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OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 836.

THE NIAGARA RAILWAY ARCH.

By R. S. BUCK, M. Am. Soc. C. E.

PRESENTED MAY 18TH, 1898.

WITH DISCUSSION.

Historical.—Probably there is no bridge site on the Western Continent of greater technical as well as historic interest than that of the Niagara Railway Arch. Each of the former bridges at this site possessed in its day new and striking features, and marked a distinct advance in American engineering.

The plan of spanning the Niagara gorge with a suspension bridge probably first took practical shape when it was suggested to the Hon. William Hamilton Merritt, of St. Catharines, Ontario, by a description of the Freiburg Suspension Bridge in a letter from a friend. This was in 1844. In 1846, through Mr. Merritt's efforts, charters were obtained from the State of New York and the Canadian Government for the construction of the first bridge across the gorge. The scope of the bridge to be built was not then definitely determined, but the charters show an appreciation of the probable development of railroad facilities and the demand for a railroad bridge at this point. At that time there was no railroad to Niagara Falls from the West, al-

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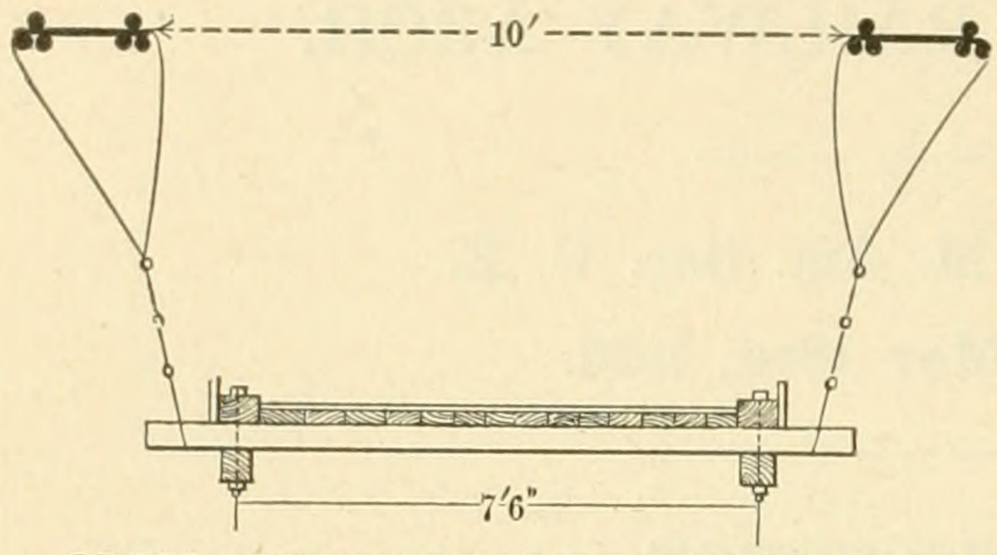
though the Great Western, afterward a lessee of the bridge, was in course of construction.

First Suspension Bridge.—In the winter of 1847 the bridge companies made a contract with Charles Ellet to construct a bridge on the site occupied by the present bridge. It was their ultimate purpose to build a railway bridge, but the plan was delayed for some years by the magnitude of the undertaking and lack of funds. Mr. Ellet first threw across the gorge a cable of thirty-six No. 9 wires, on which a light iron carriage was run for about a year and used for the purposes of the subsequent work and for passenger service. From this was developed the earliest bridge (shown in Plate III), which was completed in 1848. This bridge had no stiffening truss. Its towers were of wood, and the expansion rollers consisted of a single wooden cylinder under each group of cables, that is, two cylinders on each tower. A cross-section of this bridge is shown in Fig. 1.



CROSS SECTION OF BRIDGE AT CENTER

FIG. 1.

Mr. Ellet's connection with the work ceased on the completion of this bridge, and he had no hand in planning the railway bridge as finally built in 1853–1855.

Railway Suspension Bridge.—The conception, development and execution of this bridge were the work of the late John A. Roebling, M. Am. Soc. C. E. Both as an engineering feat and as an historical event, Mr. Roebling's great work is of enduring interest. It is fair to say that in the Niagara Railway Suspension Bridge the results of theoretical research were more successfully applied to practical conditions, so far as the strength of materials is concerned, than in any other bridge built, up to that time. A view of this bridge is shown in Plate IV.

Prior to this, Mr. Roebling had built six suspension bridges, but these were for light highway traffic and did not demonstrate his abilities to the extent shown by this work.

The idea of a suspension bridge for railway service met with strong opposition, some of it from high sources. Its opponents insisted that a suspension bridge under the weight of a railway train must neces-

PLATE III.
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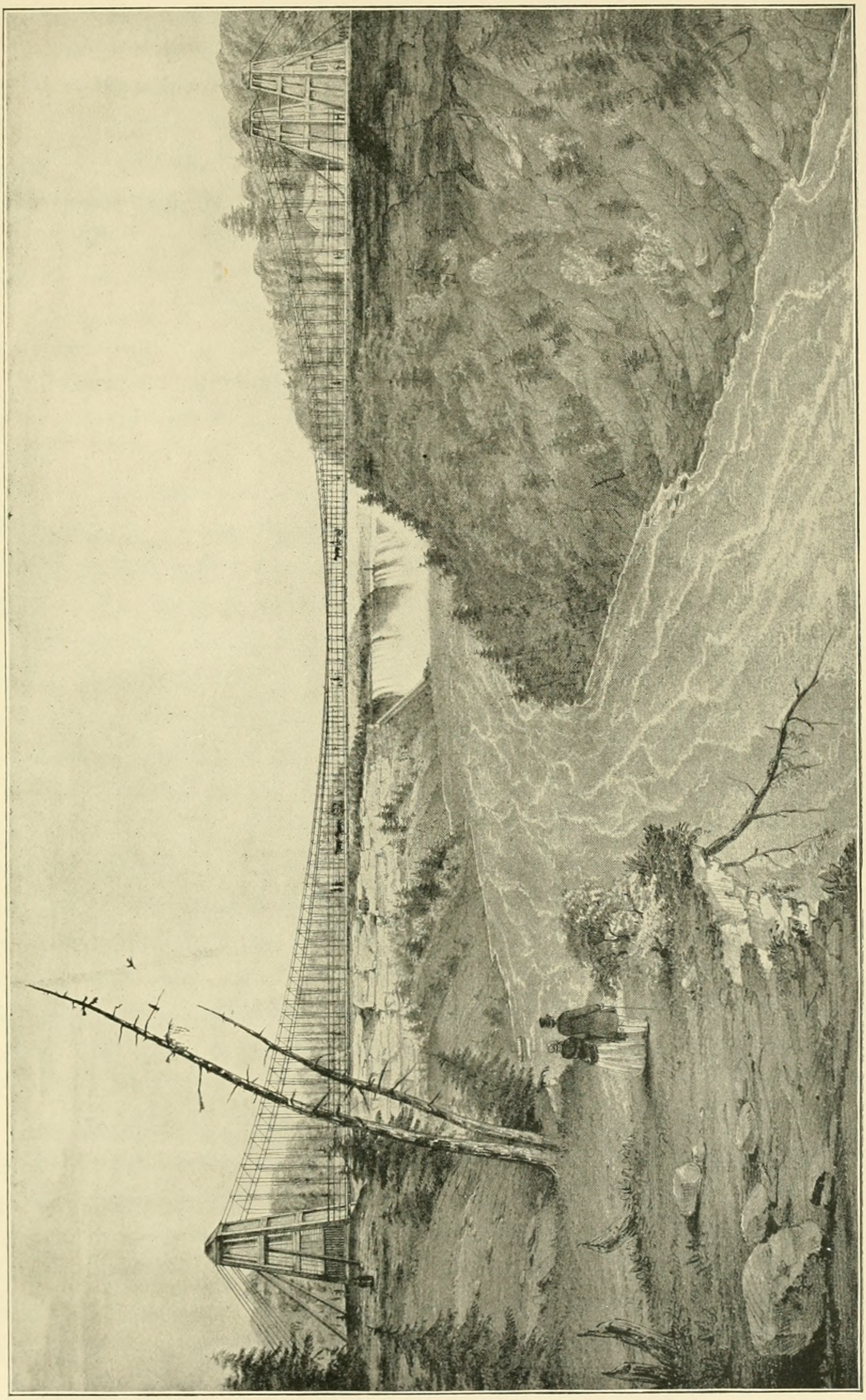
BUCK ON NIAGARA RAILWAY ARCH.



sarily be subjected to excessive and dangerous deflection. Mr. Stevenson was at this time evolving the plans for the Victoria Tubular Bridge at Montreal, and opinion was divided as to the comparative merits of the two types. It was no small part of Mr. Roebling's task to overcome the prejudice against his chosen type of bridge. Even after its completion and successful operation for several years, it was still the object of much criticism, most of which was biased and absurd. It is a strange coincidence that these two bridges, the Niagara Railway Suspension Bridge and the Victoria Tubular Bridge, built at about the same time and for the same object, but so totally different in principle, should serve for almost the same length of time and pass out of existence together, to give place to more vigorous successors, better capable of meeting the ever-growing exactions of trade and travel.

It was Mr. Roebling's firm conviction that no other type of bridge was adaptable to the Niagara gorge, and that the suspension bridge was the coming type for long-span bridges. In the first view he was mistaken, for of the three bridges now spanning the gorge, only one is a suspension bridge, and that is being replaced. In the second, he was likewise wrong, and yet in a measure right. The Niagara Bridge was the only railway suspension bridge ever built. Only one other was commenced, and that was not completed. Still, this type has been accepted by high authority as available for spans of a length beyond the reach of other types, for railway as well as highway service. Even viewed in the light of increased experience, and among the vastly multiplied works of the engineer, despite flaws developed by long service, and reconstruction rendered necessary by time and abuse, the Niagara Railway Suspension Bridge will always be considered a monument to engineering skill. It was a great leap toward the high plane occupied by bridge construction at the present day.

Reconstruction of Railway Suspension Bridge.—In 1877, examination disclosed that the outside layers of wires in the cables had corroded at the anchorages. The cables were here embedded in concrete. The strain on them due to moving loads had worked them loose from the concrete, and left a small surrounding space open to the admission of water. This resulted in considerable corrosion, especially underneath the cables, from the face of the masonry back to the shoe.

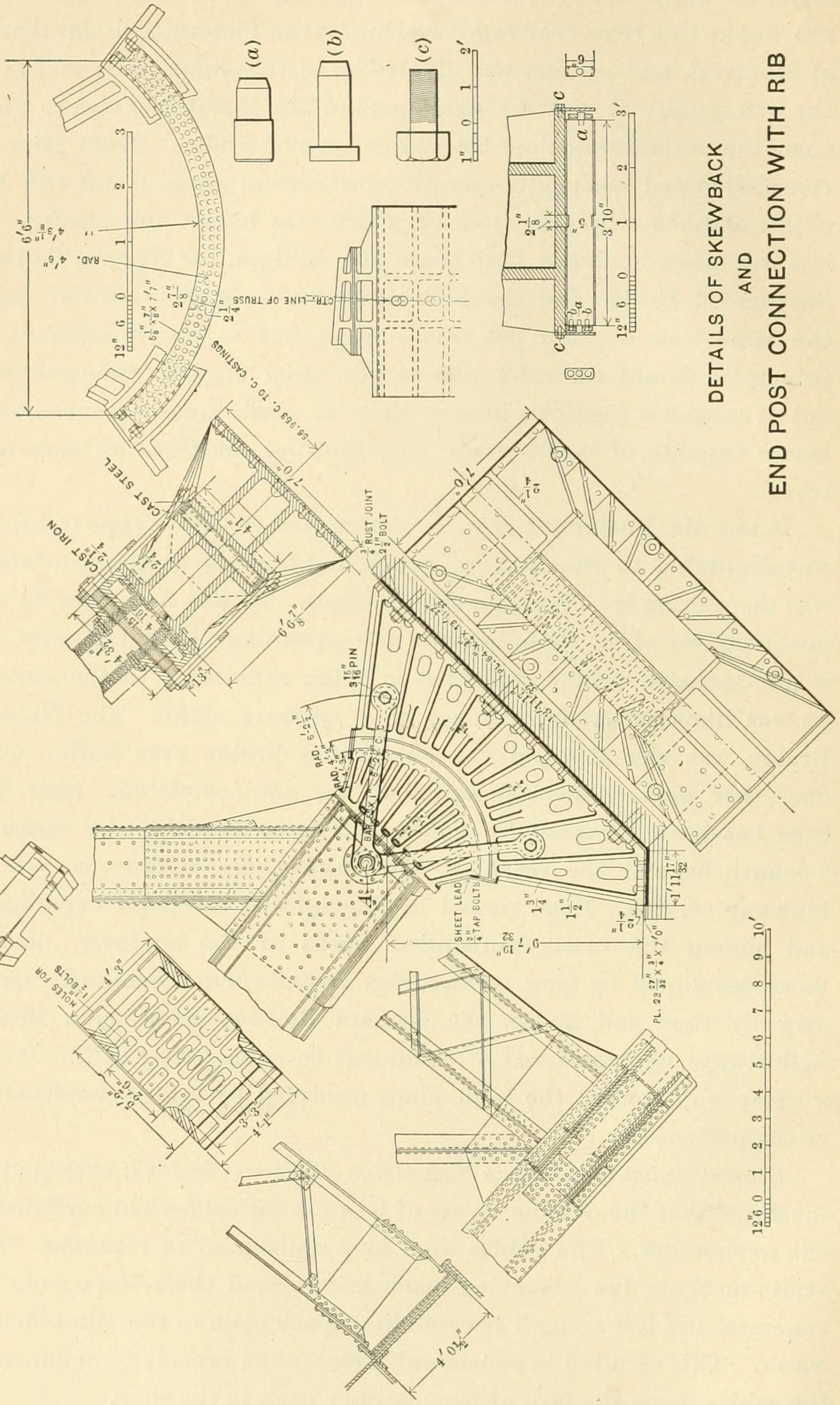

DETAILS OF SKEW BACK
AND
END POST CONNECTION WITH RIB
(a)
(b)
(c)
CTR. LINE OF TRUSS
CAST STEEL
CAST IRON
3" RUST JOINT
1" BOLT
SHEET LEAD
TAP BOLTS
PL. 22 27/32" X 3/4" X 7'0"
HOLES FOR 1" BOLTS
BAR 6" X 1" 6'2" C. C.
RAD. 4'-5"
RAD. 6'-2 1/2"
3 15/16" PIN
66.963 C. TO C. CASTINGS
7'0"
6'6"
4'3 1/8"
RAD. 4'6"
12" 6 0
2 1 0 1 2 3'
1" 0 1 2'
3'10"
2 1 0
12" 6 0
12'6 0 1 2 3 4 5 6 7 8 9 10'

FIG. 2.

