0T_1	Max. -487 Rev. +110 Req. 41 in.²	1 Cov. Pl., 53 x 7/8 2 Top L^s, 5x5x5/8 2 Top L^s, 8x8x5/8 4 Bott. L^s, 8x8x3/4 2 Webs, 39 x 1 3/8 Net 189.71 in.²	B_0B_1	Max. -9701 Req. 713 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 2 Cov. Pls., 60 x 1 1/16 2 Cov. Pls., 60 x 1 Gross 714.78 in.²
T_1T_2	Max. -964 Rev. +234 Req. 81 in.²	1 Cov. Pl., 53 x 7/8 2 Top L^s, 5x5x5/8 2 Top L^s, 8x8x5/8 4 Bott. L^s, 8x8x3/4 2 Webs, 39 x 1 1/4 Net 181.90 in.²	B_1B_2	Max. -9321 Req. 680 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 2 Cov. Pls., 60 x 1 1/16 2 Cov. Pls., 60 x 11/16 Gross 677.28 in.²
T_2T_3	Max. -1491 Rev. +356 Req. 125 in.²	1 Cov. Pl., 53 x 7/8 2 Top L^s, 5x5x5/8 2 Top L^s, 8x8x5/8 4 Bott. L^s, 8x8x3/4 2 Webs, 39 x 15/16 Net 162.37 in.²	B_2B_3	Max. -9107 Req. 664 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 4 Cov. Pls., 60 x 13/16 Gross 662.28 in.²
T_3T_4	Max. -2057 Rev. +430 Req. 170 in.²	1 Cov. Pl., 53 x 5/8 2 Top L^s, 5x5x5/8 2 Top L^s, 8x8x5/8 2 Bott. L^s, 8x8x5/8 2 Webs, 39 x 15/16 Gross 175.64 in.²	B_3B_4	Max. -8961 Req. 649 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 2 Cov. Pls., 60 x 13/16 2 Cov. Pls., 60 x 11/16 Gross 647.28 in.²
T_4T_5	Max. -2599 Rev. +346 Req. 208 in.²	1 Cov. Pl., 53 x 3/4 2 Top L^s, 5x5x5/8 2 Top L^s, 8x8x5/8 4 Bott. L^s, 8x8x3/4 2 Webs, 39 x 1 3/16 Gross 209.08 in.²	B_4B_5	Max. -8860 Req. 637 in.²	Do.
T_5T_6	Max. -2928 Rev. +133 Req. 227 in.²	1 Cov. Pl., 53 x 7/8 2 Top L^s, 5x5x5/8 2 Top L^s, 8x8x5/8 4 Bott. L^s, 8x8x3/4 2 Webs, 39 x 1 5/16 Gross 225.46 in.²	B_5B_6	Max. -8723 Req. 628 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 2 Cov. Pls., 60 x 13/16 2 Cov. Pls., 60 x 9/16 Gross 632.28 in.²
T_6T_7	Max. -2948 Req. 222 in.²	Do.	B_6B_7	Max. -8414 Req. 606 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 4 Cov. Pls., 60 x 5/8 Gross 617.28 in.²
T_7T_8	Max. -2606 Req. 196 in.²	Do.	B_7B_8	Max. -7758 Req. 558 in.²	8 Webs, 66 x 5/8 12 L^s, 8x8x3/4 2 Cov. Pls., 60 x 3/4 Gross 557.28 in.²

VERTICALS			DIAGONALS		
Member	Resultant Max. and Reversal Stresses	Make-up	Member	Resultant Max. and Reversal Stresses	Make-up
T_0B_0	Max. -2144 Req. 193 in.²	2 Pls., 42 x 1 4 Cor. L^s, 8x8x7/8 4 Diaph. L^s, 8x8x7/8 1 Web, 34 x 1 Gross 223.84 in.²	T_0B_1	Max. -614 } Rev. +771 } Req. 84.5 in.² Max. +1132 } Rev. -283 } Req. 75 in.²	2 Pls., 36 x 3/4 4 L^s, 8x8x5/8 Gross 92.44 in.² Net 76.69 in.²
T_1B_1	Max. -1521 Rev. +3 Req. 136 in.²	2 Pls., 36 x 3/4 4 Cor. L^s, 8x8x5/8 4 Diaph. L^s, 6x6x1/2 1 Web, 36 x 5/8 Gross 136.10 in.²	T_1B_2	Max. -628 } Rev. +644 } Req. 114 in.² Max. +1031 } Rev. -278 } Req. 74 in.²	2 Pls., 36 x 1 4 L^s, 8x8x11/16 Gross 114.12 in.² Net 94.87 in.²
T_2B_2	Max. -1391 Req. 148 in.²	2 Pls., 36 x 1 4 Cor. L^s, 8x6x5/8 4 Diaph. L^s, 6x6x1/2 1 Web, 36 x 5/8 Gross 150.94 in.²	T_2B_3	Max. -602 } Rev. +542 } Req. 87 in.² Max. +968 } Rev. -221 } Req. 67 in.²	Same as T_0B_1
T_3B_3	Max. -1275 Req. 118 in.²	2 Pls., 36 x 5/8 4 Cor. L^s, 8x6x1/2 4 Diaph. L^s, 6x6x1/2 1 Web, 36 x 5/8 Gross 117.50 in.²	T_3B_4	Max. -543 } Rev. +443 } Req. 69 in.² Max. +934 } Rev. -132 } Req. 62 in.²	Do.
T_4B_4 and T_5B_5	Max. { T_4B_4 -1171; T_5B_5 -1096 Req. { T_4B_4 = 97 in.²; T_5B_5 = 84 in.²	2 Pls., 36 x 1/2 4 Cor. L^s, 6x6x1/2 4 Diaph. L^s, 6x6x1/2 1 Web, 36 x 5/8 Gross 104.5 in.²	T_4B_5	Max. -403 } Rev. +529 } Req. 55 in.² Max. +960 Req. 59 in.²	Do.
T_6B_6	Max. -1131 } Rev. 0 } Req. 81.5 in.² Max. +182 } Rev. -757 }	4 Pls., 33 x 11/16 4 L^s, 8x8x11/16 2 Pls., 16 1/2 x 11/16 Gross 155.55 in.²	T_5B_6	Max. -594 } Rev. +407 } Req. 66 in.² Max. +1052 } Rev. -53 } Req. 65 in.²	Do.
T_7B_7	Max. -1146 } Rev. +49 } Req. 82 in.² Max. +298 } Rev. -861 }	4 Pls., 33 x 11/16 4 L^s, 8x8x11/16 2 Pls., 16 1/2 x 11/16 Gross 155.55 in.²	T_6B_7	Max. -1045 } Rev. +718 } Req. 115 in.² Max. +1374 } Rev. -374 } Req. 93 in.²	2 Pls., 36 x 1 4 L^s, 8x8x3/4 Gross 117.76 in.² Net 97.76 in.²
T_8B_8	Max. -717 Req. 51 in.²	2 Pls., 25 x 5/8 4 L^s, 8x8x11/16 Gross 73.37 in.²	T_7B_8	Max. -1469 } Rev. +1031 } Req. 162 in.² Max. +1616 } Rev. -878 } Req. 121 in.²	2 Pls., 36 x 1 4 L^s, 8x8x3/4 2 Pls., 19 x 3/4 4 Flats 8 x 5/8 Gross 166.26 in.² Net 138.39 in.²

Stresses in 1000-lb. Units

FIG. 5—STRESS AND MAKE-UP SHEET FOR NEW NIAGARA ARCH

Design made for E70 train loading and based on unit stresses of 20,000 lb. per sq. in. for dead load and 18,000 lb. per sq.in. for live load. Ample wind and lateral and longitudinal force calculations were included, and secondary stresses were allowed for in the middle half of the structure, insofar as they exceed one-third of the primary stresses.

generally composed of 6-in. channels attached on the inner face of the flanges.

The gusset plates, built into the chord members, are 1-in. and 1¼-in. plates. Their edges within the web of the chord are not planed to bearing but are fully cover-spliced.

Rivets ranging from ⅝ to 1¼ in. in diameter are used. In general, the bottom chord has 1-in. shop rivets and 1¼-in. field rivets, the side and top main connections of the top chord 1-in. rivets, the laterals and lacing of the same chord ⅞-in. rivets, the web members generally 1-in. rivets, and all minor parts ¾-in. rivets. In every case where the grip is over four diameters, taper rivets are specified; these have a diameter for 1-in. from the head $\frac{3}{64}$ in. larger than the nominal size, and thence taper to a point diameter $\frac{1}{64}$ smaller than the nominal size. Such rivets must have their points cooled.

Rather high-grade workmanship is specified for the fabrication of this structure. All main riveting is to have drilled or sub-punched and reamed holes, and quite detailed requirements are set up for the accuracy of match of holes. The arch is to be assembled in the shop yard in sections of three panels at a time, the connection holes being here drilled or reamed to the unstrained position of the members. The lengths of the members, however, are to be made to a camber equal to the deflection under dead load plus one-half live load plus impact.

At the skewbacks, a simple pin hinge was adopted, as contrasted with the circular roller bearing used in the near-by 550-ft. arch. The load is transferred by this pin to a cast-steel shoe having a base area 14 ft. x 14 ft. 8¾ in. where it rests against the abutment concrete. This shoe as finally designed consists of three pieces, the bottom section being formed of right and left halves while the top or pin section is one piece. The pin, 19 in. in diameter, is not seated in the steel of the chord but in a cast-steel pin casting which receives the end of the chord. Upper and lower pin castings are held together at either end of the pin bearing by a steel retainer collar 3x3 in. in cross-section.

The maximum thrust delivered to the abutment through the shoe is 11,150,000 lb. The change in inclination of the thrust under load and temperature variations is represented in Fig. 4.

In the same figure the form of the concrete abutments is represented. The four abutments differ considerably in size, according to the location of the edge of the hard rock stratum. They range in volume from 750 to 2,200 cu.yd. of concrete each. As the firm rock is covered by a considerable depth of talus at these points, a relatively large amount of excavation is required, totaling 28,000 cu.yd. for the four. The bedding surface for the shoe is to be dressed plane, and a $\frac{1}{16}$-in. sheet of lead is to be placed under the casting.

Material—Normal bridge steel or mild steel will be used in constructing this bridge, with specified ultimate strength of 56,000 to 64,000, elastic limit 33,000 minimum, and elongation 25 per cent (less for thick material, down to 20 per cent). Rivet steel will be softer, with 46,000 to 54,000 ultimate, 28,000 elastic limit, and 30 per cent elongation. All material is required to stand a bend test, which for the structural material is flat closing for thicknesses of ⅞ in. or less, ranging to a bend around twice the thickness of the piece for thicknesses over 1¼ in.

Medium or high-carbon steel was not considered desirable for this bridge. Alloy steel was considered, but in the opinion of the engineers its use was not warranted if its cost exceeded that of ordinary steel by more than 1 cent per pound. Present differences of price are considerably greater than this.

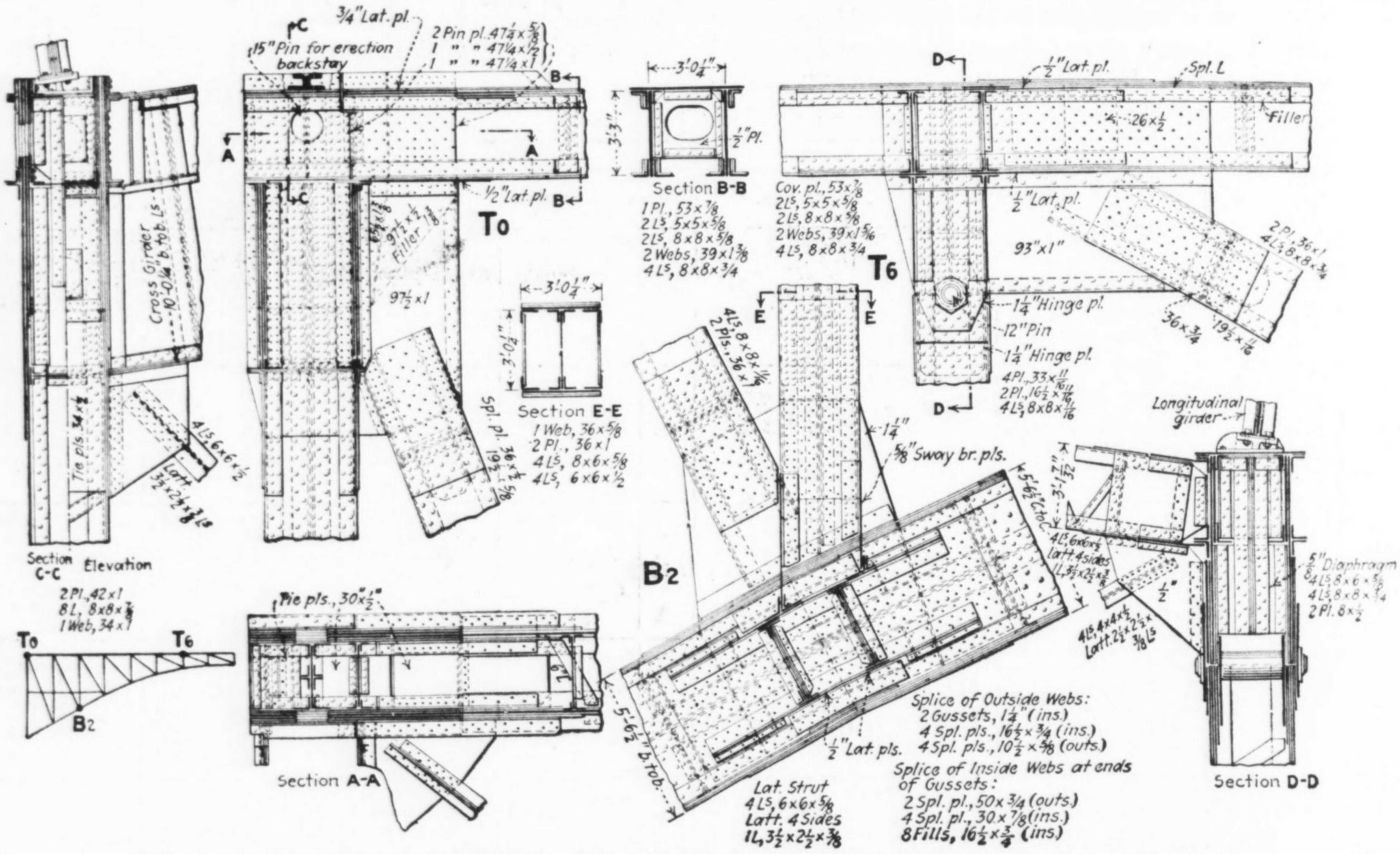


FIG. 6—REPRESENTATIVE DETAILS OF MICHIGAN CENTRAL ARCH OVER NIAGARA GORGE

The detailing is exceptionally simple for a structure involving members of maximum proportions. Two-web construction and pairs of 1¼-in. gusset plates are used in the chords. All latticing is made of angles or channels. The 1 to 8 batter of the two arch trusses affects only the lateral bracing connections and cast-steel floor girder pedestals.