PLATE IV.
TRANS. AM. SOC. CIV. ENGRS.
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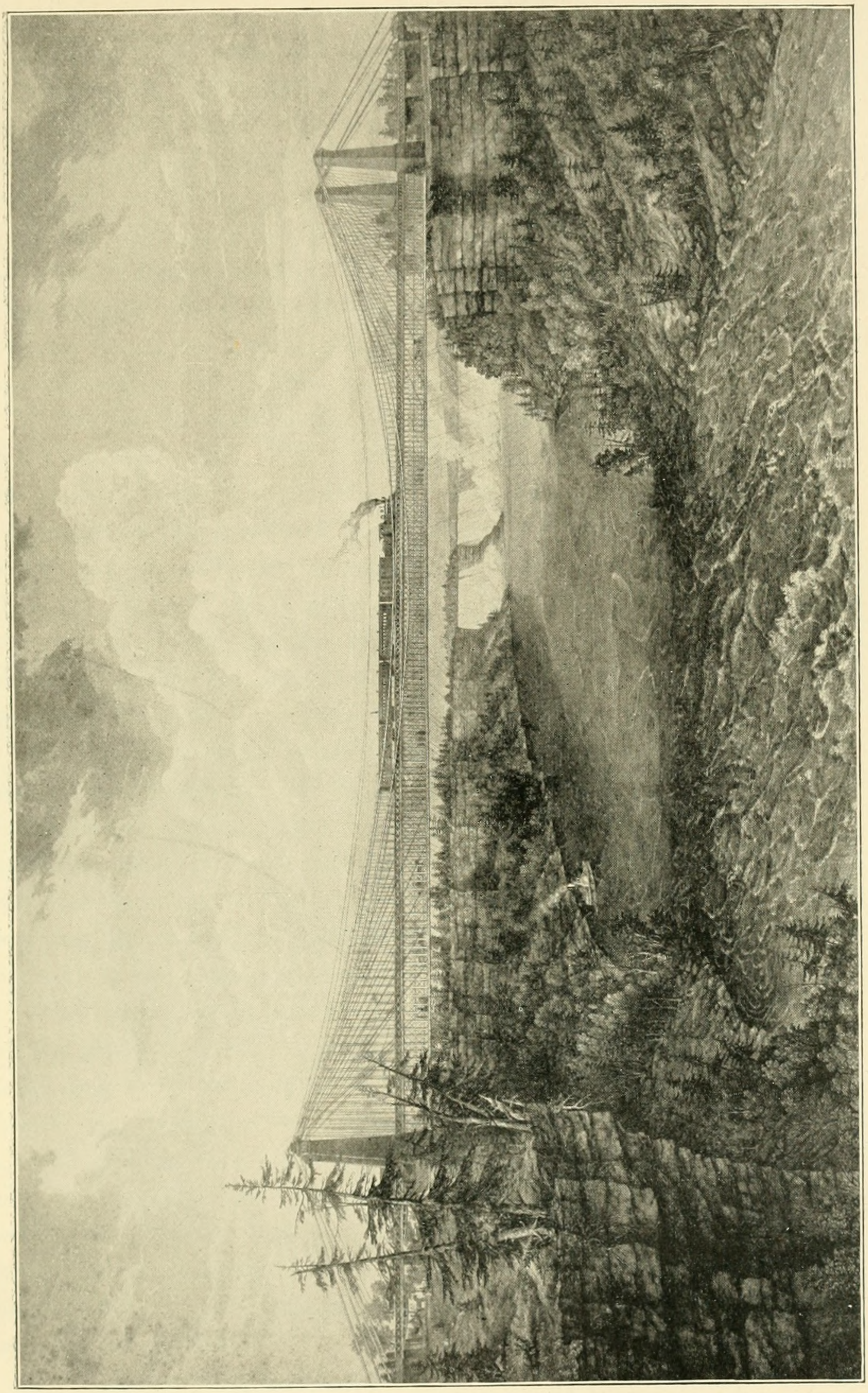
BUCK ON NIAGARA RAILWAY ARCH.



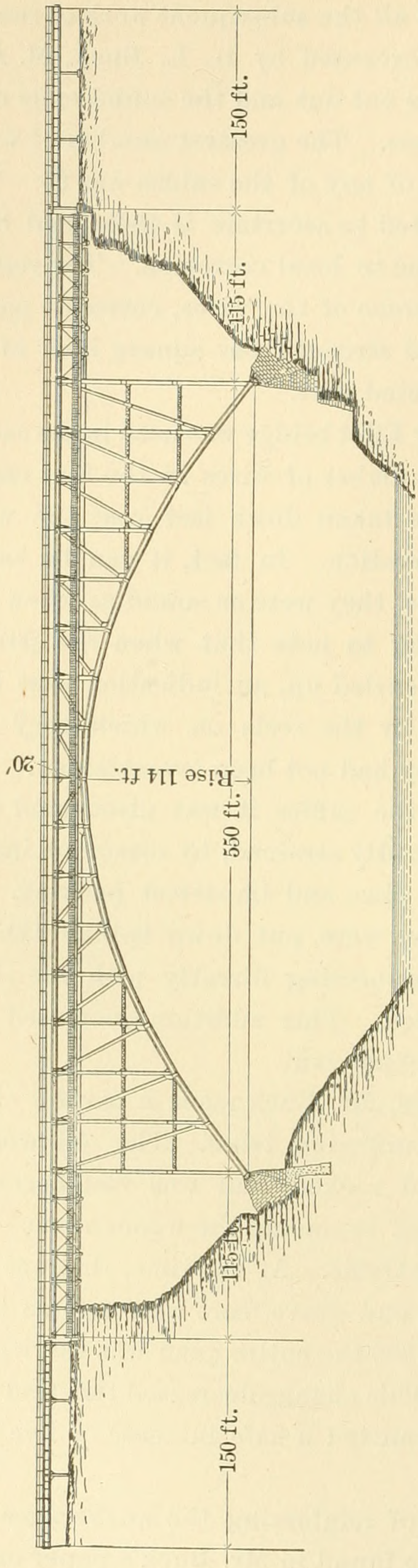
The renovation of the cables, and all the subsequent work of renewing the bridge, was designed and executed by L. L. Buck, M. Am. Soc. C. E. The defective wires were cut out and the sound ends connected and spliced under proper stress. The greatest number of wires that required splicing at one end of any of the cables was 65. The wires removed were thoroughly tested to ascertain if there had been any deterioration other than that due to local corrosion. The results indicated none whatever. In fact, some of the wires, corroded partly through, showed a greater ultimate strength per square inch of remaining section than did the unaffected wire.

The wire in the cables of the early Ellet bridge was used in the cables of the railway bridge. The total number of wires in the four cables was 14 560. When the cables were taken down last year the wires were in an excellent state of preservation. In fact, it can be safely stated that, after 42 years of service, they were as sound as when first placed in position. It is interesting to note that when the strands were cut into short lengths they curled up, an indication that they still retained the set given them by the reels on which they had originally been coiled, and that they had not been overstrained.

During the work of repairing the cables it was discovered that parts of the anchorage had been badly strained, by reason of imperfectly formed eye-bar heads, light pins, and imperfect packing. To reinforce these, two new anchorages were put down behind the old ones, on each side of the river, connecting directly with the shoes carrying the strands of the cables. This addition increased the strength of the anchorages about 50 per cent.

While reinforcing the anchorages, Mr. Buck made a careful study of the problem of renewing the stiffening truss. The old wooden truss was very badly decayed and racked, and was fast becoming ineffective. His plans contemplated replacing the wooden truss with a metal truss without interrupting traffic. At the time, this was considered a very daring undertaking, and grave fears were felt as to its safety and success. However, in 1880 the entire plan was carried out without a single serious mishap. This change decreased the dead load on the cables by 178 tons, and permitted a safe increase of live load of from 200 to 350 tons.

A full description of the work of reinforcing the anchorages and renewing the stiffening truss can be found in Mr. Buck's paper on the subject in the *Transactions* of the Society, June 15th, 1881.


150 ft.
115 ft.
Rise 114 ft.
550 ft.
150 ft.

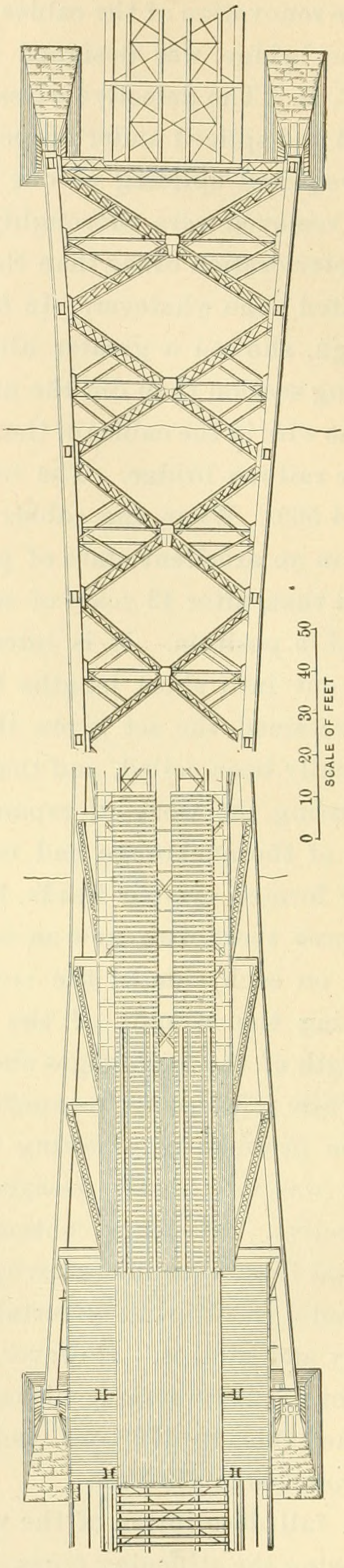
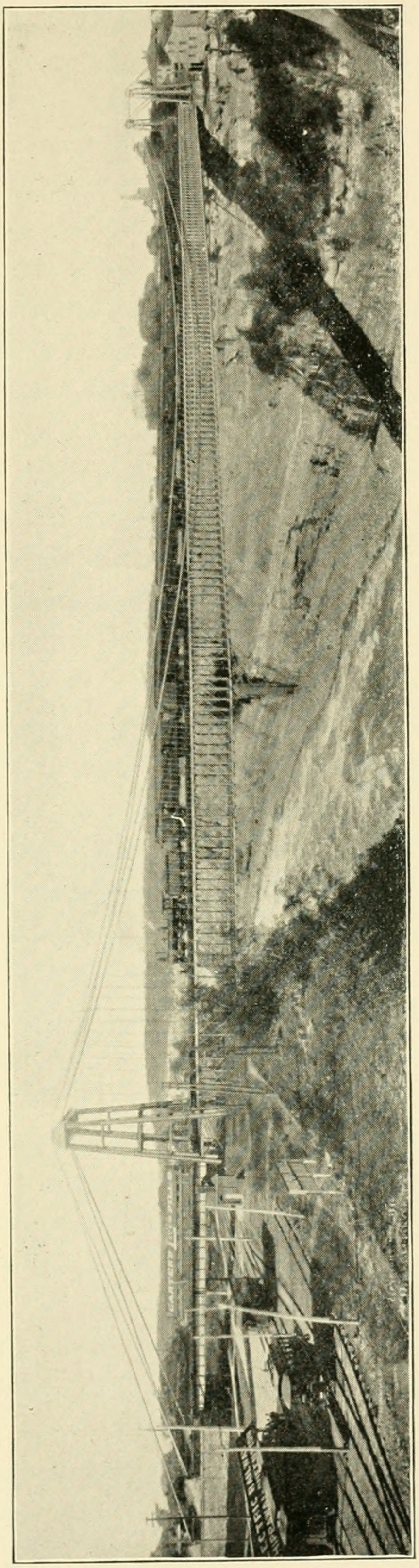

0 10 20 30 40 50
SCALE OF FEET

FIG. 3.



In 1886 it was decided that safety demanded the renewal of the stone towers carrying the cables. These had for some years shown signs of disintegration, but they were kept in fair condition by replacing defective stones from time to time with sound ones. The disintegration was due to the inferior quality of the stone, and was augmented by the failure of the rollers under the cable saddles to perform their function because of rust. In fact, when the roller beds were taken out, the rollers were found to be fixed immovably in a mass of rust and cement, which had worked its way in from the mortar with which the saddles were originally covered by Mr. Roebling. This caused rocking of the towers under live load and changes of temperature, and greatly accelerated the destructive action of the frost on the masonry. It was at first attempted to preserve the towers by cutting away the defective surface stones and casing them in sound masonry, but it soon developed that the disintegration had penetrated too deeply to be remedied by this means. It was then decided to replace them with towers of iron.

Briefly described, this was accomplished as follows:

The corners of the stone towers were cut away, to admit the piers and legs of the new towers, which were then placed in position and temporarily secured to the former with clamps. The saddles carrying the cables on one tower were then lashed securely to a lifting frame, consisting of bent eye-bars and built beams, and the two cables were raised together by means of six 125-ton hydraulic jacks, resting on the new tower leg. When lifted high enough, the weight of the cables was taken on four short cast columns, one at the top of each tower leg. The old saddle bearings and three courses of masonry were then removed, and the heavy built base to take the bed plates under the saddles was moved into place. The new rollers and bed plates were then placed under the saddles and the weight taken on them. While the cables were being lifted, a period of about eight hours, no trains were allowed on the bridge. This completed the work of reconstruction of the suspension bridge. There then remained nothing of the original structure, except the cables, saddles, suspenders and anchorages. The reconstructed bridge is shown in Plate V.

It was thought, when it was decided to replace the suspension bridge, that the old bridge could be utilized at another site, but when

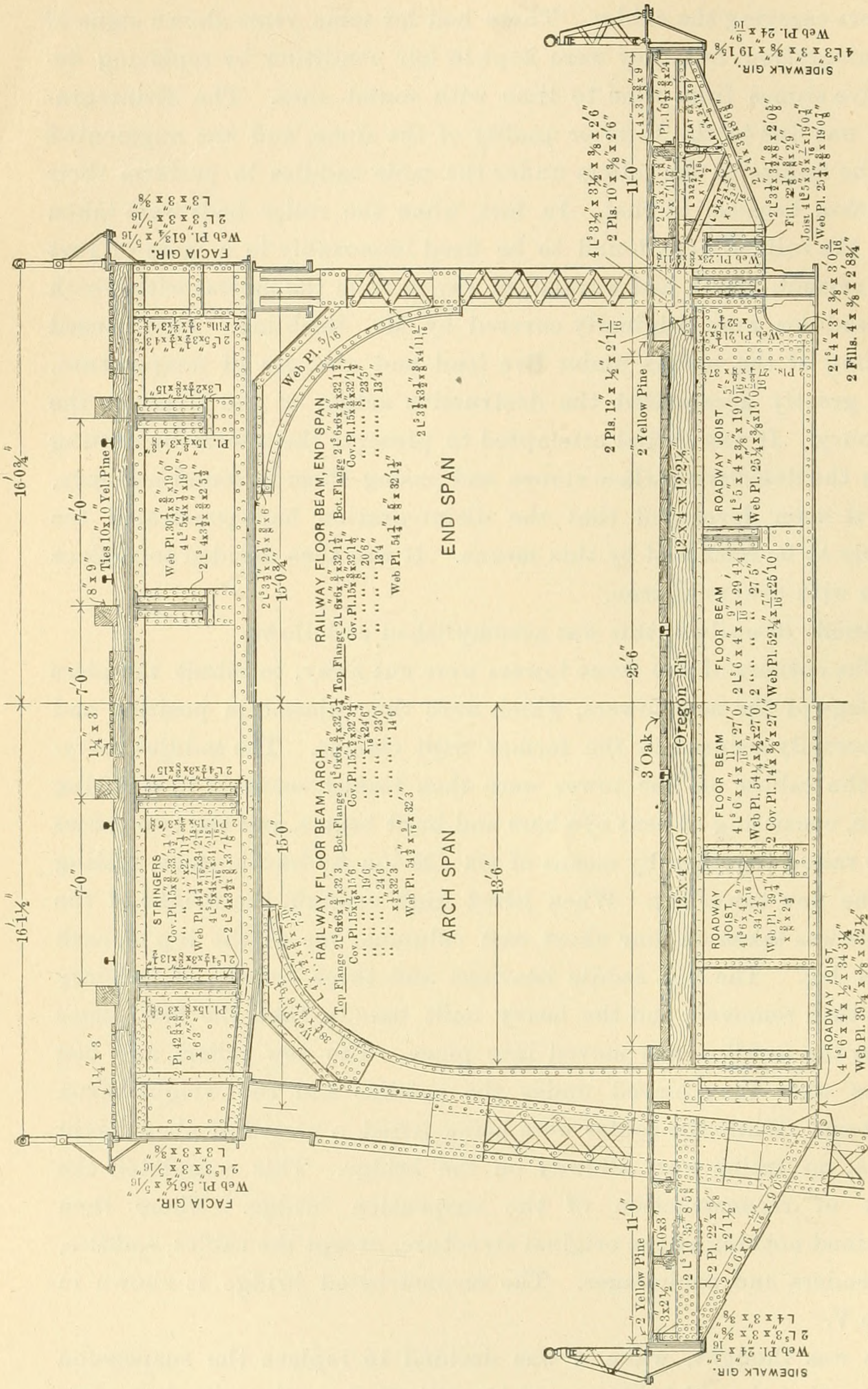

END SPAN
ARCH SPAN
RAILWAY FLOOR BEAM, END SPAN
RAILWAY FLOOR BEAM, ARCH
STRINGERS
ROADWAY JOIST
FLOOR BEAM
FACIA GIR.
SIDEWALK GIR.
Oregon Fir
Yellow Pine
3 Oak
16′-0¾″
16′-1½″
15′-0″
13′-6″
12′-4″ x 12′-2½″
12′-4″ x 10′
25′-6″
11′-0″
7′-0″
8″ x 9″
Ties 10x10″ Yel. Pine
1¼″ x 3″
Web Pl. 61¾″ x 5/16″
2 L 5″ 3 x 3″ x 5/16″
L 3″ x 3″ x ⅜″
Web Pl. 56½″ x 5/16″
Web Pl. 24″ x 9/16″
4 L 5″ 3 x 3″ x ⅜″ x 19′ 1½″
2 Pls. 12″ x ½″ x 2′ 1 1/16″
2 Pls. 10″ x ⅜″ x 2′ 6″
4 L 5″ 3½″ x 3½″ x ⅜″ x 2′ 0⅝″
Fill. 2 2 L 5″ 3½″ x 3″ x ⅜″ x 2′ 0⅝″
Joist 4 L 5″ 3″ x 3″ x 5/16″ x 19′ 0¼″
Web Pl. 25¼″ x ⅜″
2 Fills. 4″ x 3″ x ⅜″ x 2′ 8¾″
2 L 5″ 4 x 3″ x ⅜″ x 19′ 0¼″
Web Pl. 25¼″ x ⅜″
2 Fills. 4″ x 3″ x ⅜″ x 2′ 0″
4 L 5″ 6″ x 4″ x ½″ x 34′ 3¼″
Web Pl. 39¼″ x ⅜″ x 2′ 6¼″
2 L 5″ 4 x 3″ x ⅜″ x 2′ 6¼″
2 Fills. 4 x ⅜″ x 2′ 0″
2 Pl. 22″ x ⅝″ x 2′ 1½″
2 L 5″ 10″ 35# 8 5/8″
2 L 5″ 6″ x 4″ x 11/16″ x 27′ 0″
Web Pl. 54¼″ x ½″ x 27′ 0″
2 Cov. Pl. 14″ x ⅜″ x 27′ 0″
Web Pl. 54½″ x ⅝″
2 Cov. Pl. 15″ x ⅝″ x 32′ 1½″
Bot. Flange 2 L 5″ 6″ x 6″ x ⅝″ x 32′ 1½″
Top Flange 2 L 5″ 6″ x 6″ x ⅝″ x 32′ 3″
Web Pl. 44″ x 1/16″ x 34′ 2 15/16″
4 L 5″ 6″ x 4″ x 11/16″ x 31′ 2 15/16″
2 L 5″ 4 x 3½″ x ⅝″ x 37′ 1½″
Web Pl. 30¼″ x ⅝″ x 19′ 0″
4 L 5″ 5″ x 4″ x ½″ x 19′ 0″
2 L 5″ 4 x 3½″ x ⅝″ x 25′ 5½″
Web Pl. 15″ x 7/16″ x 19′ 0″
2 L 5″ 4 x 3″ x ⅜″ x 19′ 0″
2 Pl. 42″ x ⅝″ x 6′ 3″
Pl. 15″ x ⅝″ x 33 4″
2 L 5″ 4 x 3 x ⅜″ x 13″
2 L 5″ 5 x 3½″ x 1/16″ x 13″
2 Fills. 3½″ x ⅜″ x 3 4″
2 L 5″ 3 x 3″ x 1/16″ x 15″
Pl. 15″ x 7/16″ x 33 5½″
Cov. Pl. 15″ x ⅝″ x 33 5½″
2 L 5″ 4 x 3″ x 1/16″ x 13″
Pl. 15″ x 7/16″ x 33 4″
Web Pl. 44″ x 1/16″ x 34′ 2 15/16″
4 L 5″ 6″ x 4″ x 11/16″ x 31′ 2 15/16″
2 L 5″ 4 x 3½″ x ⅝″ x 37′ 1½″
2 L 5″ 6″ x 6″ x ⅝″ x 32′ 3″
Cov. Pl. 15″ x 7/16″ x 15′ 6″
Pl. 1′ 6 1/16″ x ⅝″ x 24′
Pl. 1′ 6 1/16″ x ⅝″ x 24′
2 L 5″ 3½″ x 3½″ x ⅝″ x 4′ 11 2/16″
2 L 5″ 3½″ x 3½″ x ⅝″ x 4′ 11 2/16″
2 L 5″ 3½″ x 2½″ x ⅝″ x 6′
L 4″ x 3″ x 8/16″ x 9
L 4″ x 3″ x 8/16″ x 9
FLAT 6″ x 8/16″ x 9′
3″ x 1/16″
3″ x 1/16″
L 3 x 2½ x 3/16
L 3 x 2½ x 3/16
L 4 x 3 x ⅜″
L 4 x 3 x ⅜″
2 L 5″ 3 x 3″ x 5/16″
2 L 5″ 3 x 3″ x 5/16″
L 3 x 3 x ⅜″
L 3 x 3 x ⅜″
2 Pl. 22″ x ⅝″ x 2′ 1½″
2 L 5″ 10″ 35# 8 5/8″
10″ x 3″
3″ x 2½″
2 Yellow Pine
11′-0″
2 L 5″ 6″ x 4″ x 11/16″ x 27′ 0″
Web Pl. 54¼″ x ½″ x 27′ 0″
2 Cov. Pl. 14″ x ⅜″ x 27′ 0″
Web Pl. 54½″ x ⅝″
2 Cov. Pl. 15″ x ⅝″ x 32′ 1½″
Bot. Flange 2 L 5″ 6″ x 6″ x ⅝″ x 32′ 1½″
Top Flange 2 L 5″ 6″ x 6″ x ⅝″ x 32′ 3″
Web Pl. 5/16″
2 L 5″ 4 x 3″ x ⅜″ x 19′ 0″
Web Pl. 25¼″ x ⅜″
2 Fills. 4″ x 3″ x ⅜″ x 2′ 8¾″
2 L 5″ 4 x 3″ x ⅜″ x 19′ 0¼″
Web Pl. 24″ x 9/16″
2 L 5″ 3 x 3″ x 5/16″
L 4 x 3 x ⅜″
2 Yellow Pine
11′-0″
2 L 5″ 10″ 35# 8 5/8″
2 Pl. 22″ x ⅝″ x 2′ 1½″
3″ x 2½″
10″ x 3″
2 L 5″ 6″ x 4″ x 11/16″ x 27′ 0″
Web Pl. 54¼″ x ½″ x 27′ 0″
2 Cov. Pl. 14″ x ⅜″ x 27′ 0″
Web Pl. 54½″ x ⅝″
2 Cov. Pl. 15″ x ⅝″ x 32′ 1½″
Bot. Flange 2 L 5″ 6″ x 6″ x ⅝″ x 32′ 1½″
Top Flange 2 L 5″ 6″ x 6″ x ⅝″ x 32′ 3″
Web Pl. 5/16″
2 L 5″ 4 x 3″ x ⅜″ x 19′ 0″
Web Pl. 25¼″ x ⅜″
2 Fills. 4″ x 3″ x ⅜″ x 2′ 8¾″
2 L 5″ 4 x 3″ x ⅜″ x 19′ 0¼″
Web Pl. 24″ x 9/16″
2 L 5″ 3 x 3″ x 5/16″
L 4 x 3 x ⅜″

FIG. 4.

PLATE VI.
TRANS. AM. SOC. CIV. ENGRS.
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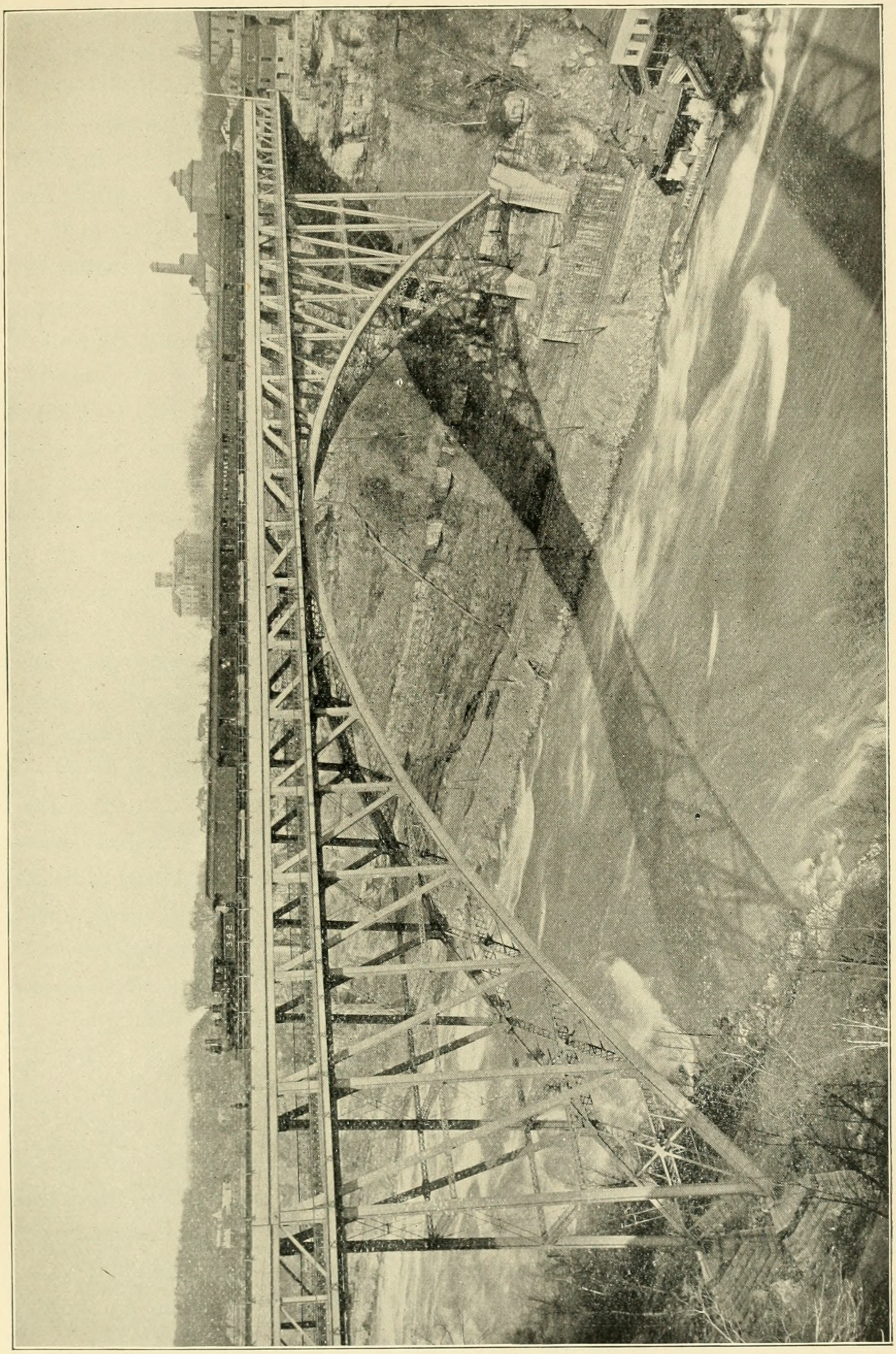
BUCK ON NIAGARA RAILWAY ARCH.



the work was done, no site was available; and, owing to the difficulty and expense of taking it down in proper condition for re-erection, the whole structure, except, perhaps, the suspenders and the wind-guys, was consigned to the scrap heap. Some of this material, after going through the furnace and rolls, will appear again in the Niagara Falls and Clifton Bridge.