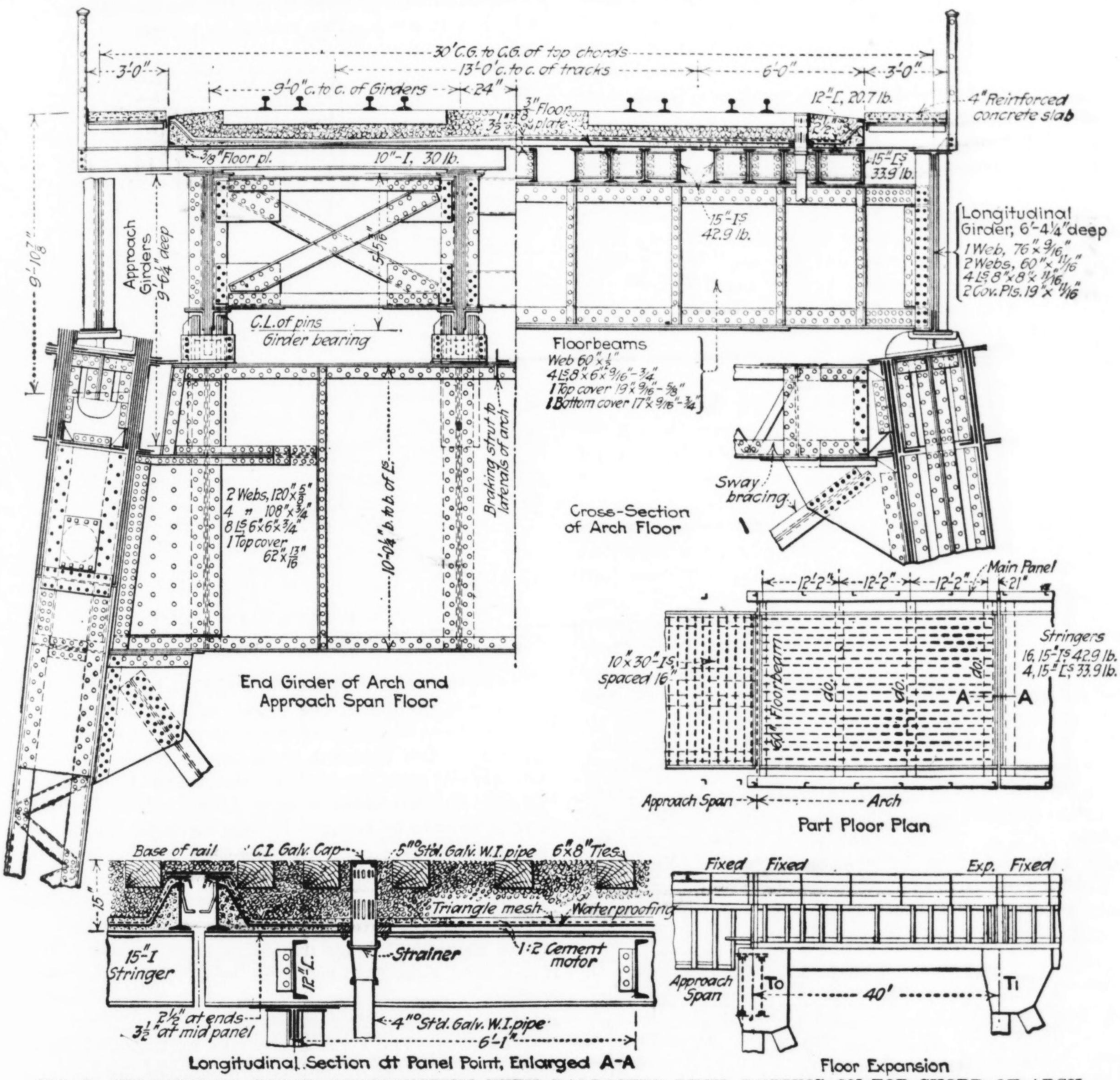


FIG. 7—SUB-PANELED FLOOR CONSTRUCTION WITH BALLASTED DECK, RESTING ON TOP CHORD OF ARCH

Design Data—The floor of the bridge was designed for Cooper's E70 locomotive loading (two consolidation locomotives, 2-8-0, with 70,000-lb. driving axles spaced 5 ft., representing a total load of 994,000 lb. on a 104-ft. wheel base, followed by 7,000 lb. per foot of track). For the arch members this wheel loading was replaced by a uniform load of 7,000 lb. per foot following a 110-ft. length of 9,000 lb. per foot of track. These loadings are higher than the New York Central Lines' standard for a 640-ft. span, which is E65; it was believed that for so important a structure a larger margin for future development should be allowed. However, as the unit stresses are higher than those generally employed, the actual design is substantially the same as would have resulted from E65 loading with ordinary unit stresses.

In applying this loading, stresses from double-track loading were reduced by 5 per cent. The transverse-beam deck of the approach spans was proportioned for a load of 18,000 lb. per foot of track, distributed over a 10-ft. width, while the longitudinal-beam deck of the main span was proportioned for the E70 wheel loading direct. Live-load stresses were increased by an impact allowance equal to half that of the Turneaure formula, or $\frac{15,000}{30,000 + L^2}$, where L is the span (or the floor panel length), but 10 per cent was used as minimum impact. For the beams of the approach-span deck, the impact was taken as 50 per cent.

Wind pressure was taken as 30 lb. per square foot on 1½ times the vertical projection of the bridge, plus 360 lb. per lineal foot applied 8 ft. above the rail; all these pressures were considered for a position of the deck 15 ft. higher than its original position. During erection, a 50-lb. wind on one and one-half times the elevation of the bridge and the traveler is to be taken into account. In addition to the normal wind pressure, the completed bridge was also required to withstand a transverse force, due to the train, of 500 lb. per lineal foot of bridge applied 6 ft. above the rail. Similarly, a longitudinal force applied at the same height was taken into account, amounting to 10 per cent of the live load for the arch and 20 per cent on the minor spans. Temperatures changes of ± 70 deg. F. were considered.

Wind and lateral-force stresses were ignored where they did not exceed 25 per cent of the sum of other stresses (except secondaries). Secondaries, whose computation has already been referred to, were similarly ignored where they did not exceed one-third the combined axial stresses not including those due to wind and lateral force.

Two sets of unit stresses were used in proportioning the members, those applicable to live-load stresses being about nine-tenths as great as those applicable to dead-load and erection stresses. The principal figures are: tension on net section, 20,000 for dead load and erection, 18,000 for live load; compression on gross section, 18,000 — 80 l/r with 17,000 lb. maximum, and 16,000 — 70 l/r with 15,000 maximum; shear, 15,000 and 13,500; pin bearing, 27,000 and 24,000, except that for reversing bearing pressure half this stress was used; rivet bearing, 30,000 and 27,000. Except for bearing pressure, cast steel was figured at 10 per cent lower unit stresses.

In the application of these stresses several special provisions were considered, among which the following are important: Connections and splices were designed 10 per cent stronger than their members. All compression splices were made full, and were proportioned for a stress in the splice material of 16,000 lb. on gross section. Countersunk rivets and field rivets were increased in number by 25 per cent over the number of shop rivets required. Members carrying reversed stress were proportioned separately for each kind of stress increased by one-half the smaller, and end connections were proportioned for the sum of the separate stresses. The riveting was increased one-third for each filler plate between splice plate and main plate, and 1 per cent for each $\frac{1}{16}$ in. by which the grip exceeded four diameters.

Columns were limited to a length of 100 radii for main members and 120 for bracing members. In all

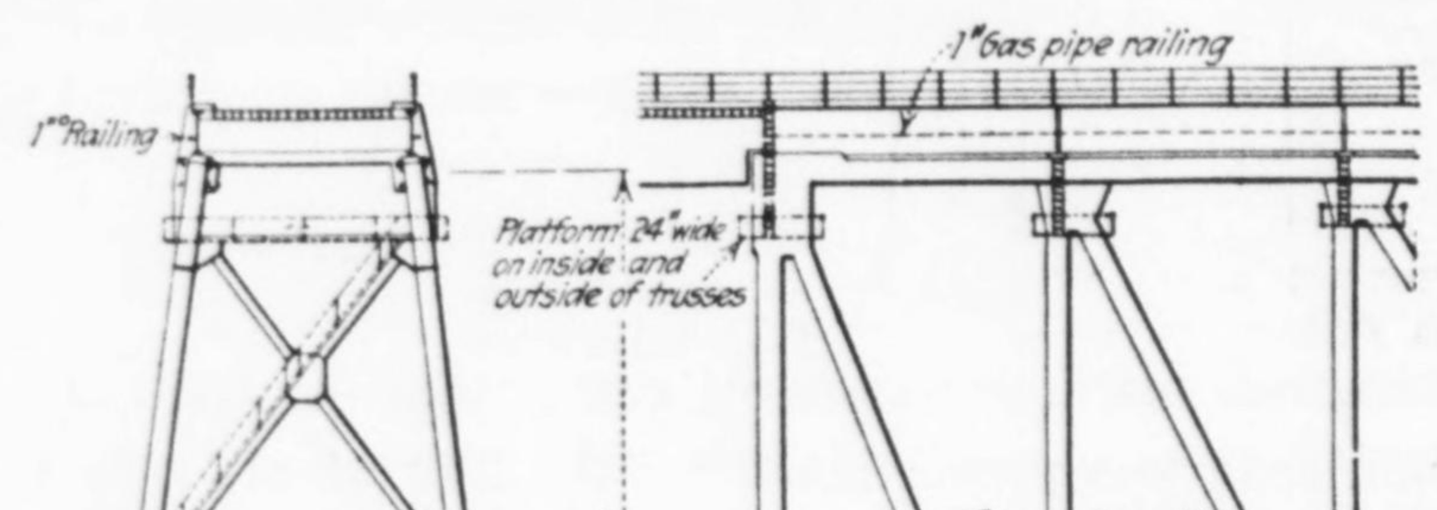


FIG. 8—INSPECTION LADDERS AND PLATFORMS

columns a shear of 300 times the cross-sectional area was provided for, except that in case of a longitudinal diaphragm the lacing was allotted only half the shear.

Erection—Each half of the arch will be erected as a cantilever, being held by an eyebar tie running back to an anchorage in the upper part of the cliff side. One such anchorage is sketched in Fig. 9; the steel anchor girders and tie members up to the face of the rock are to be left in place, after casing in concrete.

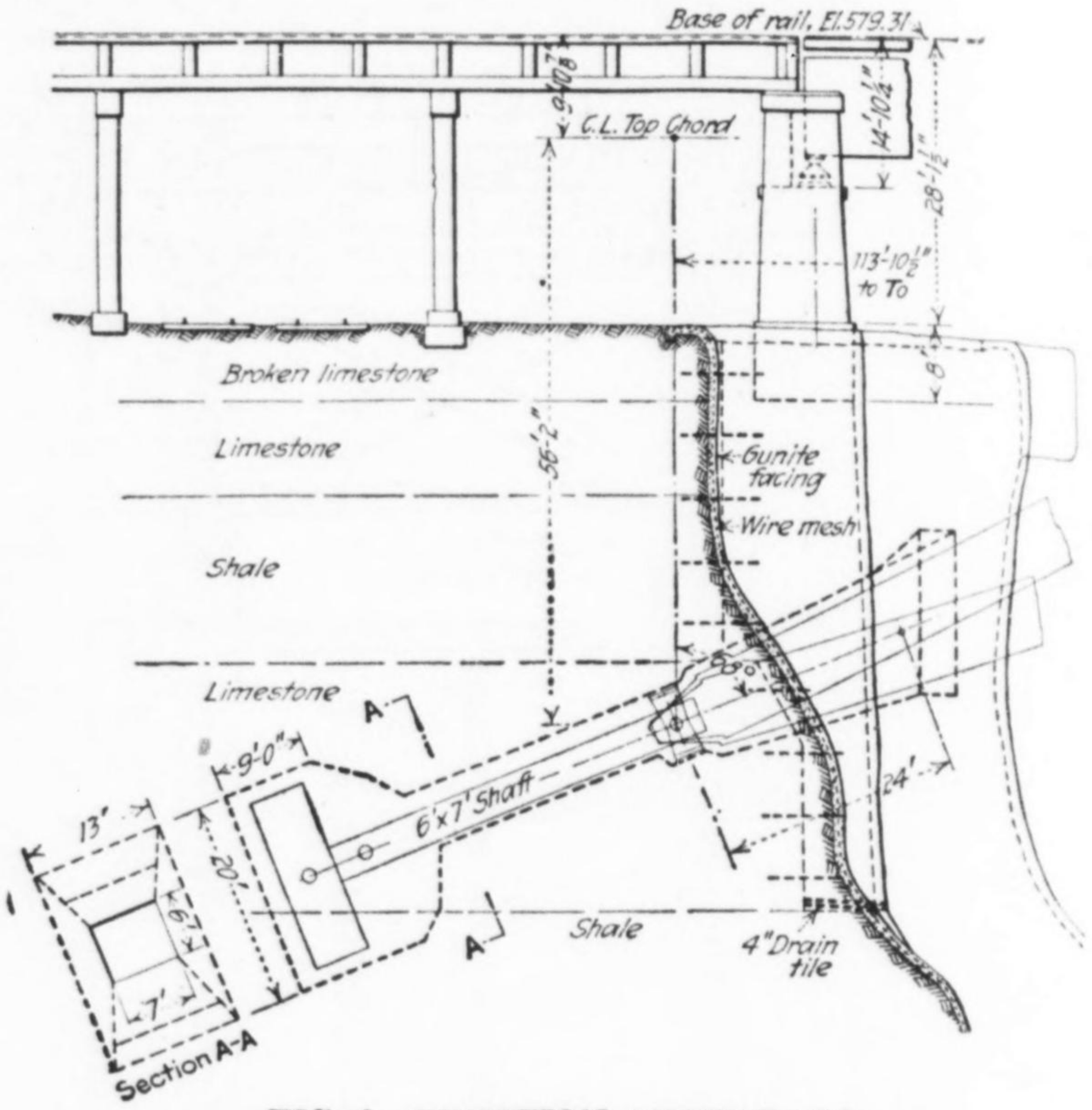


FIG. 9—ERECTION ANCHORAGE
The steel anchor members will be left in place, encased in concrete.

The bottom-chord splices are not to be riveted until the arch is joined at the crown of the bottom chord. The top chord center joint is not to be drilled or riveted until after the complete arch structure and the floor steel are in place, so that all these dead loads will be resisted by three-hinged arch action. The top-chord joint is then to be brought to bearing at 50 deg. F., and the abutting segments held together by ties until the joint splice is drilled and riveted.

Provision will be made in the members of one-half of one of the two arches for strain-gage readings, by twelve gage-point holes drilled in each member. It is intended to make such readings before, during and after erection and under live load, so as to obtain accurate information concerning stresses.

Quantities—The structure of the arch itself includes 4,800 tons of steel, including bracing and castings. A further weight of 1,250 tons of steelwork is contained in the floor. Nearly 1,500 tons of steel are required for the two 100-ft. approach girder spans, the adjoining street spans and the erection backstays, making the total steel weight 7,500 tons. About 11,000 cu.yd. of concrete will be required in the abutments and auxiliary foundations.

The contract price for the steelwork, erected, is about $116 per ton, of which about $7 is freight from fabricating shop to site.

The design of the bridge was developed under the direction of H. Ibsen, special bridge engineer, Michigan Central R.R., assisted by C. L. Christensen, assistant bridge engineer. The project is under the general direction of J. F. Deimling, chief engineer of the railway. Col. George H. Webb, late chief engineer, had the general direction of the project from its start until he went to France in 1917 as lieutenant-colonel with the 16th (Railway) Engineers, and again after his return in April, 1919, until his death on Nov. 3, 1921. Olaf Hoff, consulting engineer, is acting as advisory engineer on the project.

New Niagara Gorge Arch Nearing Completion

The Two Cantilever Arms of the Michigan Central's 640-Ft. Span Were Joined in October

On October 11, after the closing members at the crown of the Michigan Central's new 640-ft. arch across the Niagara gorge had been placed in position, the two halves of the structure, which had been extended from the two sides of the gorge by cantilever erection, were brought to bearing by releasing the backstays and the arch became a self-supporting structure. This bridge, which has the distinction of being the longest railway arch bridge in America except one, the great Hell Gate arch, has been built to replace the Michigan Central cantilever bridge erected in 1883, increased train loadings having rendered the old structure inadequate. A description of the design and details of the new bridge and an account of the preliminary engineering studies and foundation investigations appeared in the *Railway Age* of June 13, 1923, page 177. The following article is therefore confined to an account of the construction work on this bridge, which entailed the solution of many interesting problems.

General Description of the Bridge

The new bridge is located in the space between the old cantilever structure and the 550-ft. arch used by the Canadian National (Grand Trunk). It has a span of 640 ft. from center to center of hinge pins and a rise of 105 ft. It is designed to carry two railway tracks on the deck at 13 ft. centers. It belongs to the spandrel braced type of steel arch with the lower chord conforming to a parabolic curve and the top chord horizontal. It was designed for erection by the cantilever method with a pin bearing between the two halves of the lower chord at the crown so that it is a three-hinge structure for dead load exclusive of the weight of the ballast and the track construction. However, as the crown joint in the top chord will be riveted solid, the structure will function as a two-hinged arch for live load.

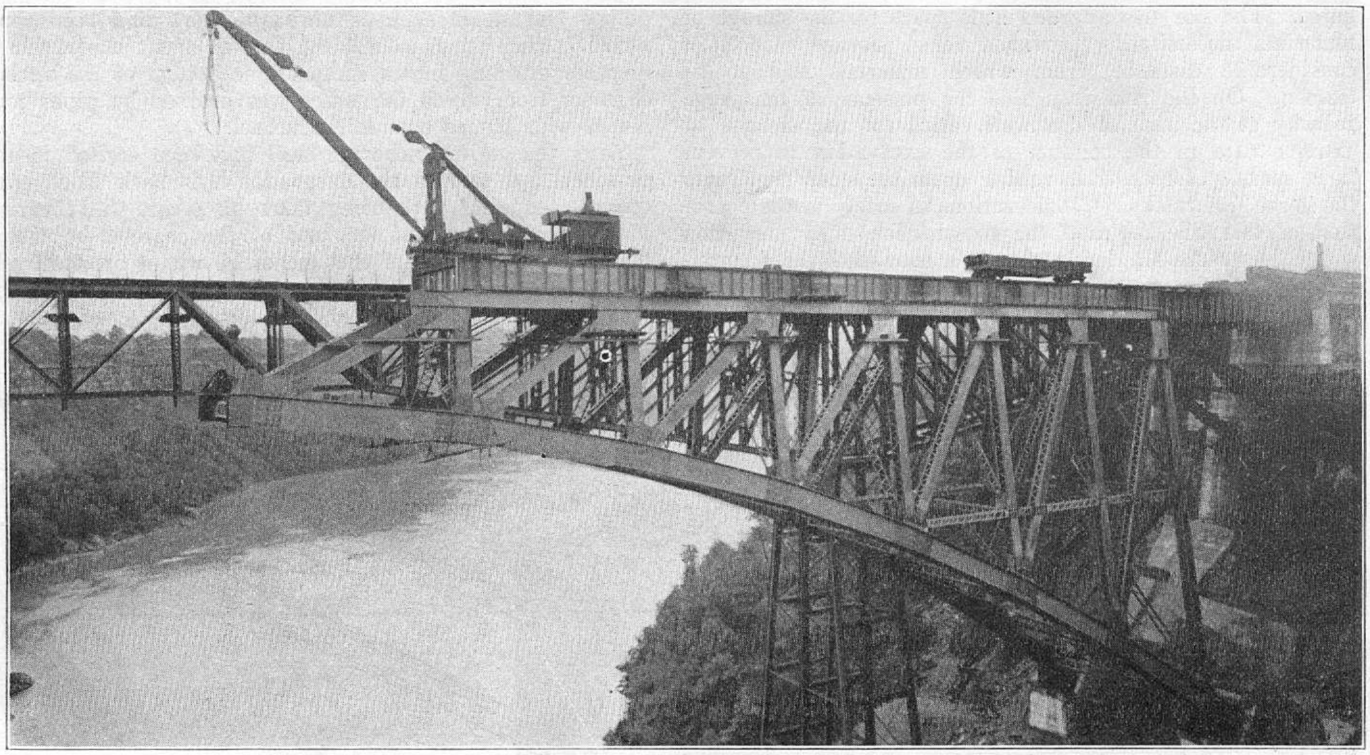
The design follows the usual practice for riveted structures except that pin connections are provided at each end of the first two vertical web members on each side of the crown. These posts are so short that deformations occurring in the arch under load or temperature changes would have introduced excessive secondary stresses if they had been provided with rigid connections to the chords.

The size and general characteristics of the main truss members are given in one of the accompanying drawings. The bottom chords are of closed box section with two webs, 3 ft. 2¾ in. center to center by 5 ft. 6½ in. deep, each web consisting of four plates ⅝ in. thick. The angles are 8 in. by 8 in. by ¾ in. throughout, the variation in the makeup being made by changing the thickness of cover plates. The maximum bottom chord section has a gross area of 714.78 sq. in., to provide for a maximum resultant stress of 9,700,000 lb. The joints in the bottom chord members are necessarily made at the panel points and the abutting ends are milled for bearing on only the middle 2 ft. 6 in. of the total depth of 5 ft. 6½ in. These chord splices have been proportioned for 100 per cent riveting. The two trusses are set on a batter of 8 vertical to 1 horizontal, all members of the trusses, including the top chord, conforming to this batter.

The Floor Is an Independent Structure

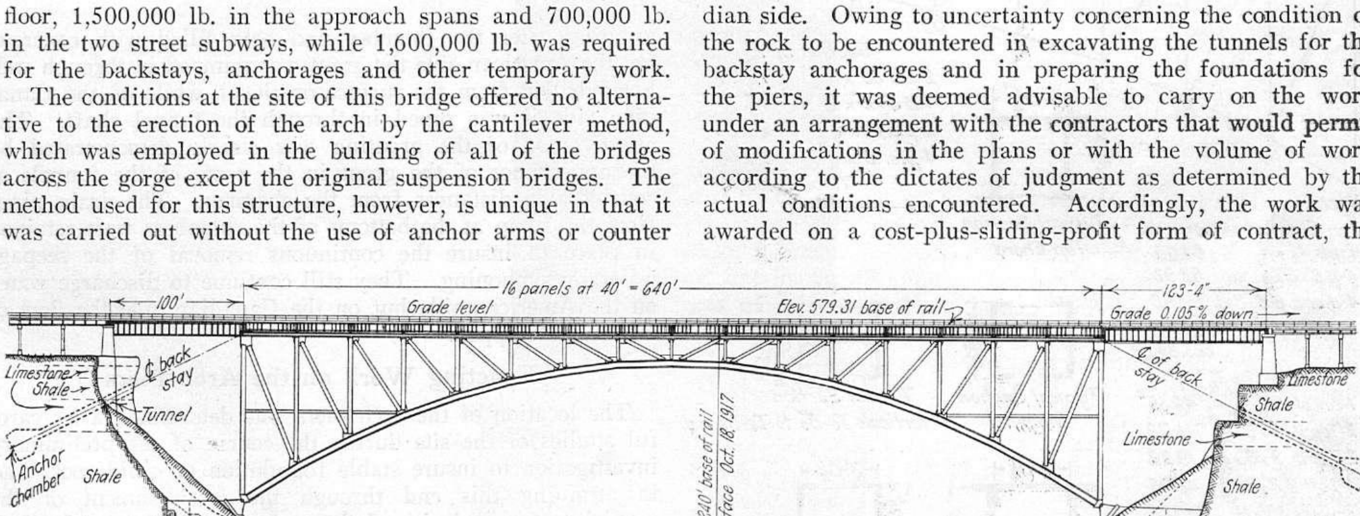
For a number of reasons, among which is the possibility of a subsequent raise in the grade of the tracks across the structure, the entire floor system is essentially independent of the trusses. The design provides for a ballasted floor with the ballast-retaining construction carried on a ⅜-in. floor plate resting on 15-in. I-beams spanning longitudinally between floor beams spaced transversely at intervals of 12 ft. 2 in. These floor beams are carried by two side girders which are supported on the trusses at the panel points by means of beveled castings which compensate for the inclined position of the top chords.

The arch bridge structure is flanked by a plate girder approach span at each end, that on the Canadian side being 100 ft. long and that on the American side 125 ft. long. These approach spans are provided with ballasted floors of similar construction. Beyond the approach spans at each end of the bridge the railway approaches consist of a number of spans of reinforced concrete viaduct terminating in street subways which carry the tracks over streets flanking the river on both sides. A total of 16,100,000 lb. of structural steel was required for the project, of which 9,600,000 lb. is in the arch trusses and bracing, 2,700,000 lb. in the arch



floor, 1,500,000 lb. in the approach spans and 700,000 lb. in the two street subways, while 1,600,000 lb. was required for the backstays, anchorages and other temporary work.

The conditions at the site of this bridge offered no alternative to the erection of the arch by the cantilever method, which was employed in the building of all of the bridges across the gorge except the original suspension bridges. The method used for this structure, however, is unique in that it was carried out without the use of anchor arms or counter



The New Arch in Relation to the Gorge

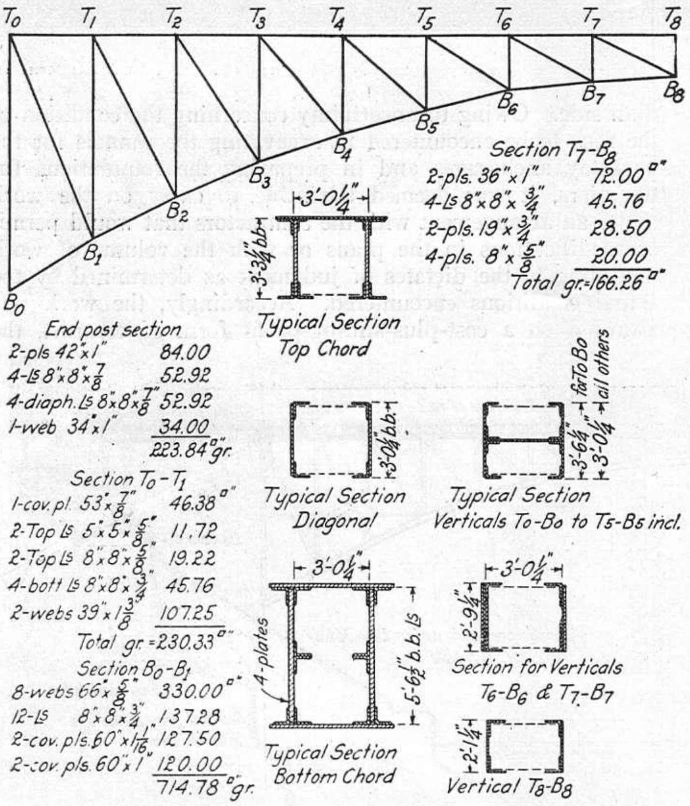
weight structures. Instead backstays were carried into anchorages imbedded in the solid rock of the bluffs, dependence being placed on the weight and strength of the rock ledges to resist the pull on the backstays.

The Substructure Work Was Difficult

The contracts for the sub-structure were let on May 9, 1923, to the Gass-Thurston Company of Detroit, Mich., for the work on the American side and to the Federal Construction Company of Toronto, Ont., for the work on the Canadian side. Owing to uncertainty concerning the condition of the rock to be encountered in excavating the tunnels for the backstay anchorages and in preparing the foundations for the piers, it was deemed advisable to carry on the work under an arrangement with the contractors that would permit of modifications in the plans or with the volume of work according to the dictates of judgment as determined by the actual conditions encountered. Accordingly, the work was awarded on a cost-plus-sliding-profit form of contract, the railway furnishing the cement, concrete aggregate, reinforcing steel and such falsework and blocking timbers as the railway's engineers deemed necessary. Other materials were furnished by the contractors and paid for by the railroad at invoice cost. The justification for this arrangement was borne out fully by experience throughout the course of the work.

Physical conditions at the site imposed a number of formidable obstacles to the prosecution of the construction, much of which involved operation on the steep faces of the gorge. The site also afforded little space for the storage of materials, necessitating provision for a storage yard at a considerable distance, from which materials had to be teamed. On the American side the presence of the gorge railway at the base of the bluff called for the exercise of extreme care in the conduct of the excavation to prevent large masses of rock from rolling down the steep slope onto the gorge line tracks. Considerations of safety in the operation of this line required the construction of a protection over these tracks similar in plan to a snowshed, the accumulation of earth on the roof of this structure providing an effective cushion to break the fall of occasional rocks.

The preparation of the site for the tunnels and the piers for the ends of the approach spans by the removal of the overhanging ledges of rock at the tops of the bluffs disclosed a fissured and broken condition of the rock face which was much more severe than had been anticipated as a result of the preliminary investigation. On the Canadian side this condition was met as regards the approach span pier by extending one corner of the pier far down the slope. On the American side it was found necessary to set the pier 25 ft. further back on the ledge with a corresponding increase in the length of the approach span. For the same reason the tunnels for the anchorage had to be driven to a greater depth than planned to insure that the anchorage



Typical Sections of the Main Truss Members

would be imbedded in solid rock. The tunneling operations involved the driving of four shafts 7 ft. wide by 6 ft. 6 in. high at a downward pitch of 26½ deg. from the horizontal for a distance of 82 ft. on the Canadian side and 105 ft. on the American side and terminating in anchorage chambers approximately 20 ft. deep, 20 ft. high and 17 ft. wide.

In addition to the difficulties attending this work, which were described above, the tunneling operations were subjected to a further obstacle by the opening of water bearing fissures in the rock, discharging approximately 40 gal. of water per min., which, in the absence of any opportunity for natural drainage, required the constant operation of pumps and steam syphons until the work had been completed. After much trouble the flow of water was confined to pipes draining into a sump at the bottom of the anchor chamber from which the water was drained by pipes connected with pumps outside the tunnel.

After the entire anchorage steel had been erected in the chamber and tunnels the excavation was back filled with concrete, especial care being taken to secure full bearing of the concrete against the roof of the chamber to insure that the uplift action of the anchorage would be effectively transferred to the overlying rock. This was effected by

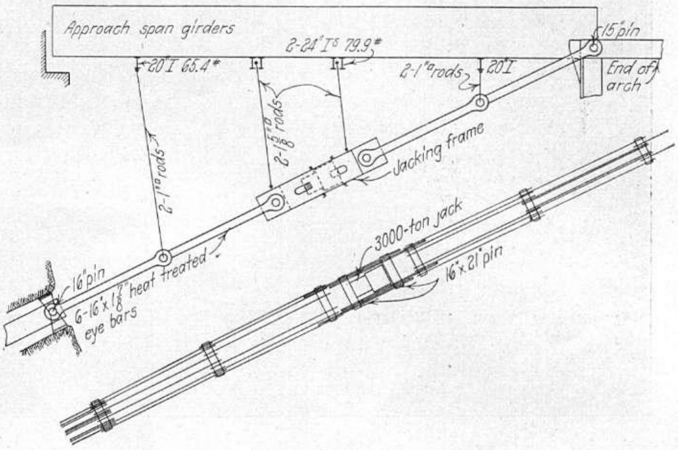


Diagram of the Back Stay Structure

grouting after the chamber had been filled with concrete. On the American side the grout was pumped in through well holes drilled from the surface overhead, while on the Canadian side it was piped in through the tunnel shaft. The effectiveness of the grouting was clearly demonstrated by the appearance of the grout in the roofs of the tunnels at considerable distances from the chamber. The drain pipes from the sump at the bottoms of the chambers were retained in place to insure the continuous removal of the seepage water by syphoning. They still continue to discharge water on the American side but on the Canadian side the flow of water has stopped.

Exacting Work on the Arch Piers

The location of the arch piers was determined after careful studies of the site during the course of the preliminary investigation to insure stable foundation on solid rock, and in attaining this end through the establishment of the requisite span and rise of the arch, the piers were fixed at an elevation that placed the footings well above the water level. The foundation work therefore consisted of the removal of the overlying talus and loose rock down to the solid ledge.

The exact pier locations were established by several independent triangulations from base lines on each side of the gorge, a feature of the work which was attended with considerable difficulty owing to the fact that irregular ground made it impossible to lay the base lines horizontal. The accuracy of the locations was checked by means of a tape line supported across the river inside of a pipe line suspended from the cantilever bridge. It was again checked by the engineers of the erecting contractor with a 1,000-ft. tape swung across the stream. However, after considering the various factors tending to vitiate the accuracy of the tape measurements under the unfavorable conditions imposed it was concluded that the triangulations insured much more accurate results.