Arch.—The completed arch is shown in Plate VI. In treating of the present bridge, a simple recital of the facts, as observed by the author, is all that he can contribute towards a discussion of the principles involved in the evolution of a work with which he was fortunate in being associated, during its design and execution. He feels that much of value can be and should be contributed on the subject of steel arch construction from many well-equipped sources, and therefore hopes that whatever is lacking in the paper will be forthcoming in its discussion.

The steel arch has, within the past few years, grown greatly in interest and importance, and is entitled to full consideration. It is rigid, and, at such a site as the Niagara Gorge, is economical beyond any other type. It also stands far ahead of all others, except, that in point of beauty, perhaps its anti-type, the suspension bridge, takes first place in the minds of some.

There is lacking the simple practical treatment of metal arches which has been given to other types of trusses, a treatment which would supply the wants of the engineer who seeks results and cannot afford to master the numerous partial and abstract treatises in order to reach them.

The fund of information on this subject is not scant, but it needs concentration.

The author has derived much assistance from the work on arches,* by Charles E. Greene, M. Am. Soc. C. E., but this does not supply the entire need.

Division of Types.—Arches are usually divided into three general types: 1st, three-hinged; 2d, two-hinged; 3d, hingeless or elastic. Without changing the form or arrangement, or in fact anything other than inconspicuous details, the same general design can be put in any of the three classes, and in each instance will be subjected to radically different stresses and deflections under load. The problem as to which

* "Trusses and Arches Analyzed and Discussed by Graphical Methods."

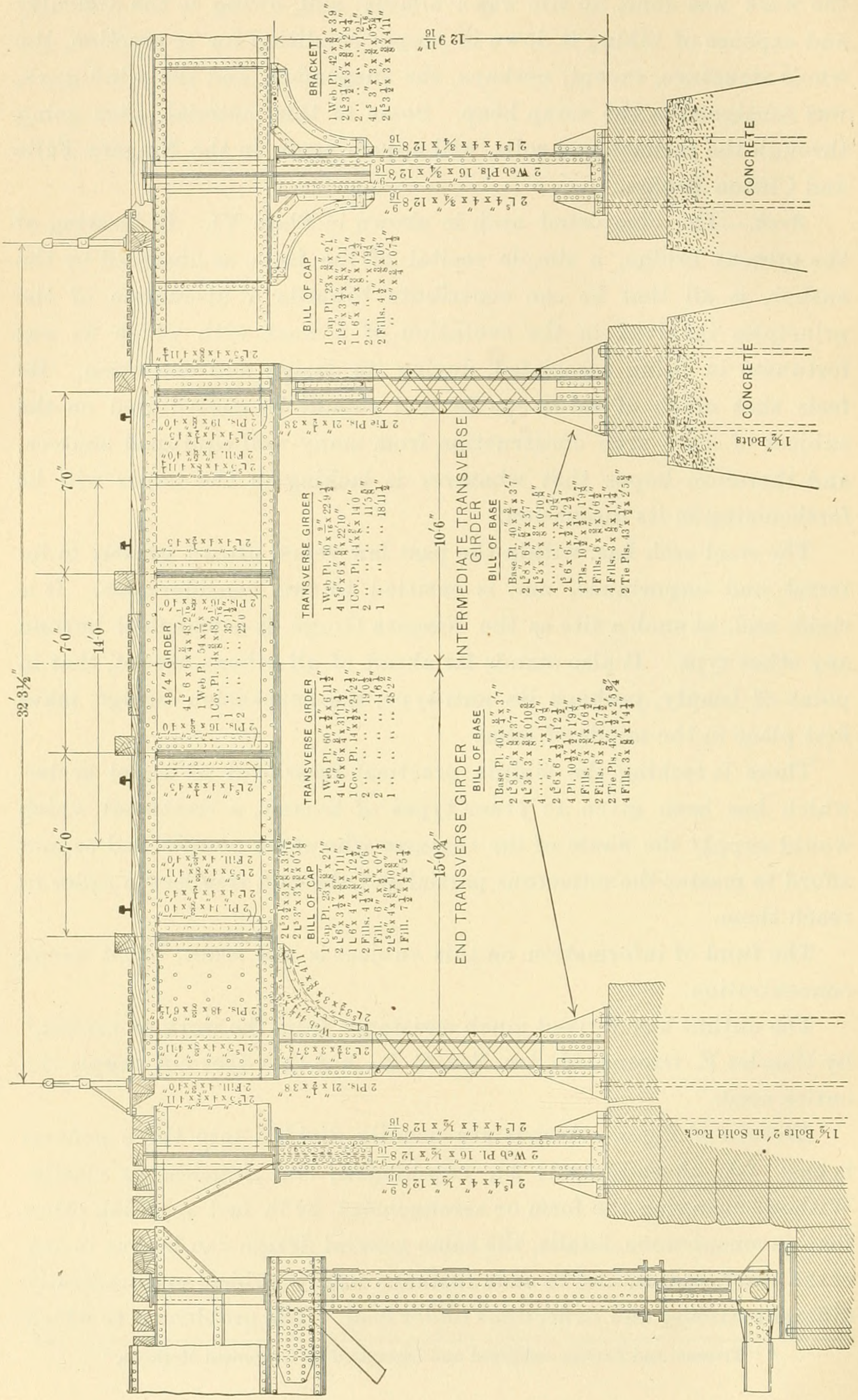

BRACKET
1 Web Pl. 42″ x ⅜″ x 3′9″
2 L⁵ 3½″ x 3″ x ⅜″ x 2′8½″
2 ″ ″ ″ ″ ″ ″ ″ 1′2 1/16″
4 L⁵ 3″ x 3″ x ⅜″ x 0′5 9/16″
2 L⁵ 3½″ x 3″ x ⅜″ x 4′11″
12 9 11/16″
2 L⁵ 4″ x 4″ x ¾″ x 12′8 9/16″
2 Web Pls. 16″ x ¾″ x 12′8 9/16″
2 L⁵ 4″ x 4″ x ¾″ x 12′8 9/16″
CONCRETE
BILL OF CAP
1 Cap Pl. 23″ x ⅜″ x 2′1″
2 L⁵ 6″ x 3½″ x ⅜″ x 1′11″
2 ″ ″ ″ ″ ″ ″ ″ 1′2½″
1 L 6″ x 4″ x ⅜″ x 0′9½″
2 ″ ″ ″ ″ ″ ″ ″ 0′6″
2 Fills. 4½″ x ⅜″ x 0′7½″
2 L⁵ 3½″ x 3″ x ⅜″ x 4′11″
2 L⁵ 3″ x 3″ x ⅜″ x 0′5 9/16″
2 Tie Pls. 21″ x ⅜″ x 3′8″
1¼″ Bolts
2 L⁵ 3″ x 3″ x ⅜″ x 4′11″
2 L⁵ 3½″ x 3″ x ⅜″ x 4′11″
2 Fill. 4″ x ⅜″ x 4′0″
2 L⁵ 3″ x 3″ x ⅜″ x 4′11″
2 L⁵ 4″ x 4″ x ½″ x 15″
2 Pls. 19″ x ⅜″ x 4′0″
2 L⁵ 4″ x 4″ x ½″ x 15″
2 Fill. 4″ x ⅜″ x 4′0″
2 L⁵ 3″ x 3″ x ⅜″ x 4′11″
2 L⁵ 4″ x 4″ x ½″ x 15″
2 Pl. 14″ x ⅜″ x 4′0″
2 L⁵ 4″ x 4″ x ½″ x 15″
2 Fill. 4″ x ⅜″ x 4′0″
2 L⁵ 3″ x 3″ x ⅜″ x 4′11″
2 L⁵ 4″ x 4″ x ½″ x 15″
2 Pls. 48″ x ⅜″ x 6′1″
2 Fill. 4″ x ⅜″ x 4′0″
2 L⁵ 3″ x 3″ x ⅜″ x 4′11″
2 Pls. 21″ x ⅜″ x 3′8″
2 L⁵ 3½″ x 3″ x ⅜″ x 4′11″
2 L⁵ 4″ x 4″ x ½″ x 12′8 9/16″
2 Web Pl. 16″ x ½″ x 12′8 9/16″
2 L⁵ 4″ x 4″ x ½″ x 12′8 9/16″
1¼″ Bolts 2′ in Solid Rock
7′-0″
7′-0″
14′-0″
32′3½″
10′6″
15′-0¾″
INTERMEDIATE TRANSVERSE GIRDER
END TRANSVERSE GIRDER
48″4″ GIRDER
1 Web Pl. 54″ x 7/16″ x 48′2 1/16″
1 Cov. Pl. 14″ x ⅜″ x 35′1½″
2 Pls. 10″ x 7/16″ x 22′0″
TRANSVERSE GIRDER
1 Web Pl. 60″ x 9/16″ x 22′9½″
4 L⁵ 6″ x 6″ x ⅝″ x 22′10″
1 Cov. Pl. 14″ x ⅜″ x 14′0″
2 ″ ″ ″ ″ ″ ″ ″ 11′5 13/16″
1 ″ ″ ″ ″ ″ ″ ″ 18′11½″
BILL OF BASE
1 Base Pl. 40″ x ⅜″ x 3′7″
2 L⁵ 8″ x 6″ x ⅝″ x 3′7″
4 L⁵ 3″ x 3″ x ⅜″ x 0′10⅝″
4 ″ ″ ″ ″ ″ ″ ″ 1′9½″
2 L⁵ 6″ x 6″ x ⅝″ x 1′2½″
4 Pl. 10½″ x ½″ x 1′9¼″
4 Fills. 6″ x ⅜″ x 0′6½″
2 Fills. 6″ x ½″ x 0′7½″
2 Tie Pls. 43″ x ½″ x 2′5⅝″
4 Fills. 3″ x ⅝″ x 1′4¼″

FIG. 5.

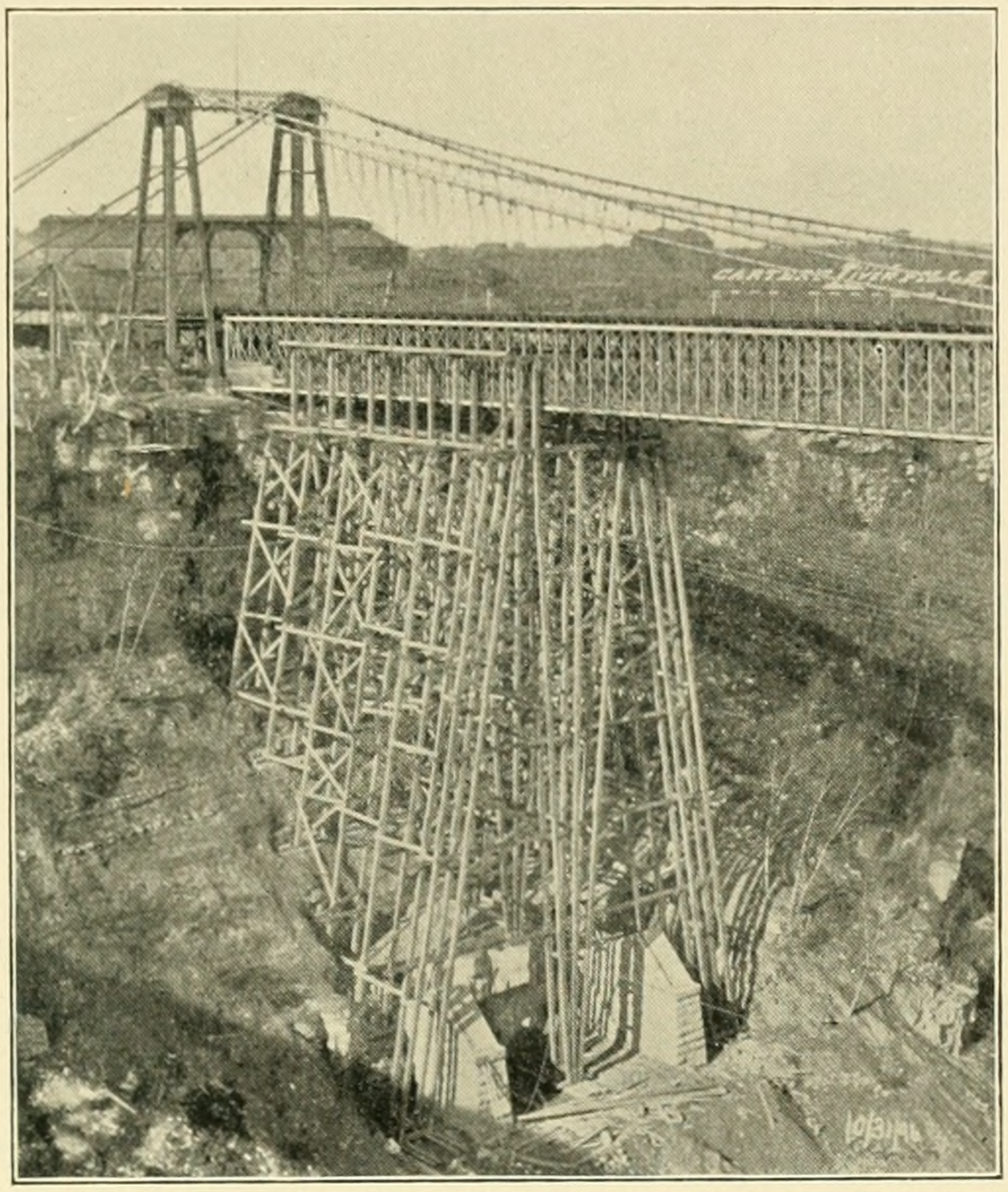


FIG. 1.