By far the most exacting feature of the pier construction was the finishing of the concrete skew backs to receive the granite coping, the setting of the coping stones and the dressing of these stones to the required degree of accuracy with respect to location and surface. The specifications permitted a tolerance in the granite surfaces of only 1/16 in., as the correction of irregularities of contact between the skew back castings and the masonry was limited to the adjustment that could be obtained with a joint of 1/16 in. sheet lead. In carrying out this work the stones were set ¼ in. high and then dressed down by stone cutters, the final cut requiring a "10 cut" finish to come within the tolerance limit. This work entailed the services of a force of stone cutters for a number of weeks.

The concrete work, including the concrete trestle approaches, involved the placing of 14,000 cu. yd. of concrete, of which 5,700 cu. yd. was in the arch piers and 1,500 cu. yd. in the anchor pits and tunnels. The largest single unit was the south arch pier on the United States side which contained 2,162 cu. yd. Of this total, 1,743 cu. yd., comprising the neat work above the footing, was placed in continuous run.

Noteworthy Features of the Erection

The contract for the superstructure was awarded to the American Bridge Company and covered both the fabrication and the erection. The first step in the erection of the steel was the placing of the spans for the street subways in the approaches at each end of the bridge. Following this the anchor girders and the tunnel sections of the backstays were installed, after which operation had to be suspended pending the back filling of the tunnels and anchor chambers with concrete.

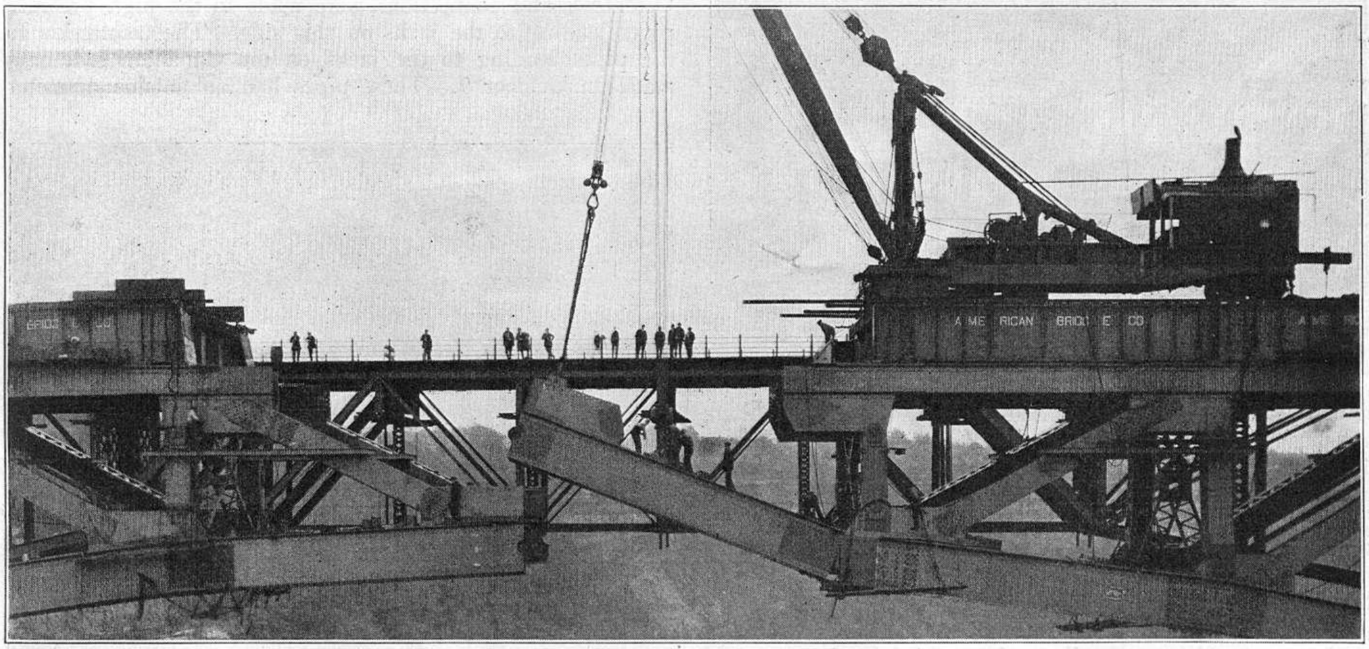
The procedure for commencing the erection of the two cantilever arms of the arch presented an intricate problem owing to the presence of the long approach spans at both ends of the arch. It was out of the question to place the skew-back bearings and the end posts of the arch trusses with the derrick cars standing on the concrete trestle approaches back of the approach span piers, because the reach was too great. On the other hand, to place the approach spans first introduced the problem of providing an outer support for them in the absence of the end posts of the trusses which were designed to carry them, while the alternative of providing falsework to enable the erection equipment to reach the ends of the arches would have involved prohibitive expense in view of the fact that some of the bents would have had to be over 100 ft. high.

The problem was solved by placing the approach girders first, providing a temporary outer support for them in the form of a temporary bent consisting of the posts for the second panel point (Canadian side) of the arch together with the cross bracing and a temporary base section for the columns. This bent was erected 84 ft. out from the approach pier on the American side and secured in place by struts and guys, after which the approach girders were set in place. This operation required the employment of two derrick cars and a carefully developed plan of procedure owing to the length and weight of the girders to be handled. The 125-ft. girders on the American side weighed 84 tons each.

With the girders in place the erecting equipment moved out on the span as far as the temporary bent and erected the skew back bearings, the end posts and the cross bracing, as well as the cross girder at the top, so that the approach girders could be brought to bearing upon their permanent outer support. This procedure was carried out first on the American side and after the temporary bent could be released and transferred across the river, the operation was repeated on the Canadian side.

The next step was the erection of the backstay system complete, including the suspender rods by means of which its dead weight was carried by the approach span. The weight involved was considerable, involving not only a chain of six lines of 16-in. by 1⅞-in. heat-treated eye-bars capable of taking a stress of 4,300,000 lb. for each truss, but also a jack of 3,000 tons capacity and the necessary jacking frame required for adjusting the length of the backstay in effecting the closing of the arch at the crown. Each backstay is connected to the arch by a 15-in. pin in the end of the top chord.

The erection of the two halves of the cantilever necessarily involved an adjustment of the position of the two arms so that the outer ends were 1 ft. 10½ in. above the normal position when the closure was made. The calculated gap between the ends of the cantilevers at the crown ranged from 7 9/16 in. to 15¾ in., depending on temperature and on the exact manner in which the stress in the backstay anchorage was transmitted to the concrete in the tunnel. In addi-

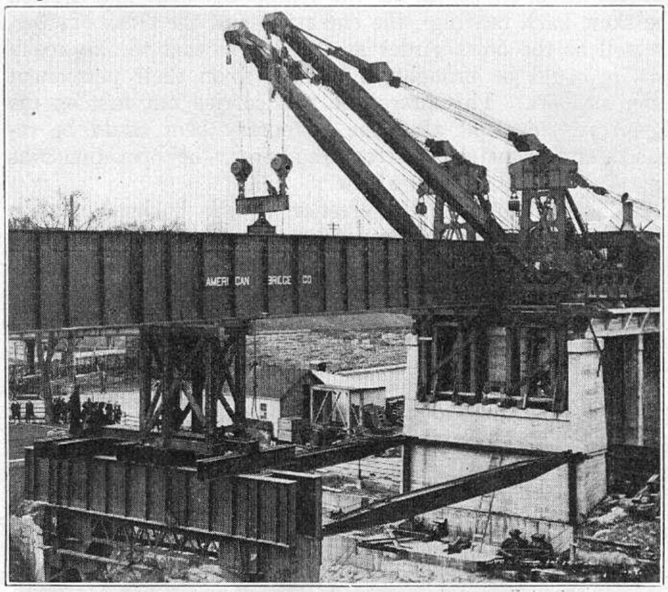


Placing the Closing Member in Bottom Chord of the North Truss

tion to the backing off of the backstays thus required to bring the arms to contact, calculations indicated that an additional adjustment in each backstay of from 2⅜ in. to 2 11/16 in. was necessary to relieve the strain on the backstays. The maximum required adjustment of each backstay at minimum temperature was computed to be 11 in. In order to provide the necessary separation of the cantilever arms at the time of closing, the end posts of the arch trusses were erected with a backward inclination amounting to 10 3/32 in. at the top. This in turn required the approach span girders to be pulled back the same amount behind their normal positions.

How the Arches Were Erected

The erection of the two cantilevers proceeded as nearly simultaneously as possible with one derrick car on each arm. As the structure was double-tracked, the cars alternated from one track to the other in placing the members in the two trusses, the members being brought forward on trucks on the other track. The normal procedure was as follows: After completing the erection of the floor for one panel, the derrick car, from its position on one of the tracks, erected the bottom chord section for the next panel on that side. Owing to the fact that the bottom chord splices are 100 per cent riveted the bottom chord members could be safely supported as a cantilever from the joint just as soon as the joint had been fully bolted and pinned and thus release the fall line for the erection of the diagonal which was placed next. Following this, the derrick car was transferred to the second track to place the bottom chord and diagonal on that side; the bottom laterals and strut and the post and top chord on that side. The derrick car was then moved to the first track for the erection of the post and top chord on that side, the cross frame and top laterals. The floor system for the panel was then erected from the same position of the derrick floor.



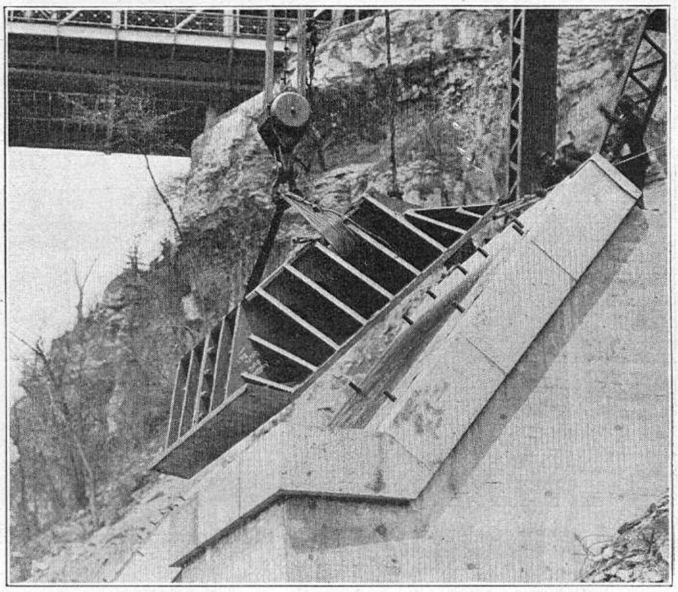
The Erection of the Approach Span Girders Was a Difficult Task

The Closing Operations

The closing gap between the ends of the completed half arches was accomplished by means of four hydraulic jacks, one for each backstay. The capacity of each of these jacks was 3,000 tons and they had been tested in the storage yard at the bridge site to 2,500 tons, which was about 25 per cent more than the required load. A jack was located at the center of each backstay between two pairs of sliding plates, of which one pair was in fixed connection with the anchorage, by way of the lower half of the backstay, and the other pair of plates was connected to the end of the top chord by way of the upper half of the backstay. Two pins, specially designed to bear on the plunger end and the base of the jack, transferred the stress from the jack to the plates. The upper of these pins was fixed to the plates attached to the anchorage and was sliding in slotted holes in the plates attached to the end of the top chord. The sliding condition for the lower jacking pin was the reverse.

At the time the backstay was erected the slotted holes in one pair of sliding plates were filled out to pin bearing with ½ in. shims, there being 28 shims in each plate, so that the jacking pin at this point transferred the backstay stress directly to the other sliding plate until the jack was put into operation. The jack then became a strut between the two jacking pins and first separated them sufficiently to release the bearing on the shims, which were then removed one by one while the plunger was retracted upward with the upper sliding plates, the lower pin and the shims, so that at no time was there a gap of more than ⅝ in. between the fixed plate and the shims.

The pumping engine for operating the four jacks, the boiler for furnishing steam to the pump, and the control levers and water connections to the four pipe lines leading to the jacks had been rigged up on a flat car which was moved out to panel point T7 on the American side on October 6 and connections were made to the high pressure pipes leading to the jacks on this side. The connection to the pipes leading to the jacks on the Canadian side was made on October 9. These pipes had an outside diameter of one inch and an inside diameter of ¼ in. The pump, which was specially designed for this job, was a double-acting, steam driven pump with four pistons, one for each of the four pipe lines leading to the jacks. Each of these pipe lines was fitted with a mercury gage on the connection to the pump registering the pressure on each jack and the control was so arranged that the jacks could be operated simultaneously or each separately as required.



Setting the Lower Bearing Casting on One of the Arch Pier Skewbacks

Telephonic connection was maintained between the engineer having charge of the removal of the shims. From the outer sliding plates in the backstays at each end of the bridge and the engineer in charge of the control of the pump at the center of the bridge. The engineer at the center of the bridge also had a full view of the ends of each half span, so that he was in direct control of all operations. Two valves were provided at each jack, one designed to act instantaneously and the other a safety valve with a 1/32 in. opening designed to release the pressure gradually. This was later found too slow and after having used it in testing out the piping and connections, it was removed for the final closing operation. The operating pressure was a little above 3,000 lb. per sq. in. The diameter of the jack plunger was 39 in., giving a total jack pressure of 3,600,000 lb., equal to the pull in the backstay after the derrick car had been moved off the bridge.

At 3:20 p. m., October 9, the American side was tried out by taking out one shim but on account of the small opening of the safety valve the half arch had only moved a fraction of an inch by 4:30 o'clock. The safety valves were then removed and the lowering of the American half of the arch was continued. By 5:30 p. m. the end of the half span had been lowered 4½ in. and moved westward two inches. Lowering of the Canadian half of the arch was started on October 10 at 7 a. m. and by 9 o'clock, four shims had been removed. The jacks on the American side were then put into operation and both halves of the arch were lowered at the same time until 11:30 a. m. There was then one inch clearance between the center pin and the pin hole at the crown of the arch. The operation of the jacks was then stopped and the ends of the trusses were

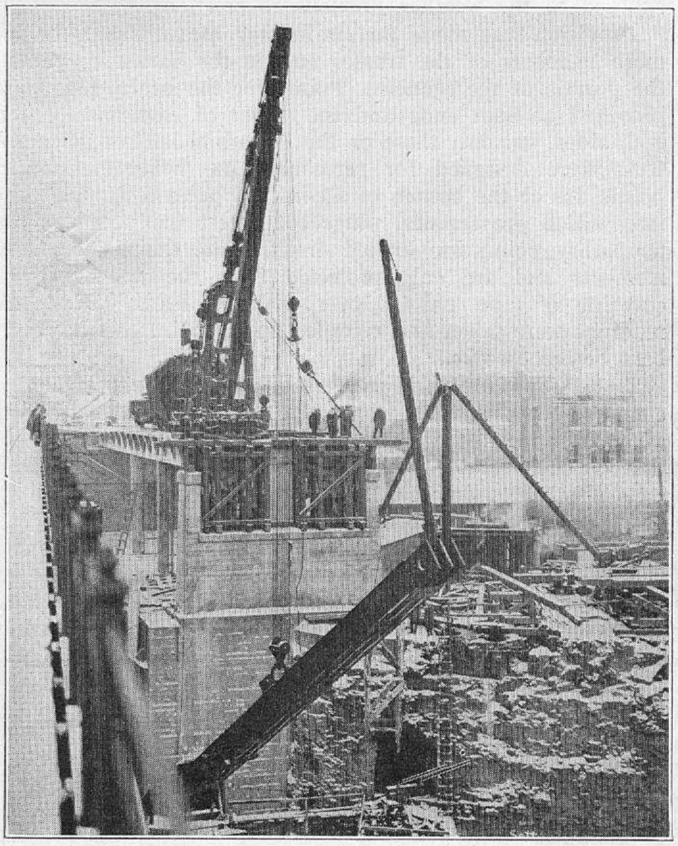
An Early Stage in the Erection of the Arch on the United States Side

lined up with little effort. As soon as this was done the placing of the splice plates at the center joint of the bottom chord was started. These plates were very difficult to enter and the work had not been finished by 6 p. m.

The placing of the splice plates at the center joint and bolting them to the Canadian half of the arch was resumed on the following morning.