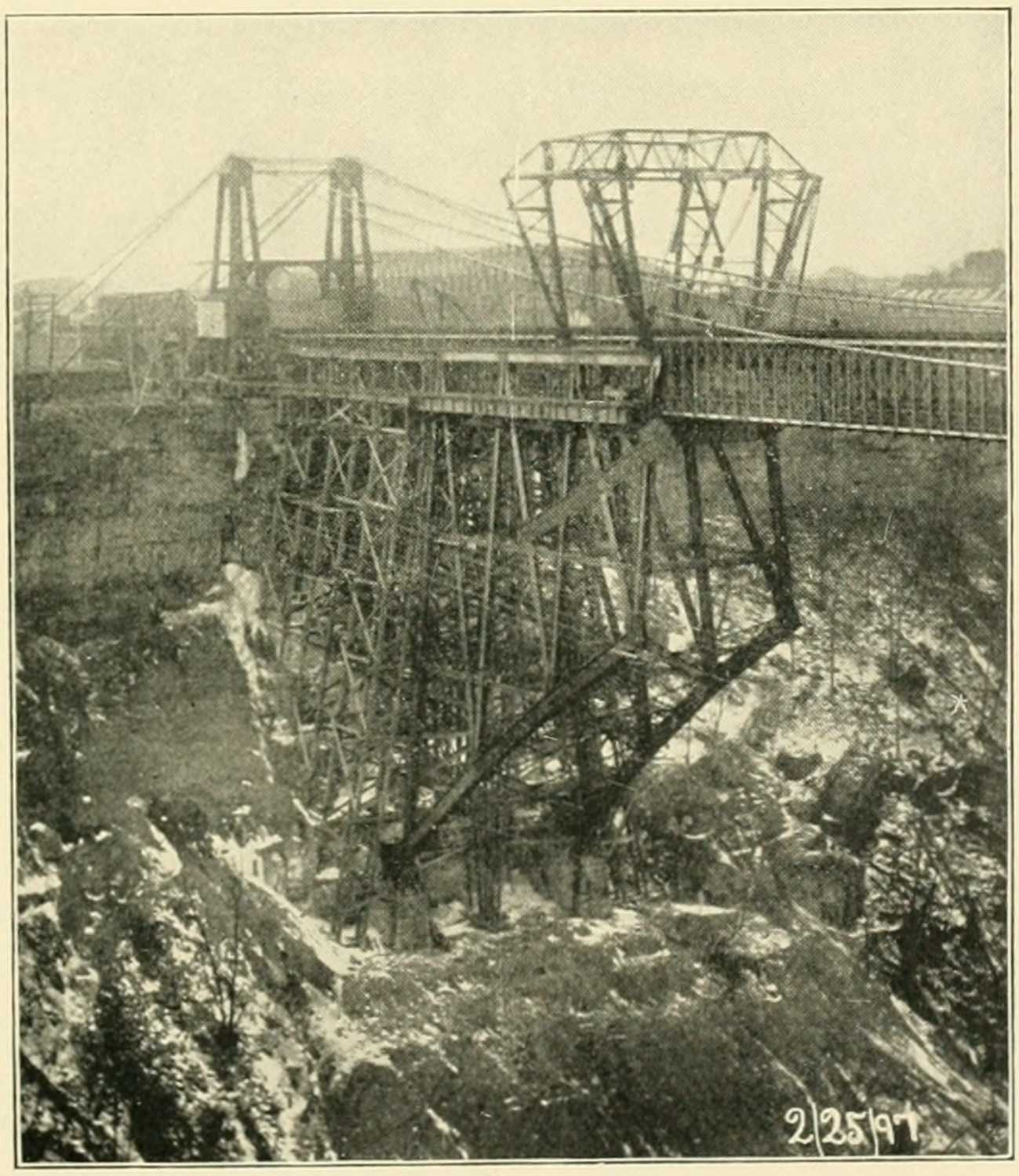


FIG. 2.

of the three types is best suited to given conditions is difficult to solve.

The three-hinged arch has been, and will perhaps continue to be, a popular type, on account of its simplicity of computation and adjustment, and the practical absence of temperature stresses. However, what is gained in these respects is at the cost of rigidity, a matter of smaller importance in roof trusses, but of great importance in bridges, especially in those for heavy service. Every hinge is intended to provide for movement, and facilitates distortion under eccentric loading.

A marked advantage in removing the center hinge is that reversal in the web members is greatly reduced, and the top chord is made to carry a larger proportion of the stresses which are otherwise carried almost entirely by the rib.

Hence the question arises: which is preferable, ease of calculation and adjustment, inconsiderable temperature stresses and greater vibration, or greater rigidity with increased temperature stresses and difficulty of adjustment? On similar grounds, comparison can be made between the two-hinged and the hingeless arches.

Niagara Railway Arch.—In the design of the Niagara Railway Arch, the problems presented by the excessive loading to be provided for, by the length of the span, and in the erection, which had to be accomplished without interruption to traffic, all required careful treatment.

The Chief Engineer, after a thorough investigation of all available types, fixed on the two-hinged spandrel-braced arch as best meeting all requirements. In 1882–83, when the subject of building a bridge for the Michigan Central Railway, across the Niagara Gorge, was under consideration, he prepared a design and estimate for a spandrel-braced arch for that work, to be erected in the same manner as the Niagara Railway Arch. This design included the center hinge. However, no opportunity was given to present it to the Bridge Company. The present cantilever was the design adopted and built. The Driving Park Avenue bridge at Rochester, also designed by him, was a three-hinged spandrel-braced arch. The cantilever method of erection was likewise contemplated in the Rochester design, but as the use of false-work there was not impossible, and the method of erection was optional with the contractors, they adhered to the latter method.

After a careful consideration of the vibrations of the Rochester bridge, under loads most calculated to produce vibrations, Mr. Buck

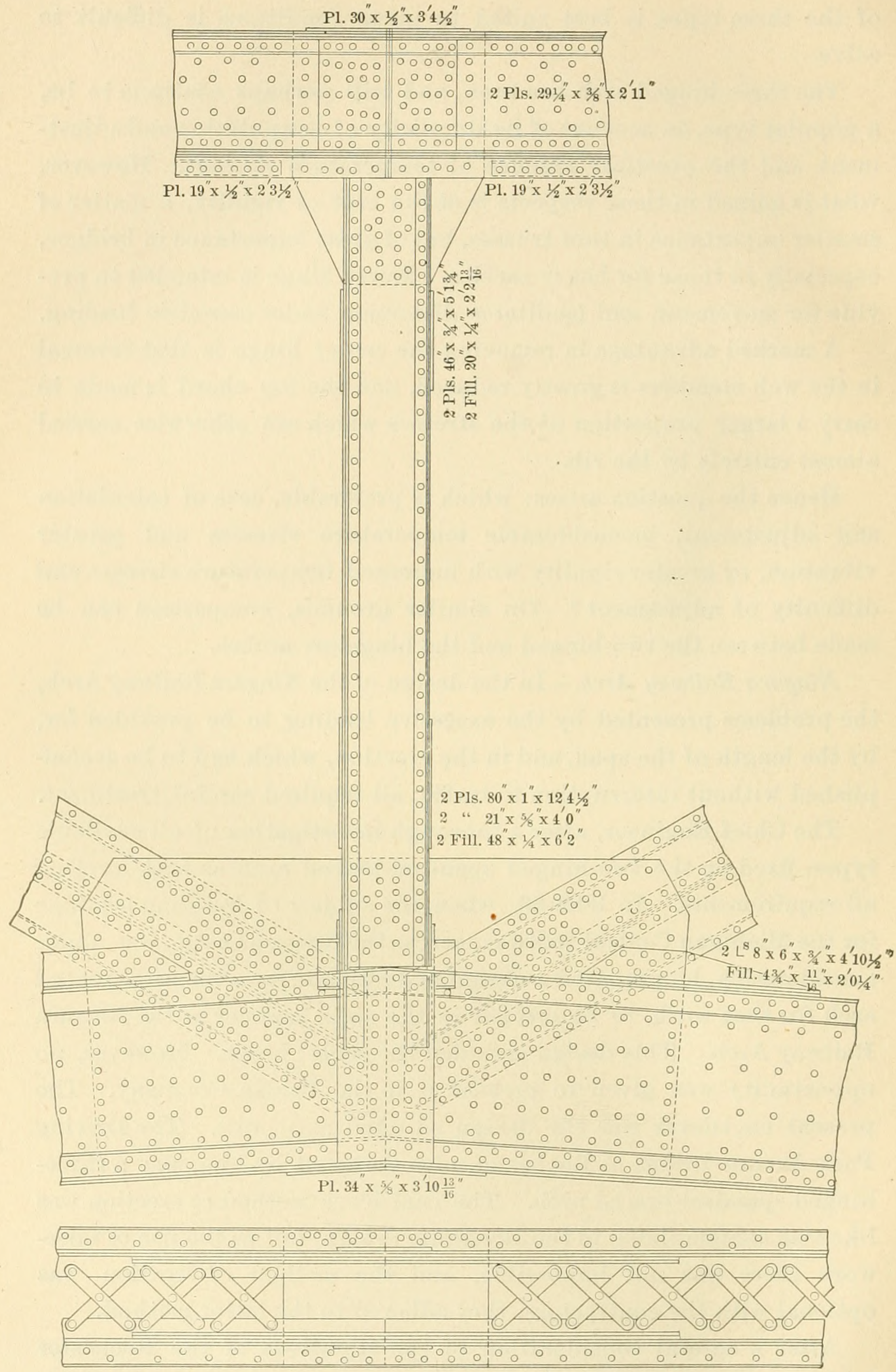
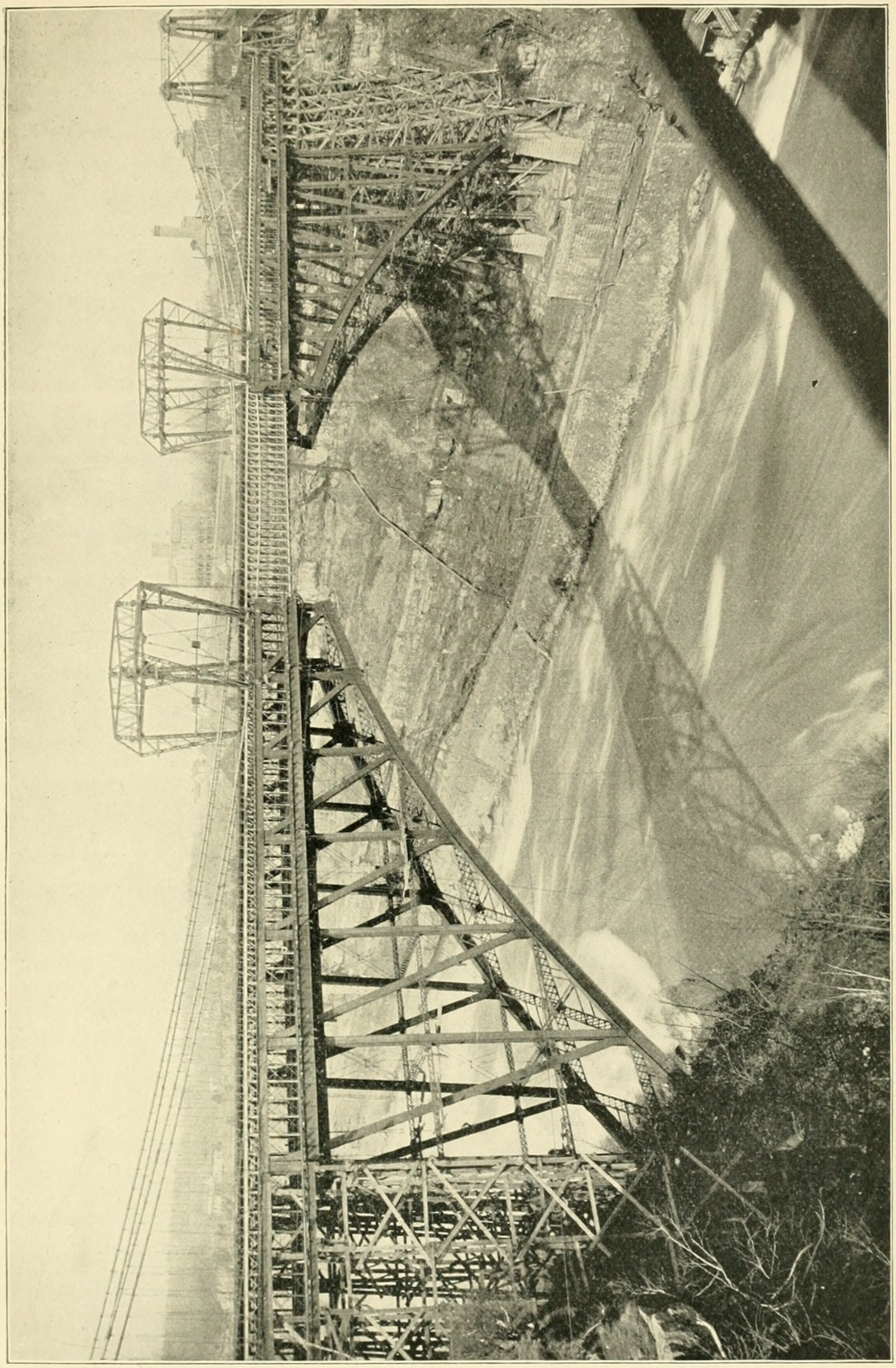

Pl. 30″ x ½″ x 3′ 4½″
2 Pls. 29¼″ x ⅜″ x 2′ 11″
Pl. 19″ x ½″ x 2′ 3½″
Pl. 19″ x ½″ x 2′ 3½″
2 Pls. 46″ x ¾″ x 5′ 1¾″
2 Fill. 20″ x ¼″ x 2′ 2 13/16″
2 Pls. 80″ x 1″ x 12′ 4½″
2 “ 21″ x ⅝″ x 4′ 0″
2 Fill. 48″ x ¼″ x 6′ 2″
2 L 8″ x 6″ x ¾″ x 4′ 10½″
Fill. 4¾″ x 11/16″ x 2′ 0¼″
Pl. 34″ x ⅝″ x 3′ 10 13/16″

FIG. 6.



decided that it would be best to omit the center hinge in the Niagara Arch. Yet his investigations showed that at Rochester, as probably in most cases, the vibrations seemed, to a person standing on the structure, to be much greater than they actually were. The results in this regard in the Niagara Arch are very gratifying. Vibrations due to trains passing over at a rate of 20 miles per hour are scarcely noticeable, while the generally irresistible jog trot of a horse seldom produces the usual responsive swing. The stiffness thus attained, the author believes, has never been found in any other bridge of equal span. The calculated deflection under a moving load of 10 000 lbs. per running foot is $1\frac{15}{16}$ ins., and the observed deflection under the test load, which was about 6 500 lbs. per running foot, was $\frac{13}{16}$ in.

Method of Calculation.—The absence of the center hinge in the spandrel-braced arch renders the calculation of stresses decidedly more difficult than in the three-hinged type. The method of calculation used was that given in Professor Greene's book on Arches, Chapter XII. However, the sections of the rib in the Niagara Railway Arch are increased so as to be a mean between those required by this method and what would be required if there were a third hinge. This was done to meet any inaccuracy of adjustment due to varying temperature.

Foundations.—The skewbacks of the arch span were located with a view to bring the thrust of the arch on the "Clinton Ledge," a solid stratum of gray limestone, from 12 to 14 ft. thick, about half way between the water and the top of the bluff. Above is a blue shale, and below is the beginning of the Medina sandstone formation, thin layers of shale and sandstone sometimes running into solid sandstone 4 to 5 ft. thick. The bearing comes very fairly on the ledge on the New York side, where the stone was cut at the right angle to receive the masonry directly. But on the Canada side, the bearing was not so favorable and concrete had to be used under the front of the south skewback and under the entire north skewback. The heavy face wall under the New York skewbacks was necessitated by the undue encroachment of the Gorge Road upon the site. The cut made for this road left here an almost vertical face, liable to disintegration on exposure, directly at the front of the skewbacks. The skewback masonry is limestone, with granite copings, entirely of dimension stones with $\frac{1}{2}$-in. joints and strong bond.

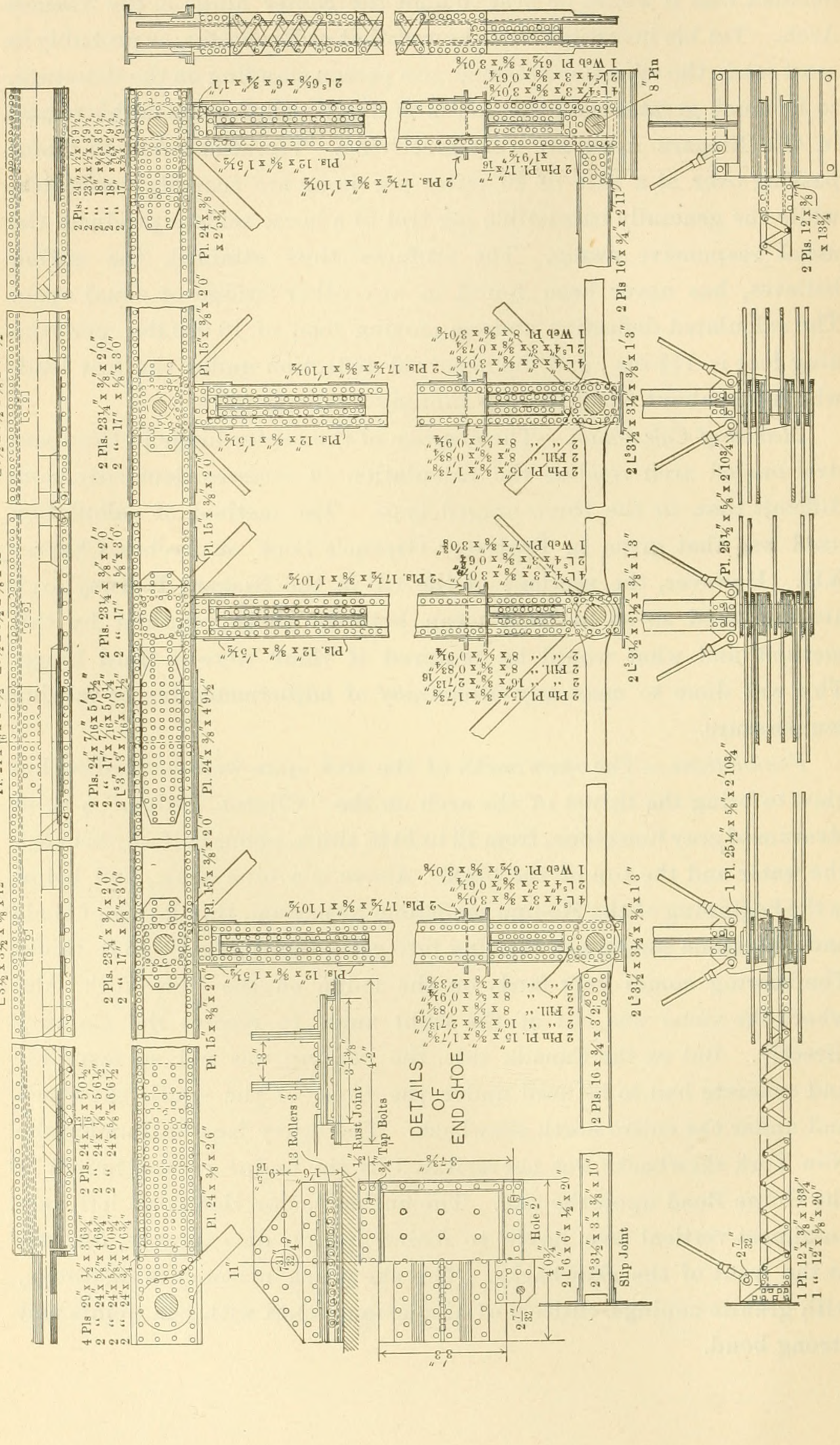

DETAILS OF END SHOE
Slip Joint
Rust Joint
Tap Bolts
Hole 2"
Pin
Web Pl.
Pls.
Fill.
Rollers
Pl. 24" x 3/8" x 2'0"
Pl. 15" x 3/8" x 2'0"
2 Pls. 23¼" x 3/8" x 2'0"
2 " 17" x 5/8"x 3'0"
2 Pls. 24" x 7/16" x 5'6½"
2 " 17" x 7/16" x 5'6¼"
2 L 5 3" x 5/16" x 3'9½"
Pl. 24" x 9/16" x 3'9½"
L 3½" x 3½" x 5/8" x 3'9½"
L 3½" x 3½" x 5/8" x 12"
4 Pls 20" x ½" x 3'6¾"
2 " 24" x ⅝" x 4'6¾"
2 " 24" x ⅝" x 6'0¾"
2 " 24" x ¾" x 7'6¾"
Pl. 24" x 3/8" x 2'6"
Pl. 25½" x ⅝" x 2'10¾"
2 L 5 3½" x ⅜" x 1'3"
2 Pls. 16" x ¾" x 2'11"
2 Pls. 12" x ⅜" x 13¾"
1 Pl. 12" x ⅜" x 13¾"
1 " 12" x ⅝" x 20"

FIG. 7.

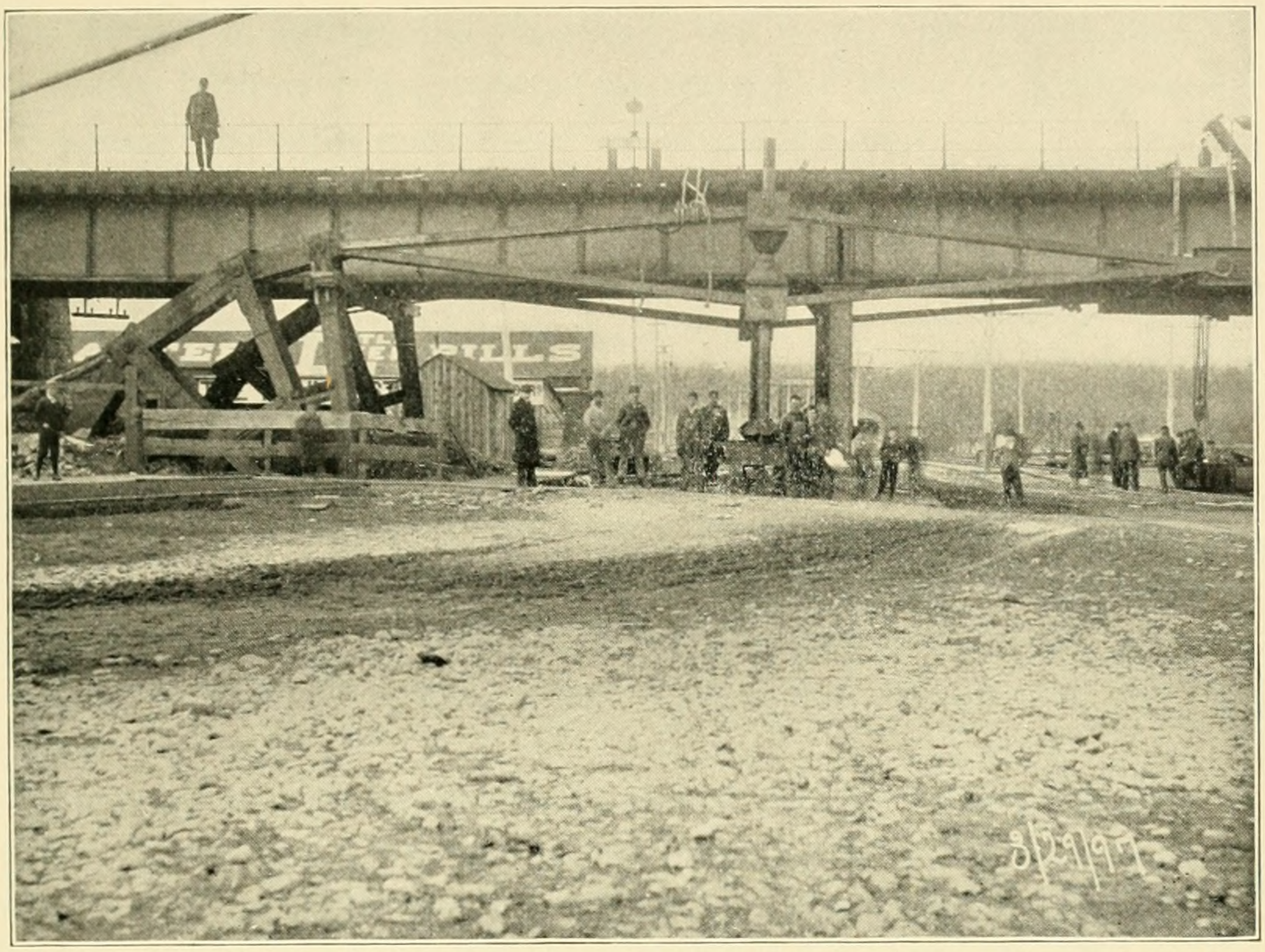


FIG. 1.

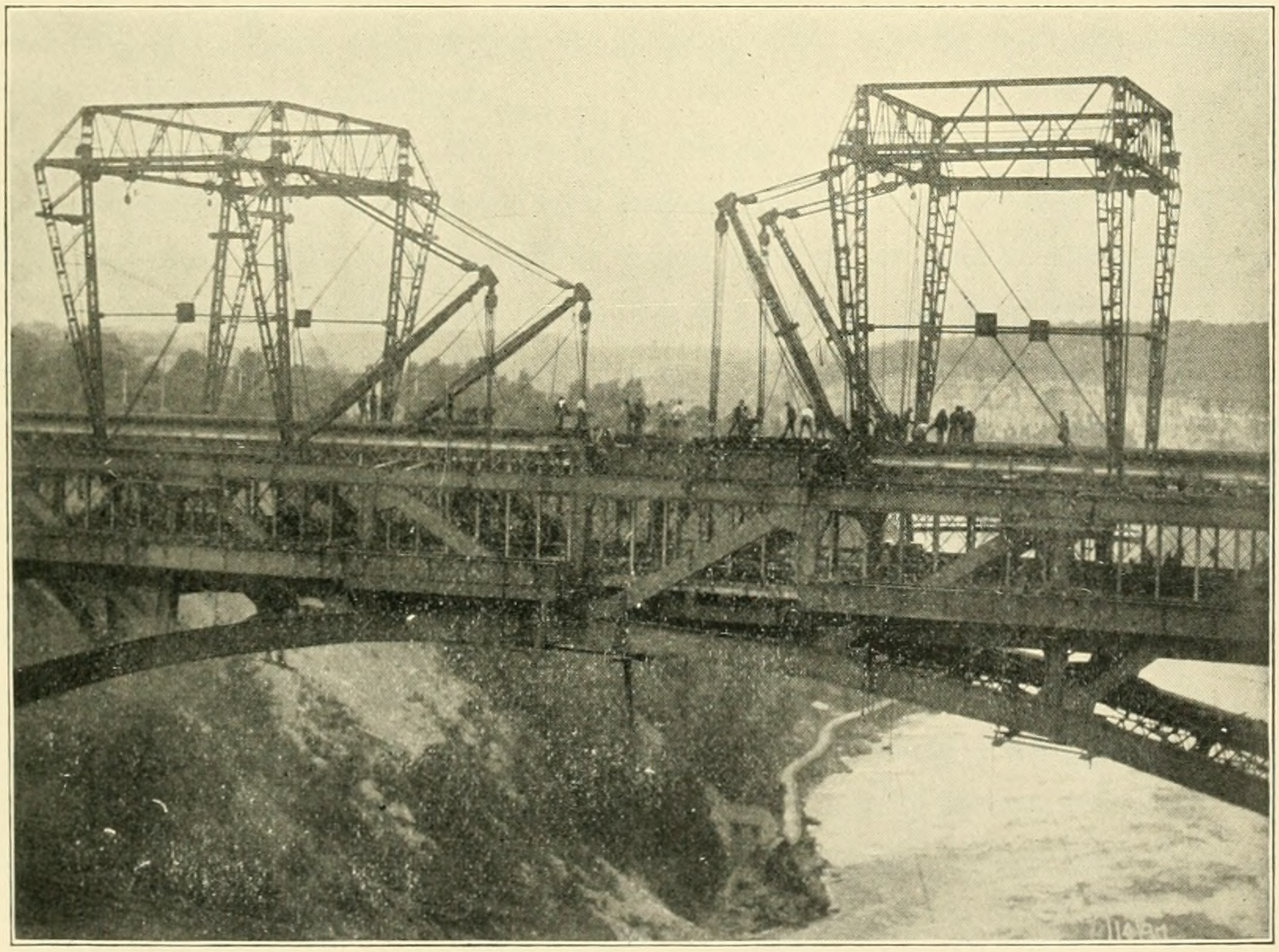


FIG. 2.

All masonry, except the retaining wall under the New York skew-backs, was put in with rented plant and hired labor, with results very satisfactory, both as to the cost and the quality of the work obtained.

The maximum loads on the masonry are as follows :

On top of coping	339	lbs. per square inch.
Under "	300	" " " "
On concrete	113	" " " "

Rust Joint.—The rust joint, between the masonry and the shoe, is a mixture of 32 parts of cast-iron filings to 1 part of sal ammoniac by weight, very thoroughly rammed. These ingredients and proportions were adopted after experimenting with several formulæ. Thorough ramming is the most important part of the operation.

End Bearings.—The details of the end bearings of the arch span are shown in Fig. 2.