The bottom chord bracing was also placed in panel B7 B8 of the U. S. half of the arch, which was connected only at panel point B7. Lowering was resumed in the afternoon but in pumping up the jacks a leak was found on the Canadian side and it was 4 o'clock before the final closing operation was started. The two halves then moved together at a rate of ½ in. in 6 min. When the pins were bearing and the pressure had dropped to 2,100 lb. per sq. in. the lowering operation was again stopped, holding the pressure on the jacks, while the lateral bracing was connected and the splice plates at the center joint were being bolted up, using 1-in. bolts in the 1¼-in. holes in the American side of the joint. By 5 o'clock this was finished and the slacking off of the jacks continued. By 5:05 the pressure on the jacks had dropped to 1,800 lb. per sq. in. and by 5:10 the pressure was zero and the structure had become self-supporting as a three-hinged arch.

While the closing operation took considerable time the whole operation was very satisfactory and the movements and deflections followed the calculated movements very closely. By October 15 the center posts and the two center



Installing a Part of the Back Stay in the South Tunnel on the Canadian Side. The North Tunnel Is Seen at the Right

panels of the top chord of the arch truss and the lateral sway bracing had been placed and everything was ready for changing the arch from the three-hinged to a two-hinged condition. For this purpose a screw 7 in. in diameter by 6 ft. long, provided with two nuts at each end, bearing on diaphragms riveted to the top chords on each side of the center joint had been furnished. The closing temperature desired was 60 deg., which was obtained at 4 p. m. on October 15, making it necessary only to screw the nuts up tight against the diaphragms and proceed with the drilling of the rivet holes which had been left blank in the top cord and in the gusset plate connecting the center post at B8 for this purpose on one side of the center joint. These holes have since been drilled, the rivets have been partly driven and the floor has been erected on the center panels so that the steel work is practically complete except for riveting.

The trusses were laid out at the shop with a camber equal to the deflection under dead load plus one-half of the live load on the whole length of the bridge. The full riveted splices in the bottom chord necessarily entailed long, heavy splice plates on both sides of the webs, but this did not lead to any difficulty in making the joints in erection.

The necessity for long rivet grips in many of the splices was made the subject of considerable study with particular reference to the difficulty of securing tight rivets and the tendency of the joint to "pack out" due to the thickness of the paint coating between the plates. A series of tests made by the railroad showed that the value of the interplate painting is largely destroyed by the heat of large rivets which burn out the oil, leaving only the inert pigment. This led to the decision to substitute a special coating known as "Hipo" oil for the shop coat on all field splices. This material gives a much thinner film over the plates, thereby favoring a much tighter packing of the splices, yet affording adequate protection to the steel for the short time that it is exposed to the weather previous to erection.

Provision was made for strain-gage measurements on all main members of the trusses and in the backstays and at the portals of the tunnels. Points for the strain gage were provided at both ends and the center of each member on both sides, but only those on the Canadian half of the north truss were designed for permanent use. The permanent points are at the bottom of ⅜-in. tap holes 5/16 in. deep, into which are screwed plugs to protect the points. The temporary points are drilled directly into the metal of the members and are only protected from the weather by a covering of tape, painted over with red lead. "No-load" readings were taken and recorded on all points of the members before erection.

In addition to strain-gage readings on the backstays, a complete set of measurements was taken of the position of all panel points in three dimensions after the complete erection of each added panel in the cantilever arms. These were checked against charts showing the calculated position of each point for the corresponding temperature and loading. To eliminate from these measurements the warping effect of unequal temperature on the two sides of the structure on account of sunlight, all measurements and readings were taken as early in the morning as conditions of light would permit.

The design of this bridge was developed under the direction of H. Ibsen, bridge engineer of the Michigan Central, assisted by C. L. Christensen, assistant bridge engineer. The project has been handled under the general direction of J. F. Deimling, chief engineer of the Michigan Central at Detroit, Mich. Olaf Hoff, consulting engineer, 50 Church street, New York City, has acted as advisory engineer on the project. The construction has been carried out under the direction of J. H. Curtin, resident engineer, and under the general supervision of Mr. Ibsen.

Annual Accident Bulletin for 1923

The Interstate Commerce Commission has issued Accident Bulletin No. 92, consisting of over 100 large pages, containing the record of collisions, derailments and other accidents occuring on the railroads of the United States during the 12 months ending with December 31, 1923.* The total number of casualties in the tables for 1923 is 179,097; made up of 7,385 persons killed and 171,712 injured. The various classes of accidents and casualties are analyzed in great detail, as in preceding years, with minute classification of causes of accidents. Most of the totals of 1923 are larger than those of 1922. The business done by the railroads was greater, as measured by the number of locomotive miles, which in 1923 aggregated 1,813 millions; while in 1922 the total was 1,600 millions; and, as stated in the bulletin, the "increased exposure" in 1923 is generally reflected in the statistics.

To this, however, there is an exception, the item of passengers killed and injured. The decrease in passenger fatalities is called "a conspicuous feature," the total, 138, being the lowest on record, though the number of passenger miles was 7.2 per cent more than in 1922. A table is given showing the total number of passengers killed from all causes, for a series of years. A part of this table is quoted below (Table D) the figures taken from the bulletin being shown in column A. The number of passengers killed in

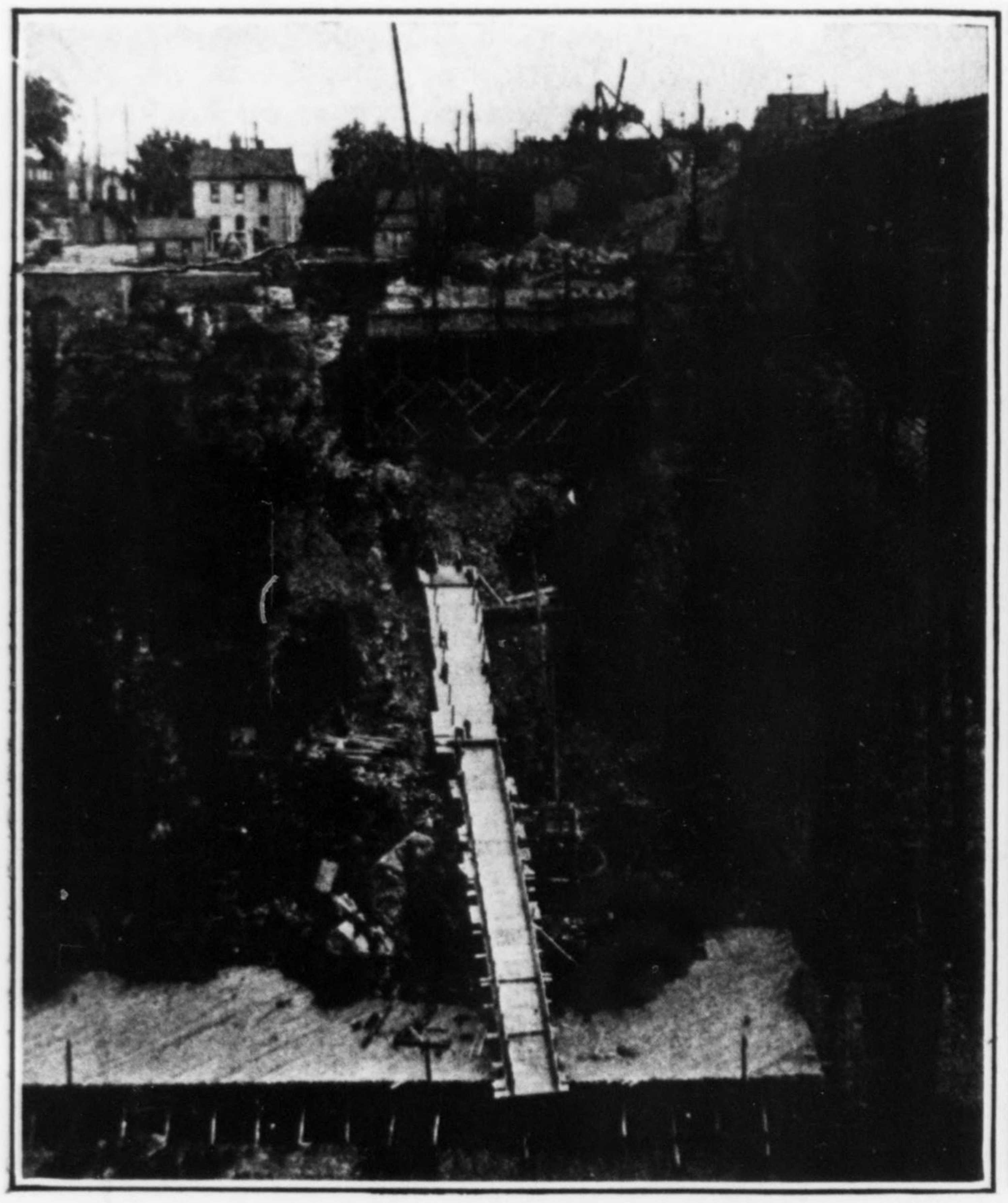
* The last annual statistical bulletin, No. 87, was noticed in the *Railway Age* of September 29, 1923, page 594, and the one preceding that on October 7, 1922, page 652.

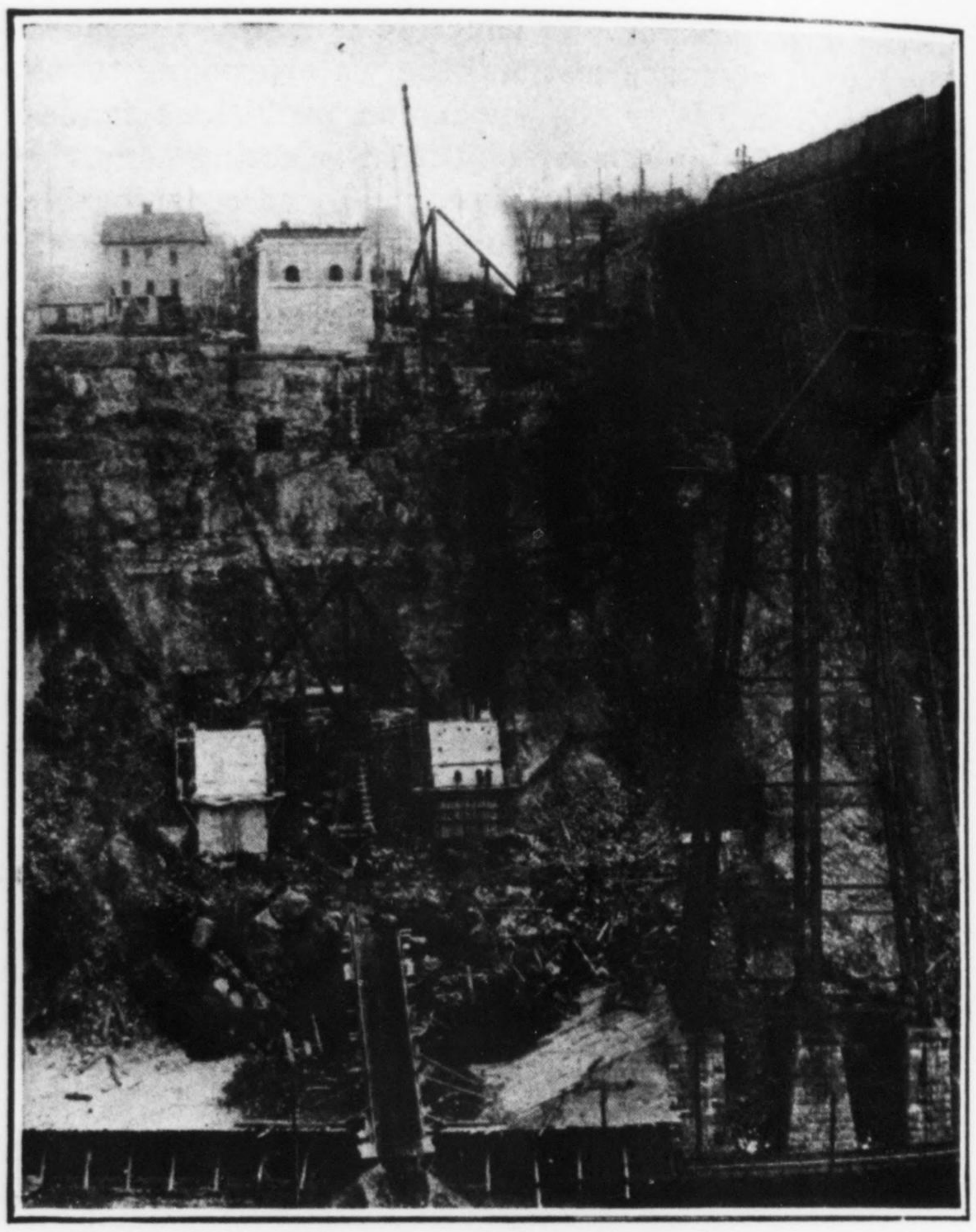
Michigan Central Niagara Bridge Abutments Completed

Excavation and Concreting of Abutments and Piers on American Side of Gorge Done in Six Months

IN LITTLE over six months' time the substructure work for the new bridge of the Michigan Central R.R. across the Niagara Gorge at Niagara Falls, N. Y., has been completed. The structure will be a steel spandrel-braced arch of 640-ft. span, as described in *Engineering News-Record* of March 1, 1923, p. 380. Four heavy concrete blocks constructed at the edge of the principal hard rock stratum at the sides of the gorge form its foundation. The beginning of work on the American side and the appearance of the completed abutments are shown in Figs. 1 and 2, herewith. The last operation, setting the granite coping, is represented in Fig. 3.

Contracts for the substructure work were let on a basis of cost plus sliding-scale profit, because the nature of the work made it desirable to keep full control of the execution in the hands of the railroad company. The Gass-Thurston Co., of Detroit, and the Federal Construction Co., of Toronto, took the contracts for the American and the Canadian work, respectively. Work was actively in progress by the beginning of June, contracts having been let on May 7, and thereafter it was carried on generally with both night and day shifts. Delay was caused by right-of-way difficulties and by the necessity of storing material at some distance from the site. Further delay occurred because, after the rock on top of the bluff had been cleared off preparatory to removing the overhanging rock at the rim, cracks and fissures were found farther back than expected, particularly on the American side, so that a great deal of rock had to be removed. Abutment excavation could not be carried on safely while this was in progress. The work was done with the aid of staging carried up so as to furnish support for the debris and prevent material falling down the cliff. On the American side the Niagara Gorge railroad track made this protection particularly necessary. On this side, also, the extra removal of rock increased the length of the approach span from 100 ft. to 125 ft., and required the anchor tunnels to be carried much deeper than anticipated.





FIGS. 1 AND 2—ABUTMENT CONSTRUCTION, NIAGARA GORGE ARCH OF MICHIGAN CENTRAL R.R.; AMERICAN SIDE

View at left, June 27, 1923, shows staging at rim of gorge for safe removal of overhanging ledge, and shelter shed over Gorge R.R. tracks at foot of cliff.

View at right, Dec. 12, shows completed abutments. Tunnels near top of cliff are to receive erection anchors. Approach viaduct nearly completed.

Excavation and concreting for the abutments went on rapidly after the work above was completed, and the stone copings were set in December, this part of the work being done under a separate contract by the Arthur McMullen Co. of New York.

An unforeseen amount of water was encountered in the inclined anchor tunnels, in which the girders and chains for the erection backstays are now being placed. Water came in at the rate of about 40 gal. per minute, and pumps had to be kept going continually. After much trouble it was possible to confine the flow to a few pipes, leading to a sump at the bottom, whence it is pumped. With this arrangement it will be possible to place the concrete without interference by water after the girders and eyebars are erected. This steel is now being placed.

FIG. 3—SETTING GRANITE COPING, SOUTH PIER

James H. Curtin is resident engineer for the railroad on this work. H. Ibsen is special bridge engineer and J. F. Deimling chief engineer of the Michigan Central R.R. Olaf Hoff is consulting engineer for the bridge.