They consist of two steel castings, having between the concave face of the lower and the convex face of the upper a nest of 45 segmental rollers set radially with respect to the center movement at *A*. The axis of the cylindrical bearing faces is likewise at *A*, and perpendicular to the vertical axial plane of the bridge. This form of bearing reduces frictional resistance much as a ball-bearing does, and was adopted to avoid the use of an excessively large pin, with which, movement is rather doubtful of realization.

In placing the rollers, the outside plugs *b* were inserted temporarily, to hold them in their correct radial position and render them fixed. At the top the rollers almost touch each other, and in the wider spaces at the bottom there are $\frac{5}{16}$-in. square bars to cause contact and restrict movement should there be any tendency to overturn. The bars, tap-bolted on both the upper and the lower castings at each end of the roller beds, are further safeguards against undue movement. Thus the rollers act like leaves, and can move either way through only a limited range without binding. As the movement of the rollers, due to moving load and temperature, is scarcely appreciable, there is no danger of the limit being reached. After the first panel of the arch was completed, connected with the anchorage, and swung back to correct position for proceeding with erection, the check plugs were removed and the rollers were thus freed. The center plugs and the guide bars remained permanently.

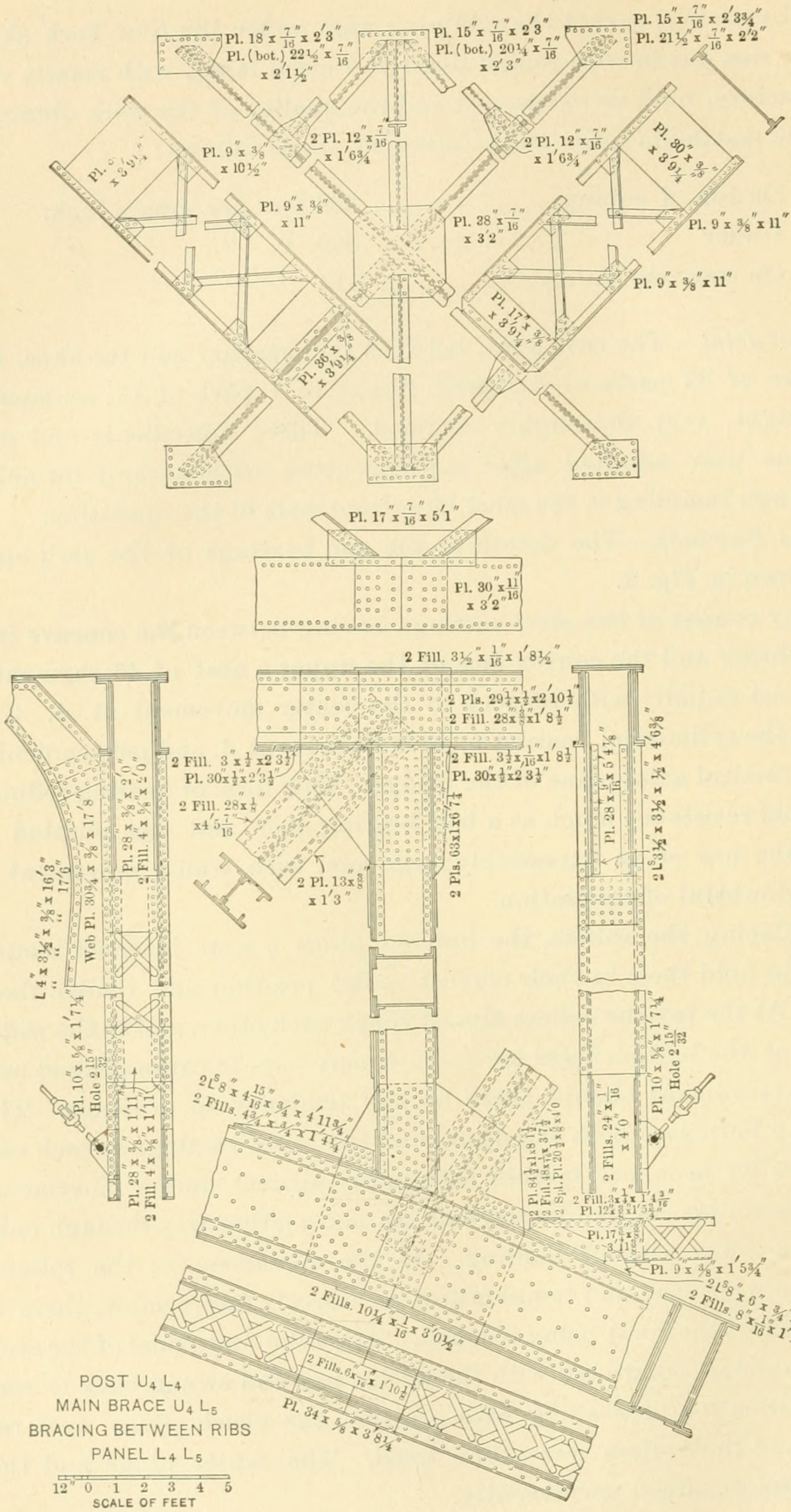

Pl. 18″ x 7/16″ x 2′ 3″
Pl. (bot.) 22½″ x 7/16″ x 2′ 1½″
Pl. 15″ x 7/16″ x 2′ 3″
Pl. (bot.) 20¼″ x 7/16″ x 2′ 3″
Pl. 15″ x 7/16″ x 2′ 3¾″
Pl. 21½″ x 7/16″ x 2′ 2″
2 Pl. 12″ x 7/16″ x 1′ 6¾″
2 Pl. 12″ x 7/16″ x 1′ 6¾″
Pl. 9″ x 3/8″ x 10½″
Pl. 30″ x 3/8″ x 3′ 9¼″
Pl. 9″ x 3/8″ x 11″
Pl. 38″ x 7/16″ x 3′ 2″
Pl. 9″ x 3/8″ x 11″
Pl. 9″ x 3/8″ x 11″
Pl. 17″ x 3/8″ x 3′ 9¼″
Pl. 36″ x 3/8″ x 3′ 9¼″
Pl. 17″ x 7/16″ x 5′ 1″
Pl. 30″ x 11/16″ x 3′ 2″
2 Fill. 3½″ x 1/16″ x 1′ 8½″
2 Pls. 29¼″ x ½″ x 2′ 10½″
2 Fill. 28″ x 1/32″ x 1′ 8½″
2 Fill. 3″ x ½″ x 2 3½″
Pl. 30″ x ½″ x 2′ 3½″
2 Fill. 28″ x ⅛″ x 4′ 5 7/16″
2 Pl. 13″ x 1/32″ x 1′ 3″
2 Pls. 63 x 1 x 6′ 7¼″
2 Fill. 3½″ x 1/16″ x 1′ 8½″
Pl. 30″ x ½″ x 2 3½″
Pl. 28 x 9/16″ x 5′ 4⅛″
2 L 3½″ x 3½″ x ½″ x 4′ 6⅜″
L 4″ x 3½″ x ⅜″ x 16′ 3″
“ “ “ 17′ 6″
Web Pl. 30¾″ x ⅜″ x 17′ 8″
Pl. 28″ x ⅜″ x 2′ 0″
2 Fill. 4″ x ⅝″ x 2′ 0″
Pl. 10″ x ⅝″ x 1′ 7¼″
Hole 2 15/32″
Pl. 28″ x ⅜″ x 1′ 11″
2 Fill. 4″ x ⅝″ x 1′ 11″
2 L 8″ x 4 15/16″ x ¾″ x 4′ 11 3/4″
2 Fills. 43/4″ x ¾″ x 1′ 4¼″
2 Pl. 84½ x 1 x 8′ 1½″
2 Fill. 48 x 1/16 x 3′ 7½″
2 Spl. Pl. 20½ x ⅝ x 4′ 0″
2 Fills. 24″ x 1/16″ x 4′ 0″
Pl. 10″ x ⅝″ x 1′ 7¼″
Hole 2 15/32″
2 Fill. 3″ x ¼″ x 1′ 4 3/16″
Pl. 12″ x 3/32″ x 1′ 5 3/4″
Pl. 17″ x ⅜″ x 3′ 11¾″
Pl. 9″ x ⅜″ x 1′ 5¾″
2 L 8″ x 6″ x ¾″ x 6′ 0½″
2 Fills. 8″ x 1/16″ x 1′ 11″
2 Fills. 10¼″ x 1/16″ x 3′ 0½″
2 Fills. 6 x 1/16″ x 1′ 10⅛″
Pl. 34″ x ⅝″ x 3′ 8¼″
POST U4 L4
MAIN BRACE U4 L5
BRACING BETWEEN RIBS
PANEL L4 L5
12″ 0 1 2 3 4 5
SCALE OF FEET

FIG. 8.

The bearing on the rollers, with maximum load on the bridge, is 2 200 lbs. per lineal inch of roller, assuming the pressure to be uniform on all the rollers. The upper and the lower castings were cast each in one piece. The $\frac{3}{4}$-in. plates on the bottom of the lower castings were intended as a precaution against any possible rupture of the castings.

The manufacture of the bottom castings gave considerable trouble on account of their failure to shrink in the usual manner of steel castings, which was doubtless due to the thinness of the metal and the unyielding nature of the cores.

The eye-bars connecting the rib directly to the lower casting were intended to prevent any possible displacement of the rib or upper casting, a precaution needed probably only during erection.

Trusses.—The trusses, as stated above, are spandrel-braced, with horizontal top chord and parabolic rib. They are battered 1 in 10. The inclination of the planes of the trusses with reference to the end bearings is provided for by a double beveled face on top of the upper casting. This is the only double beveled face in the arch.

The camber of the arch was designed to be 8 ins. at 60° Fahr. It has been observed to range from 10 ins. at summer heat to 7 ins. at zero.

The arrangement and details of the arch span, as well as of the end spans and approaches, are shown in Figs. 3, 4, 5, 6, 7, 8 and 9, and need no further explanation here.

Erection.—The erection was an interesting feature of this work. One of the main objects in view was to maintain traffic, and this was very fully accomplished. Not a single train was delayed, and traffic on the highway floor was suspended only for about two hours each day while the upper floor system was being put in, the time of day selected being that when there were the fewest trains. The lower floor was closed because of the danger to people passing below during the necessarily hurried operations of tearing out the old and putting in the new upper floor.

The deflection of the old bridge under moving load, the constant passing of trains, and the scant clearances at many points were considerations demanding close and constant attention; but as each anticipated difficulty was reached, it usually lost much of its formidable aspect. Besides, there was the comforting assurance that

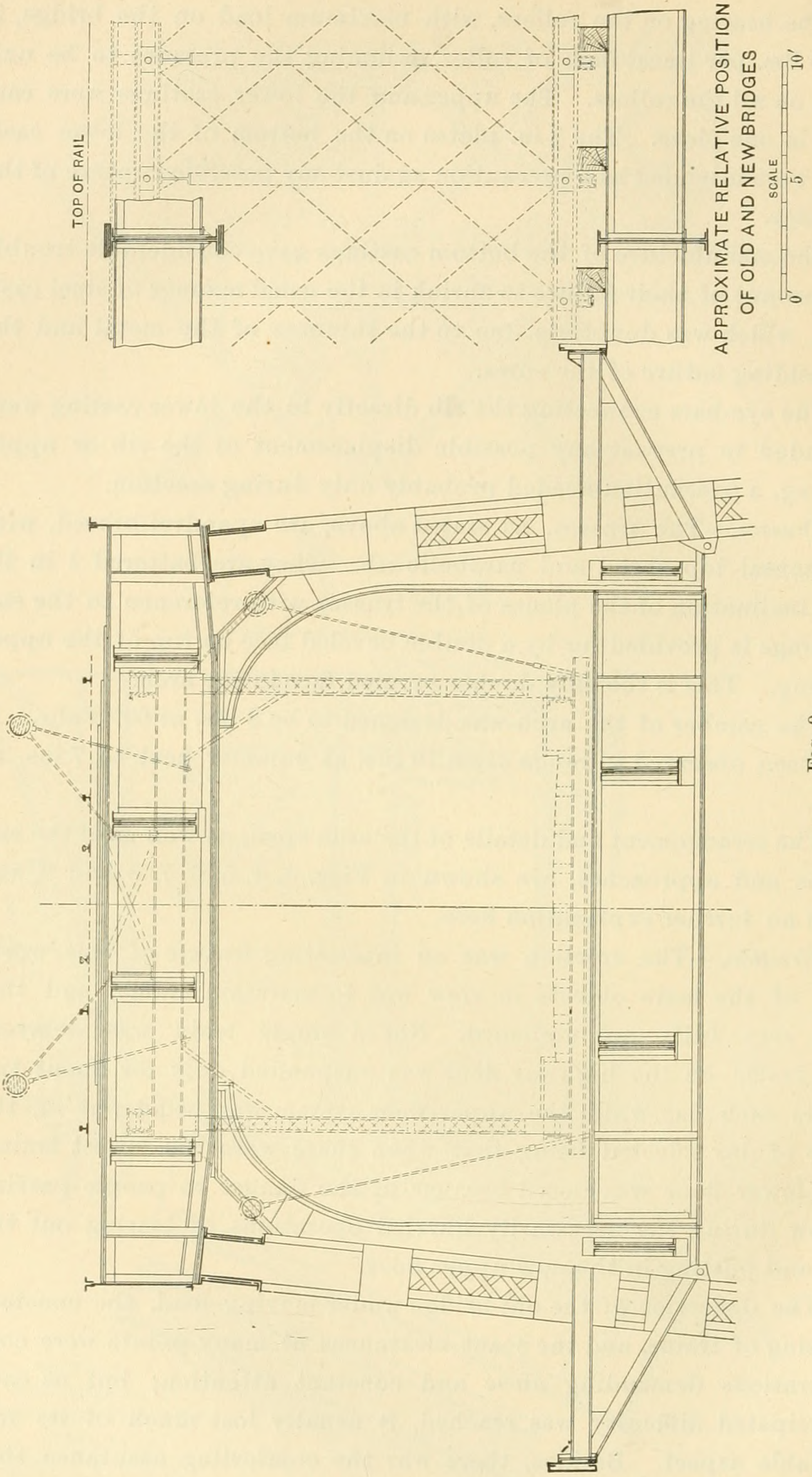

TOP OF RAIL
APPROXIMATE RELATIVE POSITION OF OLD AND NEW BRIDGES
SCALE
0'
5'
10'

FIG. 9.