Severe Water Hammer Causes Break of Big Creek No. 8 Penstock

TECHNICAL study by the Southern California Edison Co. engineers of the penstock break which occurred at the company's Big Creek No. 8 hydroelectric plant on Jan. 1 (reported in *Engineering News-Record*, Jan. 10, p. 92) has developed the fact that the accident was due to an unprecedented combination of circumstances. As the details of the happening are of general importance to hydraulic engineers, the company has made them available for publication. A noteworthy feature of the case is that, while the accident abruptly took out of service the 30,000-hp. capacity of the plant, electric supply on the entire transmission network continued uninterrupted; few consumers even knew that the system was meeting any unusual condition. Moreover, by remarkably rapid work in planning and executing the penstock repair, the plant was put back into service within two weeks, a feat that was made particularly difficult by the remoteness of the plant, which is located in the mountains 250 miles from headquarters.

Conditions at Time of Break—Just before the break the 30,000-hp. unit (only one unit is installed at this plant) had been shut down by closing the guide vanes of the turbine. The Johnson balanced valve, which is provided at the lower end of the penstock as a shut-off, was left open, and for the time being the water flowed through a pressure-relief valve connected to the turbine casing, which opened automatically as the turbine gates were closed. This valve was closed by action of a slow-moving dashpot, and ordinarily would have remained closed, when the Johnson valve could have been closed safely under no flow. But on the present occasion the automatic control of the pressure-relief valve had been shut off, in order that the draft tube would be unwatered for inspection, and therefore until the Johnson valve could be closed the pressure-relief valve was to be kept closed by water pressure brought through a small pipe controlled by a manually-operated valve.

This latter valve the operator opened, preparatory to closing the Johnson valve; but in doing so he neglected to close a second valve in the same pipe system leading to an ejector. This neglect allowed the pressure-relief valve to open, contrary to the operator's expectations. In consequence, when he later undertook to close the Johnson valve in a way which would have been quite safe under balanced pressure, the full force of flow through a 6-ft. pipe under a head of 680 ft. caused the valve to slam shut, setting up a severe water hammer in the 2,800-ft. penstock.

The Effect—The sound of the sudden closure of the Johnson valve was heard some distance from the plant. It was followed by two other reports, presumably caused by the rupture of the penstock. The penstock broke in two places, one of them about a fourth of the way up, the other in the anchor section just outside the power house. Both sections consisted of lap-welded steel pipe, and in both cases the ruptures were clean breaks.

The upper break tore the pipe open for the length of one section, by a longitudinal rupture on the side opposite the longitudinal weld, crossing the circumferential weld approximately at right angles. In the break near the power house the lower end of one of the thickest sections of the penstock, which was further strengthened by being embedded in a concrete anchor block, was cracked for a length of about 5 ft. The greater portion of this crack was in that part of the pipe embedded in the concrete block. Outside the anchor block the break extended through the cast-steel flange of the pipe, which is some 4½ in. in thickness.

In addition to the loss of two lives, previously reported, the damage was limited to the penstock itself.

Repair of the Break—In the case of the upper break the ruptured section of the pipe was removed bodily from the line and was replaced by a new section of the same dimensions riveted up in Los Angeles on a rush order. The lower break was repaired by attaching a butt strap over the crack inside the pipe. The butt strap, cut from the upper ruptured section, was bolted in place inside the pipe and was then welded to the pipe shell along the edges. External bands were applied to the pipe to strengthen the repaired section still further, giving to the finished job a greater strength than it had before the break.

French Architects and Engineers Disagree

In the city of Paris, France, a discussion is going on regarding the relative functions of the architect and the engineer in municipal work. The building and construction staffs of the city consist of various technical services—subways, sewers, streets and bridges being all separate, and finally an architectural department. This latter department now has charge of the construction of a municipal swimming pool and is paid by commissions of 5 per cent on the cost of the work it supervises. The city engineers, on the other hand, are paid salaries and are required to go through all the public formalities of advertising and letting contracts. The architects are permitted to enter into private contracts with builders. In the case of this swimming pool, charges are being made that the architects have been unduly extravagant and have let the contracts to favored bidders without the public advertising that is required for engineering work. As a result, there is in progress an unpleasant controversy between the municipal architects and engineers.

Demolition of Niagara Falls Cantilever Bridge

Expeditious Fieldwork in an Unusual Undertaking Complicated by Difficult Location—Three-Truss Bridge Taken Down by Reversal of Cantilever Erection Method

By H. Ibsen
Bridge Engineer, Michigan Central R.R. Co., Detroit

On Feb. 16, 1925, the new 640-ft. steel arch bridge carrying the Michigan Central R.R. across the Niagara Gorge at Niagara Falls was opened to traffic. Two months before, a contract for the demolition of the old cantilever bridge, alongside, had been let to the American Bridge Co., on a cost-plus-sliding-profit basis. This demolition work, completed on Sept. 1, will be described as an interesting instance of dismantling by reversal of the cantilever erection procedure.

While falsework would be costly, considering the expense in preparing good foundations, the cost of the special traveler and the extra handling of the material would probably balance most of this, and in addition the second method involved greater risk of accident. For these reasons it was decided to adopt the first method.

Falsework—The wooden falsework (Fig. 2) was framed in the storage yard. Each 30-ft. bay was spliced on the center line of the bent, so that each half bent section could be shipped connected up from the yard to the bridge on flat cars and there picked up by the crane and lowered to its position in the bent. Heavy struts following the slope of the bank were wedged between the feet of successive bents, butting against the tower piers, to prevent any downhill movement of the bents. The foundations had to be dug much deeper than shown on the drawing, and on this account the bottom bays of the bents had to be much longer and they were set on a continuous bed of mud sills. Work was started on the bent next to the tower and the mud sills and bottom bay of the bent were placed before starting excavating for the next bent up the hill. The excavated material from the next footing was then wasted down hill, so that it buried the footing of the bent below by 8 to 12 ft., dependent on the soil conditions found. All together about 5,000 cu.yd. of material had to be moved, much of it being boulders, some so large that they had to be drilled and broken up before moving.

As a matter of economy the falsework used on the American side was used over again for the Canadian side. This increased the time for the work about three

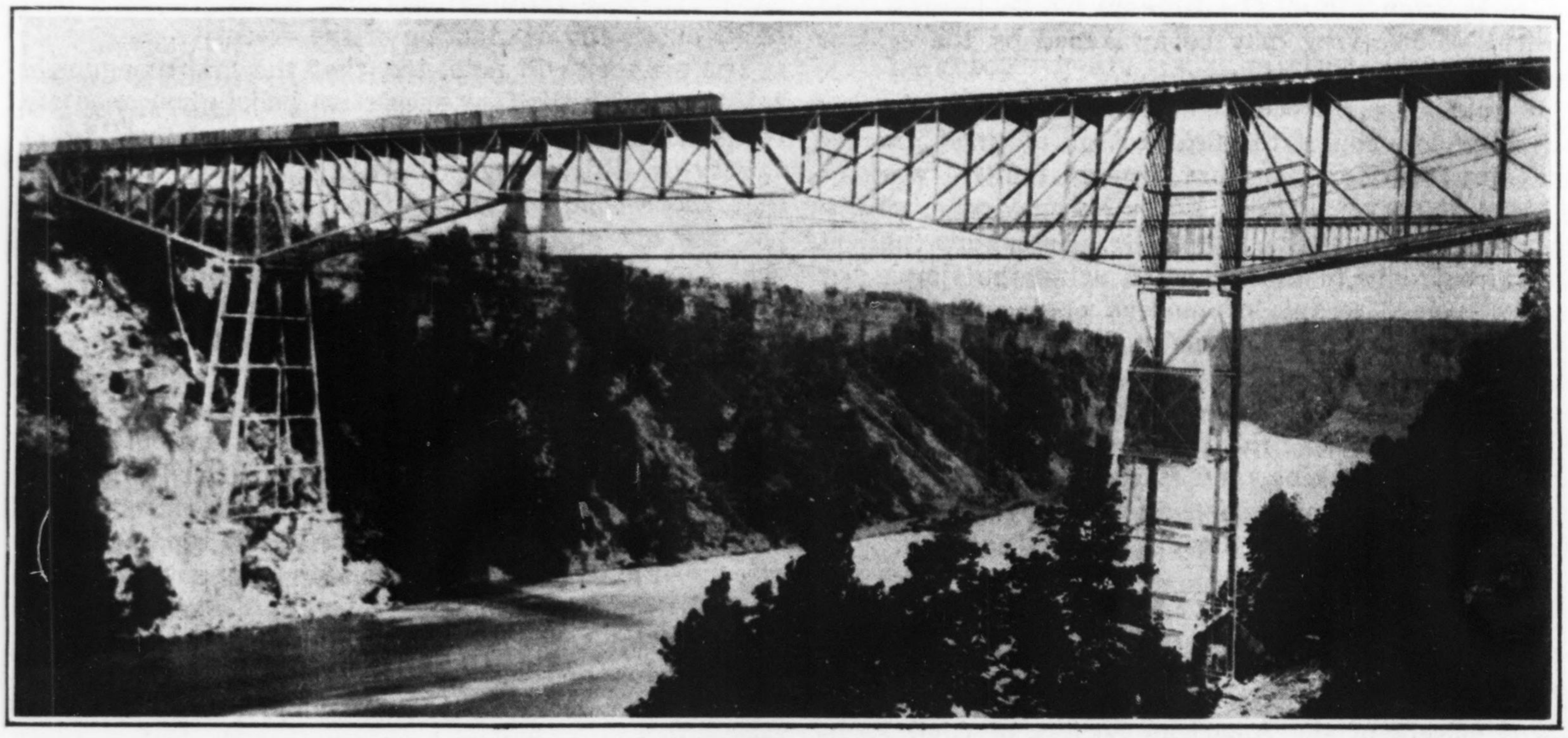


FIG. 1—NIAGARA CANTILEVER BRIDGE AS ORIGINALLY BUILT
Old suspension bridge in background replaced in 1897 by 550-ft. double-deck steel arch built for both railway and highway traffic. Cantilever replaced by 640-ft. steel arch opened to traffic Feb. 16, 1925

Both the cantilever and the new arch were described in *Engineering News-Record* last year (March 1, 1923, p. 380, and March 6, 1924, p. 398). A view of the cantilever soon after its construction in 1883 is given in Fig. 1, from an old photograph, and an outline elevation is shown in Fig. 2.

Choice of Method—Two methods of demolition were considered:

First, Reversing the method used in erecting the bridge, that is, erecting falsework under the shore arm and starting at the center of the suspended span, taking out the bridge piece by piece with a self-propelling crane, loading the material on cars as taken out and backing toward the shore;

Second, Removing the bridge without falsework under the shore arm, which would mean balancing the two arms on the towers, as first one panel in the river arm was removed and next one panel in the shore arm was removed.

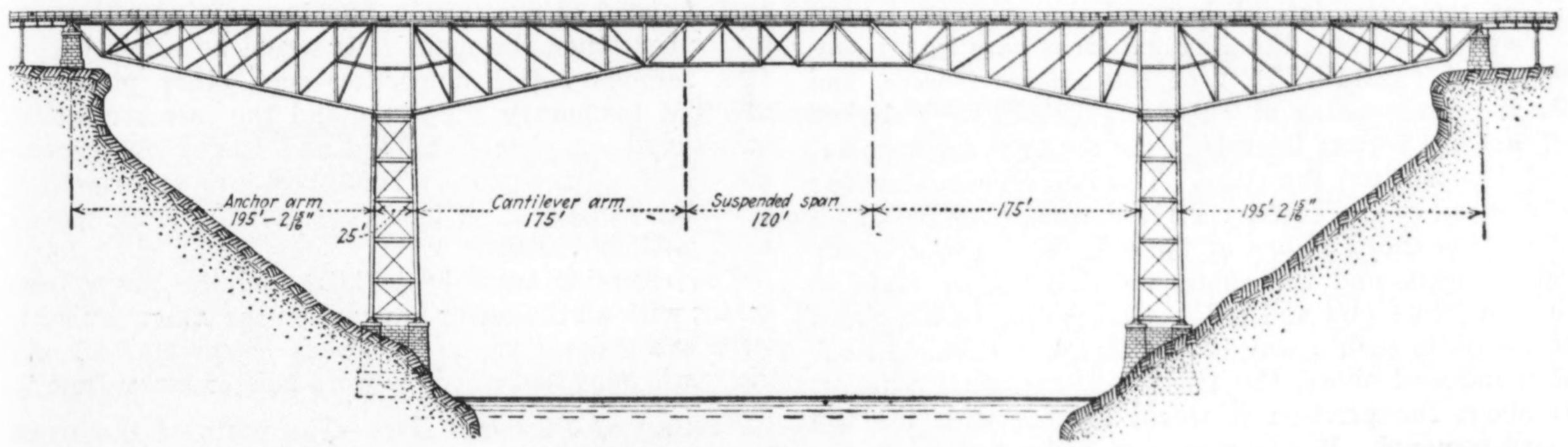
The second method would have meant building a special traveler with booms at either end and lowering the material down on the steep river bank, snaking it up the bank and hoisting it from the foot of the bluff to track level for loading.

weeks but it saved about 200,000 ft. b.m. of timber and the labor of framing. Falsework construction was started at the end of February, 1925, and carried on as the weather permitted. Removal of the superstructure began in March.

Preparing the Bridge—The first thing done was to rearrange the bridge floor so as to have one track on the center line of the bridge. In 1901 an extra line of stringers had been added under each track between the two old stringers; the outside line of old stringers were therefore removed, and the two lines of 1901 stringers were moved in, close to the inside lines of old stringers, and ties were placed close together on these four stringers, which gave a good support for the heavy concentrated loads of the demolition cranes. The members of all three trusses could be easily reached and handled from this center track and, as the crane could swing clear around, the car to receive the members removed could be placed back of the crane. Except for some of the 1901 stringers, the material could be used only for remelting; therefore the members were all burned apart to suitable dimensions for shipment, and the material was sold as loaded, so that no rehandling was necessary.

By the end of April the rearrangement of the floor was completed and the falsework (under the American anchor arm) had been finished. To bring the trestle bents to bearing, the pressure from a 35-ton jack was applied under each panel point of the trusses, which closed up the joints between the timbers of the bents and brought the footings to a good bearing. There was very little settlement after the falsework carried the full load of the shore arm.

For removing the bridge members two self-propelling cranes weighing 195,000 lb. distributed on four axles were used. The heaviest member to be removed weighed 26,000 lb., and this the cranes could safely handle.



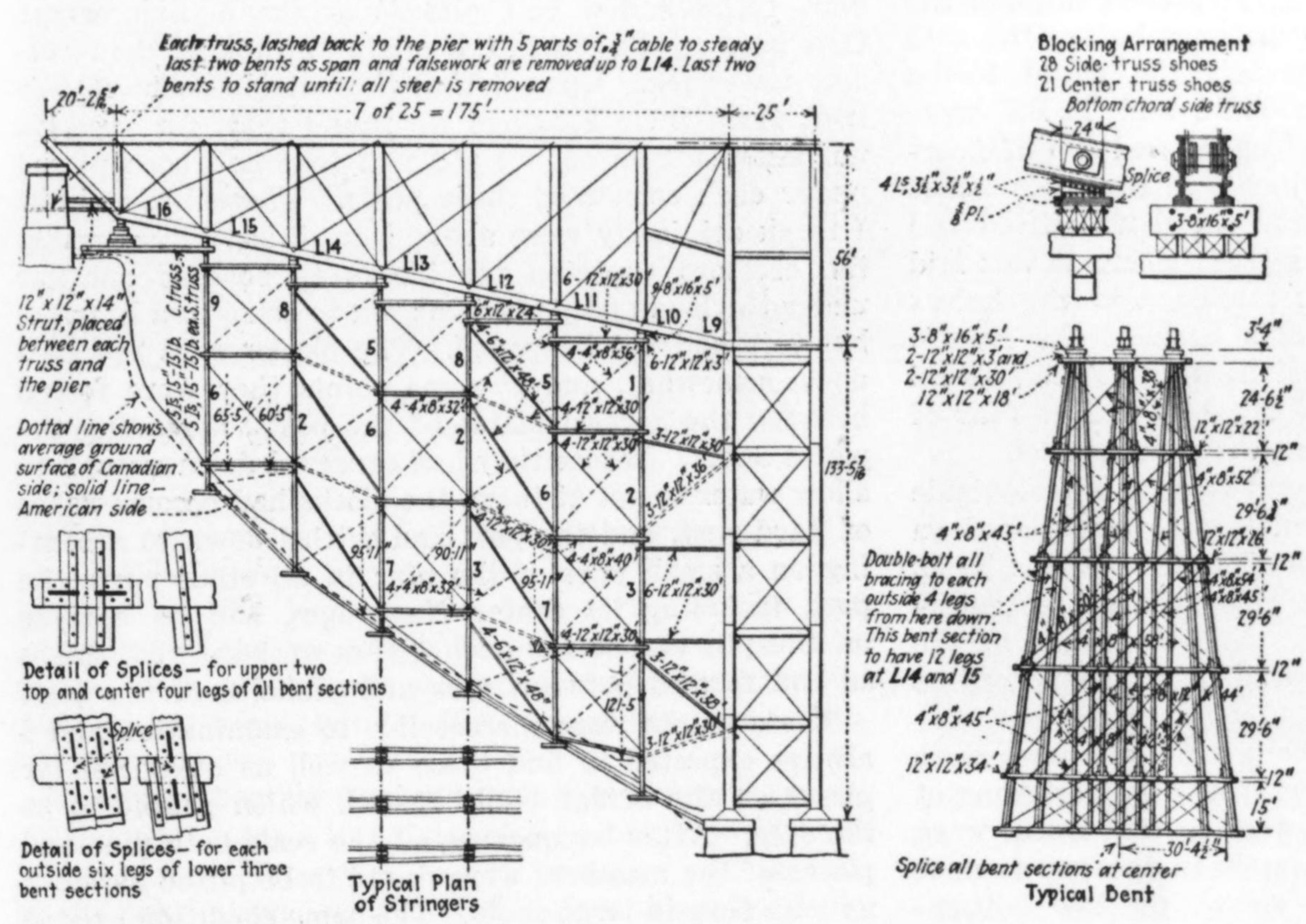


FIG. 2—MICHIGAN CENTRAL CANTILEVER BRIDGE, AND DEMOLITION FALSEWORK

It was desirable to keep the work on the Canadian side going while waiting for the falsework. But in the bridge as built this met with the difficulty of stress reversal in the top chord of the anchor arm as soon as the suspended span had been removed, at which time the compressive stress set up by the crane with a loaded car moving over the shore arm would overcome the dead-load tensile stress due to the weight of the river arm. The old (outer) trusses had a top-chord section consisting partly of eyebars and partly of a built-up member, which latter was stiff enough to take care of the compressive stress, but in the center truss (a later addition) the first three panels back of the tower were made up entirely of eyebars and therefore could not be relied upon to take the compression. To correct this condition closefitting wooden blocks were driven between the eyebars at intervals of 2 ft., and 1-in. rods were placed above and below the eyebars at the blocking to tie them together. The three panels were thus made safe for compressive stress, so that the entire river arm could be safely removed and the loaded cars switched over the shore arm by the crane.

Taking Down Suspended Span—While this work was being done the floor and stringers had been removed from the suspended span. The outside trusses of the suspended span were connected to the ends of the cantilever arms by struts with slotted pinholes in both top and bottom chord, but the center truss had such a strut only at the bottom chord and no tie in the top chord. The slotted pinholes in the struts were tightly wedged with shims cut to fit the pinholes, and the hip joint of the center-truss top chord was tied to the end top-chord pin of the cantilever with six lines of ¾-in. wire cable. To stiffen the center truss while the side trusses were being removed, timber struts were placed between the top-chord hip joint of the center truss suspended span and the end joints of the outside trusses of the cantilever arms.

The top-chord lateral bars of the suspended span were now removed; the diagonal truss bars in the center panel (T2B2–T2′B2′) of the outside trusses, and the rod swaybracing at T2B2 and T2′B2′, were slacked off, and the bottom laterals in the center panel were cut and dropped into the river. The two cranes, standing at the end of the cantilever river arms, then took hold of the top chord of one of the side trusses of the suspended span and the chord was burned off close to the hip joints (U1 and U1′). The cutting of the chord was done in such a way that with the rods slacked off, as mentioned above, the truss would gradually deflect to about the position it would assume with the top chord removed. It was necessary to burn through the chord in the same place several times before the cuts remained open enough to permit the chord to be removed. The vertical posts T2B2 and T2′B2′ were then cut between underside of chord and top of floorbeam (the floorbeams were riveted to the posts about 6 in. under the chord), and the chord was lifted and swung over close to the center truss where it was laid down, burned through at its middle, and the halves were removed to the scrap cars back of the cranes. Then the center panel diagonal bars T1B2′ and T1′B2 were cut and dropped into the river as well as four of the six bottom-chord eyebars in the center panel.

The same process was repeated for the other outside truss. Now temporary wooden struts were placed on top of the floorbeams T2B2 and T2B2′ in panels T1T2 and T2′T1′, to hold bents in place when the center-truss top chord was removed. The top chord of the center truss was cut and removed in the same way as the outside trusses.

Posts T2B2 and T2′B2′ in both outside trusses were taken out and loaded, and thereupon the floorbeams at T2 and the same posts of the center truss were removed. Then the bottom laterals in the panels next to the center panel and the remaining two bottom-chord bars in the center panel were burned off and dropped into the river. The last chord bars were burned through gradually, with the idea that the bars should stretch when the remaining section became small enough, if there still was any appreciable amount of pull in them, so that they would not break suddenly producing impact in the struts supporting the remaining parts of the trusses. The precautions taken were effective and there was very little movement when these members were finally cut through and the center panel of the suspended spans was completely removed. (In all the work over the river the lighter members, such as bars and small struts, were dropped into the river when their value as scrap was not enough to pay for the labor of handling and loading them.)

Two days' more work removed all of the suspended span except the hanger rods and bottom struts at the ends of the cantilever arms, and the cranes were moved back to U2 ready to remove the first panel of the river arm (Fig. 3).

Rapid Work on River Arms—Removal of the river arms was commenced May 6, and was completed by May 26, when the tower panel on the American side was taken in hand. On the Canadian side two panels remained, one of which was left in place until the falsework was set under the anchor arm.

There was nothing to deserve detailed description in the work of removing the river arms and the towers. It became a work of routine. The burners would keep as far ahead as practicable, burning part way through a member; then the main falls would be hitched to it (the secondary fall supporting such other pieces as required temporary support), and the member would be severed completely, hoisted and loaded on the car placed behind the crane. This procedure was repeated until the panel was all taken out and the crane moved back another panel.

The riverside truss bent U8L8 and the tower bent below, with all the tower bracing, on the American side, were taken down next. The work began May 26 and the castings at foot of tower were hoisted away June 5.

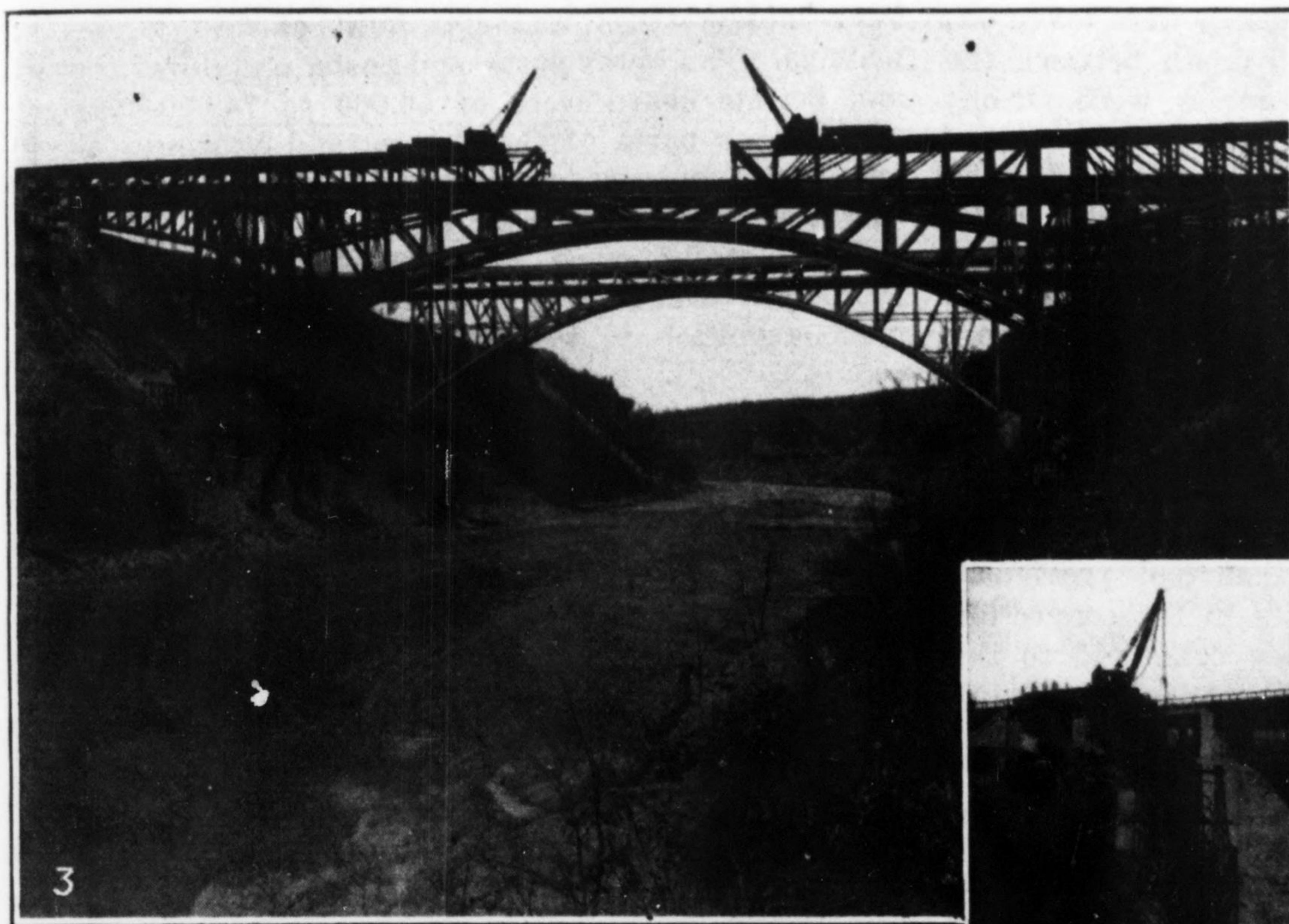
Rusting of Pins and Bars—The posts of the truss bent U8B8 had a half pinhole at the bottom, where they bore on the pin in the casting on top of the riverside tower leg. Upon lifting the outside posts of this bent from their bearings we found that, for erection purposes, a 1-in. square piece of iron had been placed under each corner of these posts. These little blocks I think originally were about ⅜ in. thick. Presumably the erectors expected the blocks to squeeze out flat under the load and permit the pin to come to a bearing, but this had not happened. The blocks were squeezed down somewhat, and at some points they were forced between the several plates of the posts, spreading the plates apart. Where they had squeezed down enough to allow water to get at them, the blocks had become flakes of hard rust, and the post had settled down to a bearing on a small part of the pin; in all other parts the post, including its center diaphragm, had no bearing on the pin except for such layers of hard rust flakes as had formed between pins and pinholes of the post.

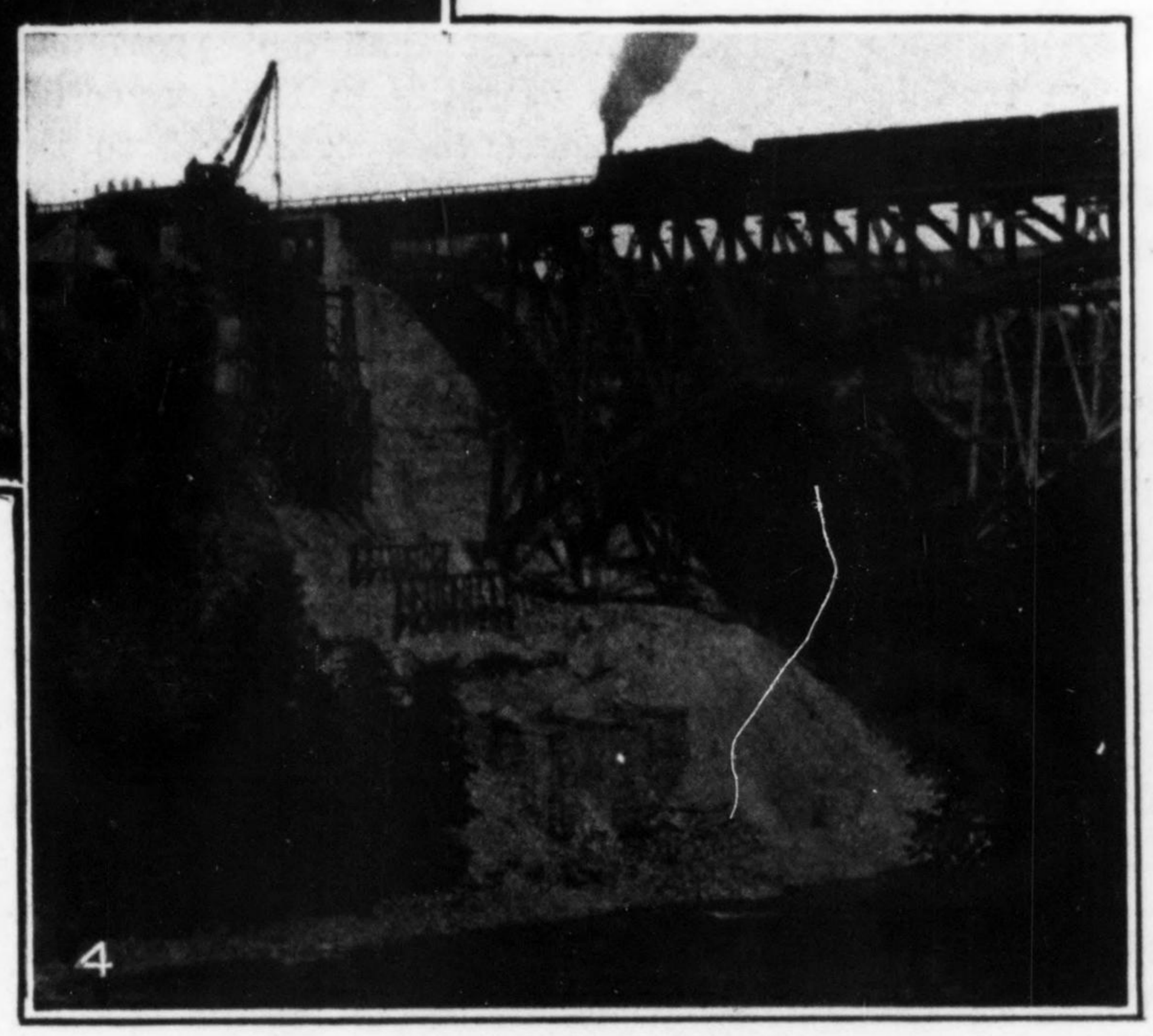
These points were inaccessible to examination and I always expected to find these as well as other similar places on the bridge badly rusted, which proved to be the case. After hammering off the scale both pins and plates of the members were found to be pitted as much as ⅜ in. deep in large spots. The same condition existed on the Canadian side.

Except for the outside top chords of the river arm, where the chord was a combination of a built-up member and eyebars and where for five panels the reversals of stress were large and there had been considerable wear in the pinholes, the eyebars in the chords and diagonals were rusted so firmly to the pins and to the adjoining bars—especially where, as in nearly all cases, they were packed so tight as to be inaccessible for cleaning and painting—that when one end of the bar was burned off the weight of the bar would not cause any movement on the pin at the other end. These bars were 8x1¾ in. in section and 22 ft. long from center of pin to the burned end. In one case I tried a joint, with eyebar stubs sticking out 3 ft. from the pin, by first striking the bars numerous blows with a heavy maul to loosen the rust and then hitting the end of the bar. Treated in this manner it moved only $\frac{3}{32}$

in. in six blows of the maul. In driving some of the bars off the pin they were found encrusted with scale to a depth of nearly ½ in. in spots, so that it would have been a very slow and expensive job to drive out the pins, which were 6 ft. long and carried 32 eyebars.

Work in Anchor Arm—On June 8 the crane was moved back off the river arm and the diagonal bars U9L10 from top of shore tower bent to the bottom of the next panel were cut, transferring most of the weight to the falsework. The crane was then moved back to panel point U10 and took out the top chord in panel U9L10 and the diagonal bars U9M10, which brought all the load on the falsework. Levels and alignment of trusses and falsework, observed regularly as the work progressed, showed a small settlement in some of the bents, but too little to require any correction to be made.





FIGS. 3 AND 4—DEMOLITION WORK: SUSPENDED SPAN REMOVED (MAY 25)—LAST TWO PANELS BEING TAKEN DOWN (AUG. 27, 1925)

The shore bent of the tower and the first truss panel were down by June 16. From this point the work went slower, as the falsework had to be taken out as soon as the panel above was removed, before the crane could move back.

By July 13 the shore arm on the American side and all of the falsework that had to be used over again on the Canadian side had been completely removed and on July 25 the falsework on the Canadian side had been completed and work started on removing the shore arm, tower and the one panel remaining of the river arm of the cantilever on that side of the river. The Canadian anchor bars, which were the last part of the main bridge, were removed on Aug. 29, and two days later the falsework had been removed and the work was completed except for the removal of the street bridges and for the cleaning up of the bridge site.

Some Service Experiences of the Bridge—One source of trouble with the old cantilever bridge was the floorbeams (see a reference to this trouble in *Engineering News-Record*, March 1, 1923, pp. 380-381). Where these were riveted to the posts of the outside trusses, the end angles connected direct to the floorbeam webs and did not, as in modern construction, extend over the legs of the flange angles of the floorbeam. In addition to this the top lateral rods were attached to the floorbeams through a pin, the center line of which was 20 in. below the center of the chord and 21 in. from the center of the truss. The pin was connected to the floorbeam by a plate fastened to the top flange of the floorbeam and by a bracket riveted to the web of the beam below the flange, and thus there was no means of transferring the stress from the laterals to the chord except through the floorbeam. In 1907, to take care of the horizontal component of this stress, a lateral chord was attached to the top of the floorbeam for four panels each side of the tower. While this helped the conditions for which it was put in, it rather aggravated other defects.

The floorbeams were riveted direct to the posts of the outside trusses as described above, and when the new center truss was added (in 1900) the new posts were put in without cutting the floorbeams, except in the tower bents. Shims were placed between the floorbeams and a seat on the new center-truss post to transfer to the center truss its computed share of the deadload. Then the floorbeams were riveted to the center trusses, making them continuous over the center truss and thereby imposing upon them the additional duty of helping to maintain equal profiles of the three trusses under live-load. In addition to this there were four lines of stringers under each track, resting on top of the floorbeams. These were supposed to slide freely on their bearing plates, but in spite of raising the ends from time to time and greasing the bearing plates, their movement was not entirely free. As the stringers and the added lateral chord did not take part in the movement of the trusses under live-load, they restrained the floorbeams and produced transverse bending stress in them.

These causes combined had the effect of cracking the webs of the floorbeams, the break generally starting just above the end connection angles. In 1911 the breaks had become so numerous, and were so rapidly getting worse, that repairs had to be made. This was done by first replacing the brackets, under the floorbeams, which formed the swaybrace connection to the floorbeam and the truss post; the new brackets were stronger and deeper than the old ones and had enough connection rivets through the post to support the floorbeam load. Then the end angles and the bracket connecting the top laterals to the floorbeam were taken off, and heavy plates, running the full depth between the horizontal flanges of the floorbeam angles, were put on; the ends of these plates were bent to form angles fitting against the face of the post, and were connected by the old floorbeam connection holes in the posts. These plates, 2½ ft. wide over the top flange angles, formed an ample splice between the split web and the angles. Where these plates connected to the truss posts, the upper holes were fitted with shoulder bolts, the holes in the plates being $\frac{1}{16}$ in. large, to provide for movement in the upper part of the floorbeam, while the lower half of the connection was made with tight bolts.

When these floorbeams were removed there was further evidence of the movement that had previously given trouble, in the form of splits in the connection plates at the bolt holes where they connected to the post. In some places there was a double split, allowing a piece of the plate opposite the bolt hole to drop out.

Some time ago we found that one of the steel castings supporting the center truss on the anchorage pier had split, under the center line of the pin to which the anchor rods were attached. The two halves of the casting had moved apart at the bottom about ¼ in., and directly under the pin about $\frac{1}{16}$ in. As it would be very difficult and expensive to renew the casting, as the anchor bars ran through it, it was repaired by drilling holes through the webs and putting in four 1½-in. bolts tying the two halves of the casting together. Frequent inspection was given the casting but there was no further movement apart of the broken halves.

The cause of the break was always a matter of speculation, as there was no sign of defect on the surface. When the casting was removed we found the trouble. The webs, especially the center web, which was 6 in. wide, contained large segregations of very coarse crystals. In the center web a piece 6x3x½ in. dropped out when the two halves came apart in removing the casting.

Corrosion Experiences—It was evident for some time that the material of the center truss (added in 1900) suffered much more from rust than the old (1883) trusses. This was particularly noticeable on the lacing of the tower struts. As early as 1911 there were holes rusted through the lacing bars of the longitudinal struts of the central tower legs. The lacing bars of the members in the outer lines were somewhat pitted, but were not in very bad condition. By 1918 the lacing bars of the middle tower struts had become so bad that several hundred of them had to be renewed, while only three or four of the bars of the outer struts were renewed and even these were not in as bad condition as the former. I think that the lacing bars of the outer struts were iron, and of the center struts medium steel.

The center truss was in somewhat worse position for catching brine drippings from refrigerator cars. The outer trusses also suffered considerably from this action, but relative to their length of service they showed much better resistance to deterioration than the center truss.

The outer or old trusses were built partly of high-carbon steel (average tensile strength 83,000 lb. and elastic limit 54,000 lb. per sq.in.), and partly of wrought iron. The high-carbon steel was used for tower posts, end posts, posts over towers, and bottom chords of cantilever arms; other material was wrought iron. The later middle truss was also built of two grades of material. The tower posts, end posts, posts over towers, and bottom chord were of 66,000 to 74,000-lb. steel, while other parts of the trusses and bracing (except lateral and sway rods, which were iron) were made of 57,000 to 66,000-lb. steel.

Paint maintenance had at all times been quite thorough. Since 1900, at least, the bridge received as much attention in the way of painting as would be required by the most stringent maintenance specifications. Except within the last couple of years, since work began on the new bridge, the cantilever bridge had been patched up almost yearly, and repainted completely at short intervals. The main parts of the material were in very good condition. The rapid destruction of the lacing of the tower struts was probably due largely to the force with which the brine drops would strike them, because of the height of fall. In respect to this action, the struts of the old and new tower bents were, of course, on equal footing.

Other Notes on Service—Service experience with this bridge leads the writer to advise against the combination of compression and tension members as used in the top chord of the shore arms of the old trusses. The compression and tension elements of these members did not work together, and it is probably not practicable to make them work together in any case. Further, in eyebar tension members having many bars packed on a pin, arrangement should be made to separate the bars either far enough for cleaning and painting or else far enough for oil to penetrate between them. A housing should be put over the bars at the pin joints to keep dirt from accumulating between the bars, so arranged that oil-soaked waste could be put within the housing over the bars.

Where floorbeams are riveted to the vertical posts of the trusses, it is desirable to have positive means to assure the free movement of the floorbeams with the trusses under load. This could perhaps be done most easily by putting rollers under one end of the stringers in each panel. The connections of the lateral bracing should be such that the laterals would not interfere in any way with the free movement of the floorbeams, and at the same time to transfer the lateral stresses directly to the chords.

Hip joints and end joints of anchor arms in a cantilever such as this one should be riveted joints, as there is considerable motion and resulting wear at these points.

Approach Spans—While the work of removing the cantilever span was going on, preparations for removing the two street spans in the approach on the American side and one on the Canadian side were made. One of the American bridges was a double-track half-through-girder span 66 ft. long, with two main girders into which were riveted deep transverse beams carrying a 9-in. concrete slab. The other two bridges were short spans with I-beam and plate ballasted floors. A very good quality of concrete was found in the slab, and removing it proved difficult, especially as the span was over a public street with car track. Extreme care was taken and the concrete was drilled and cut out in small sections. The contractor's foreman rigged up a very efficient tool for drilling and cutting the concrete, by making special drills and chisels to fit the large pneumatic rivet-busters, which saved considerable time.

The steel on which the concrete rested was found in absolutely perfect condition, with its red-lead coating still showing its bright color. The reinforcing

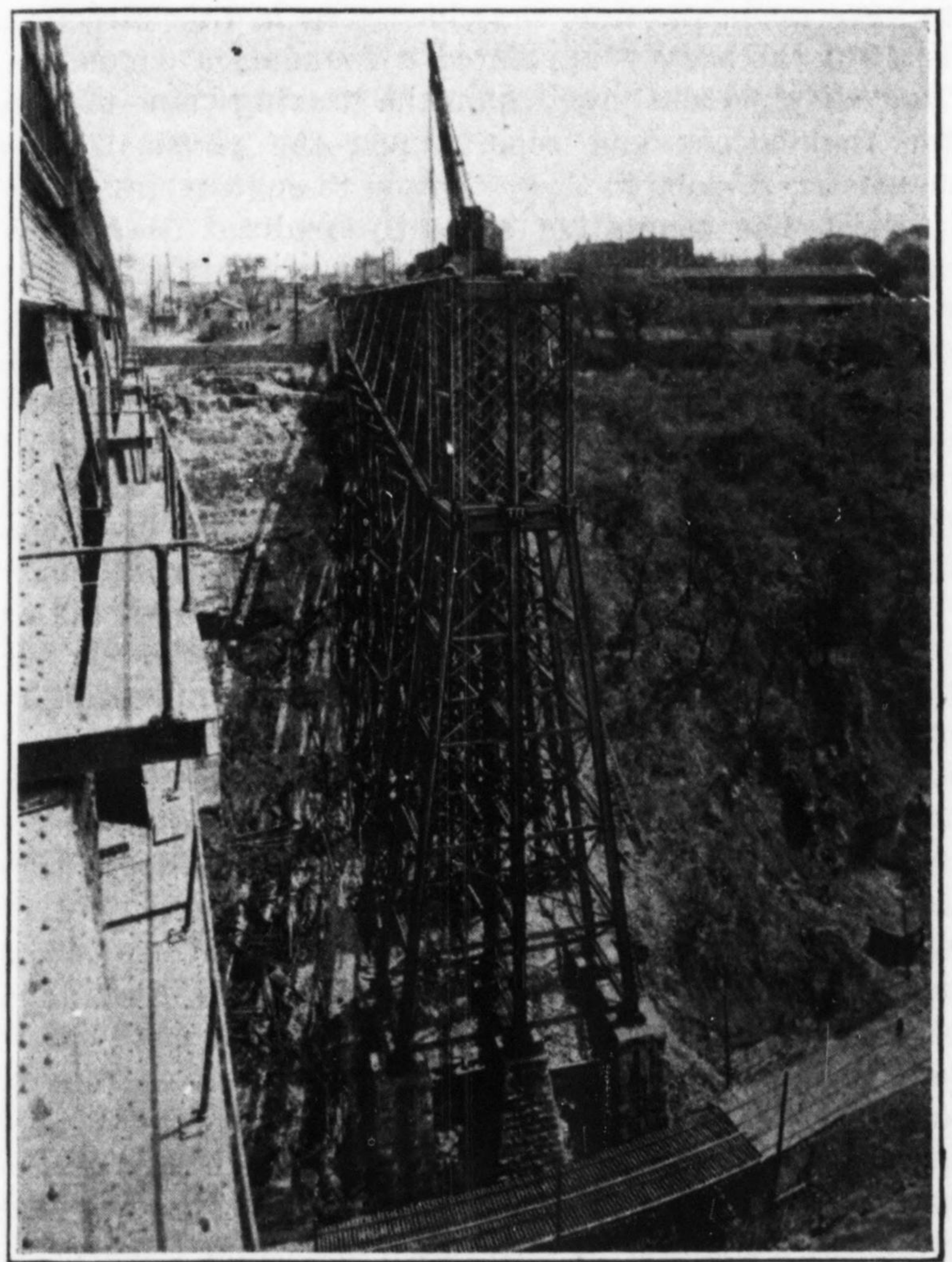


FIG. 5—AMERICAN TOWER AND ANCHOR ARM; RIVER SPAN REMOVED (MAY 26, 1925)

steel in the slab was as good as new, showing no rust. These bridges were all built in 1911, to replace the original iron trestle approaches which had never been strengthened and which were in very bad condition at that time.

The waterproofing on these bridges, which consisted of three layers of raw burlap laid in "Sarco" asphalt and covered with 1½-in. asphalt mastic protection coating, was in remarkably good condition, after fourteen years' service under heavy traffic. The asphalt apparently was as good as when it was put in, and the raw burlap, which is generally said to deteriorate very fast, was as sound as new, as far as I could judge.

The work of removing the cantilever bridge was successfully completed without accidents of any kind and with only few minor injuries to the men; this was due to a large extent to the care and foresight of the American Bridge Co.'s foreman, S. F. Frantz, who had charge of the work. The men who handled the torches for burning off the steel suffered a good deal from the fumes caused by the burning of the old paint of the members, and a couple of them had to go to the hospital to recuperate.

At the present time the anchor pier on the Canadian side and the old concrete abutment for the Front St. crossing in Canada are being removed by the Federal Construction Co., Toronto, Ont.

James H. Curtin has had charge of the field work under my supervision.

Model Tests of Wind Pressure on High Buildings

As A result of many inquiries about wind pressure on buildings the U. S. Bureau of Standards recently undertook some experiments to determine the amount and distribution of such pressure. There appeared to be a general feeling among engineers that the values of wind pressure commonly used in the design of engineering structures are too high. The experiments on which these values are based were carried out many years ago, by relatively inaccurate methods, and on models not resembling actual structures. The success of modern wind tunnel methods in the field of aeronautics suggested their use for the determination of wind pressures on models of structures. Experiments were therefore made on a model in the 10-ft. wind tunnel of the Bureau.

The first model investigated was a model of a tall building, in the form of an 8 x 8-in. square prism 24 in. high. The pressure due to winds of speeds up to 70 m.p.h. was measured at 70 stations on one face and 49 stations on the top, for wind directions varying from 90 deg. to one face to 90 deg. to the opposite face by 15-deg. steps. The effect of the ground was represented in two ways, first, by the floor of the tunnel, and second, by a fixed platform. The pressures were integrated to give total forces and moments on the separate faces and on the complete model.

The chief conclusions of interest to structural engineers are as follows:

(1) The greatest average pressure against the building occurs when the wind blows normal to a face and is equal to 1.5 times the velocity pressure, i.e., 22 lb. per sq.ft. for a true speed of 76 miles per hour (100 miles per hour indicated by a Robinson type anemometer).

(2) The average decrease in pressure over the roof is about 0.83 times the velocity pressure, i.e., 12.2 lb. per sq.ft. for a true speed of 76 miles per hour.

(3) The maximum moment about a vertical axis is equivalent to a horizontal displacement of the resultant force of 0.077 times the width of the building.

While the above conclusions apply exactly only to geometrically similar structures, they apply approximately to structures of the same general shape.

A full report on the tests will be published shortly as a paper in the scientific series of the Bureau. It will include a general survey of the effects of steady wind and some discussion of allowances for gustiness.

Known Sources of Typhoid Only 60 Per Cent

It is a wise and fortunate epidemiologist who can determine satisfactorily the source of infection of more than 60 per cent of the cases of typhoid fever occurring in a given city, declared Dr. Louis I. Harris, director Bureau of Preventable Diseases, New York City, in an address on the control of communicable diseases before the New Jersey Sanitary Association on Nov. 30.