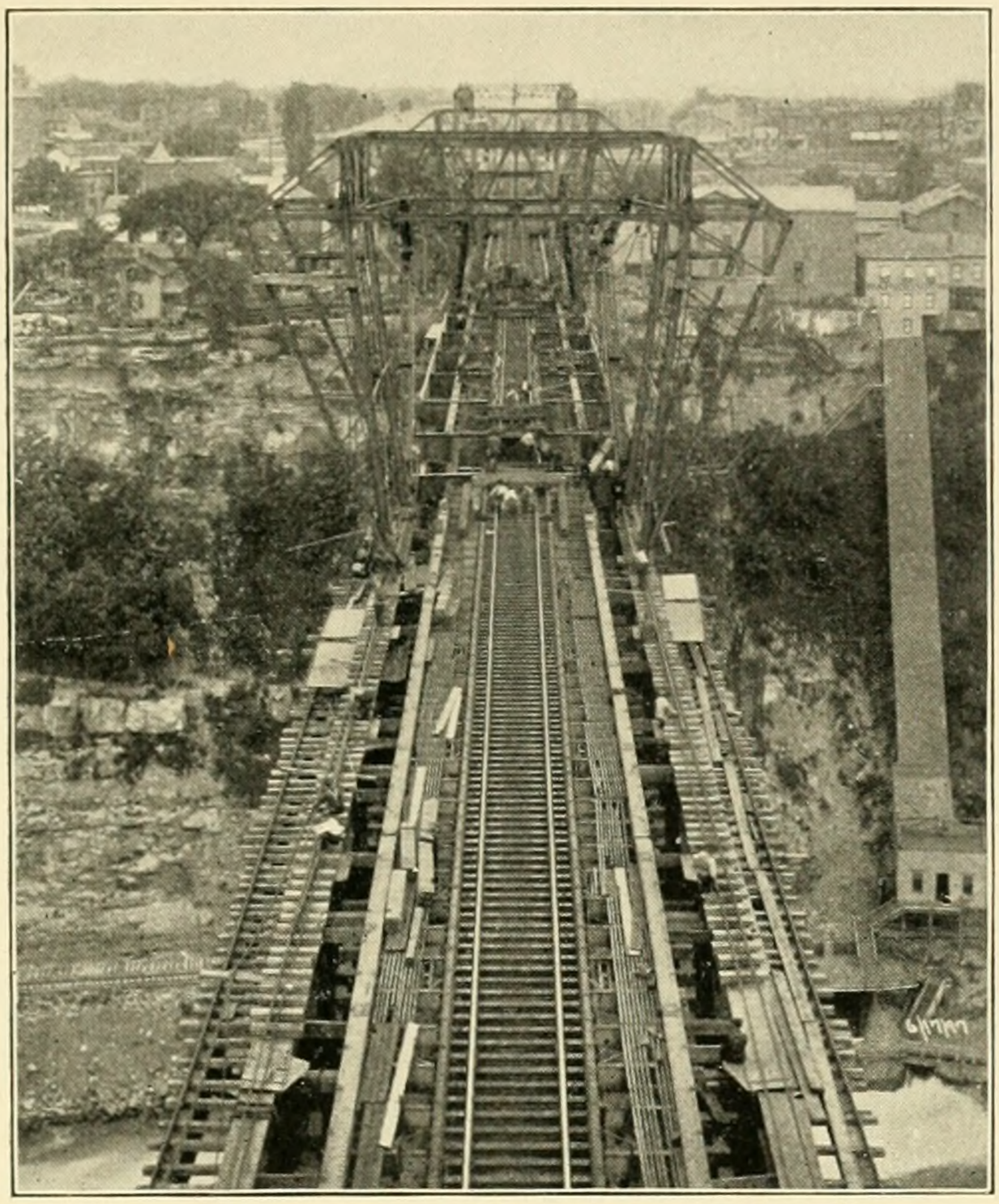


FIG. 1.

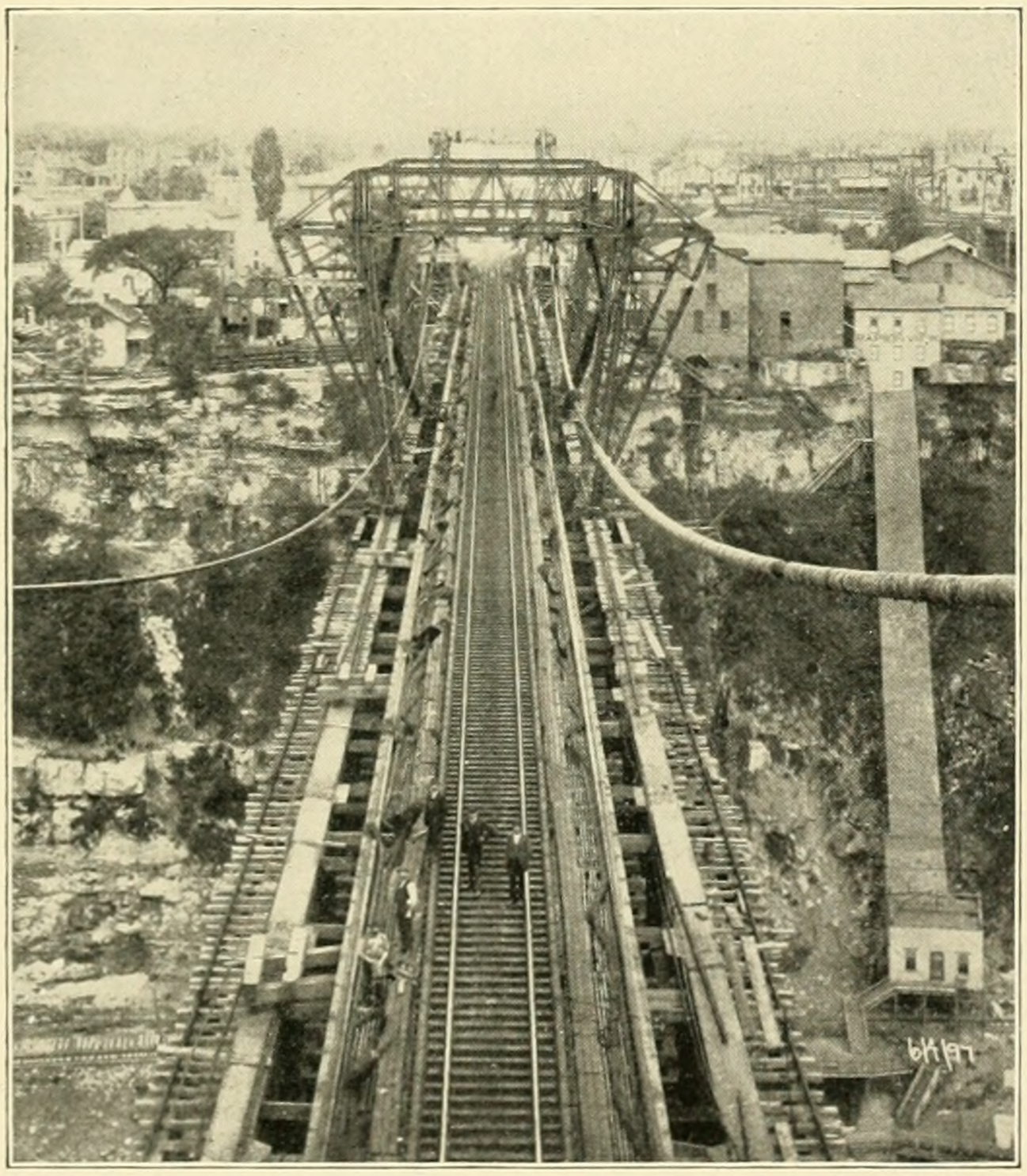


FIG. 2.

what had been accomplished with the old bridge under considerably less favorable circumstances ought to be accomplished again.

Briefly, the principle of erection was to build out the two halves of the arch as cantilevers anchored to the solid rock on top of the bluff, by means of adjustable anchor chains connecting with the arch at the top of the end post. The anchor chains consisted of the top chords of the end spans, such of the eye-bars of the end spans as were adaptable to the purpose, and such additional eye-bars and slabs as were necessary to complete the connections. The slabs were used as a matter of economy to serve as short eye-bars. The anchor chain was brought from vertical to horizontal by means of the "spider" shown in Fig. 10, which also shows the principal details of the anchor and the adjusting toggle. This is also shown in Fig. 1, on Plate IX. The anchor pits were cut into the solid rock, back near the anchor walls of the old bridge. They are shafts 3 ft. x 6 ft. in section, and 19½ ft. deep. Chambers were excavated at the bottom of sufficient size to admit the anchors. These anchor pits had to be excavated with great care, to avoid shattering the surrounding rock, and the work was done by hired labor. Border holes were drilled as closely together as possible, to the full depth of the shaft, and the core was then blasted out with light charges of dynamite.

After the anchors and the first two sections of the anchorage chain were placed, the anchor pits were filled with concrete to the top of the rock. Although no provision was made to allow the eye-bars bedded in the concrete to stretch without interference with the concrete, no cracks appeared on its surface until six panels of the arch had been completed. Then some very slight cracks were observed at the corners of the outside bars, but these showed no increase as the work continued to the center.

Adjusting Toggle.—The principle of the adjusting toggle is not new, but its adaptation to this case was very effective. Its operation is apparent from the figure. The right and left screw was turned by hand with capstan bars. Some doubt was felt as to its ability to lift as well as to lower the load coming on it from the weight of the half spans. But this was done without difficulty, nineteen men working each screw.

Erecting Plant.—Under the plan of erection originally contemplated, material was to be conveyed and placed by means of cable-ways sup-

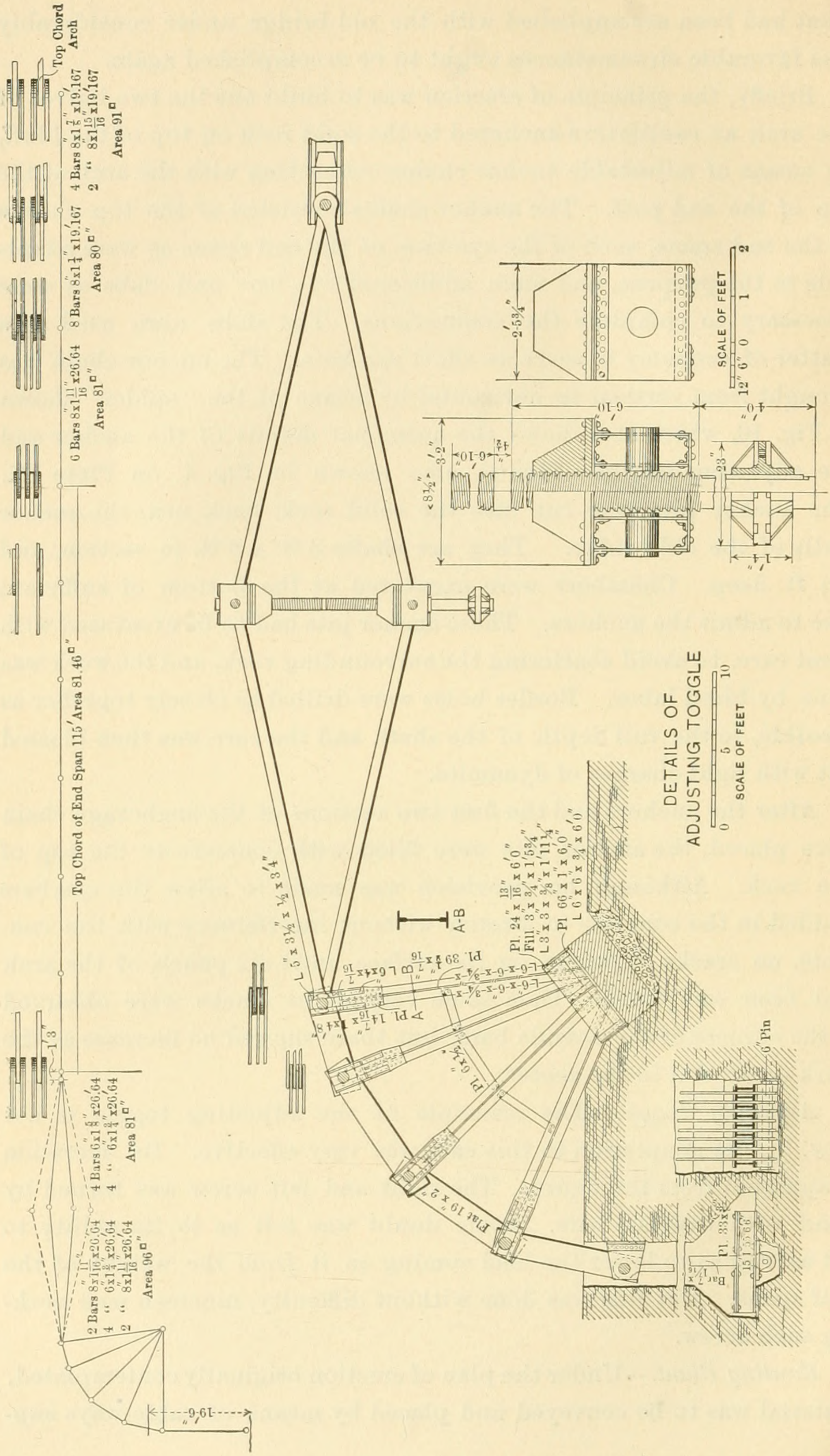

Top Chord Arch
4 Bars 8x1 7/8" x19.167
2 " 8x1 15/16" x19.167
Area 91 □"
8 Bars 8x1 1/4" x19.167
Area 80 □"
6 Bars 8x1 11/16" x26.64
Area 81 □"
Top Chord of End Span 115' Area 81.46 □"
4 Bars 8x1 5/8" x26.64
4 " 6x1 3/4" x26.64
Area 81 □"
2 Bars 8x1 11/16" x26.64
4 " 6x1 3/4" x26.64
2 " 8x1 11/16" x26.64
Area 96 □"
19'-6"
1'3"
11"
DETAILS OF ADJUSTING TOGGLE
SCALE OF FEET
0 5 10
SCALE OF FEET
12" 6" 0 1 2
2'-5 3/4"
6-10
4-0
3'-2"
81/2"
23"
A-B
L 5" x 3 1/2" x 1/2" x 3'4"
Pl. 24" x 13/16" x 6'0"
Fill. 3" x 3/4" x 1'5 3/4"
L 3" x 3" x 3/8" x 1'11 1/4"
Pl 66" x 1" x 6'0"
L 6" x 6" x 3/4" x 6'0"
Pl. 39" x 7/16" B L 6x4x7/16"
A Pl. 14 7/16" x 1" x 4'8"
Pl. 6" x 1/2"
Flat 19" x 2"
6" Pin
Pl. 33" x 3/8"
Bar 1 7/16" x 15"

FIG. 10.

ported on the towers of the old bridge. This method was used with very satisfactory results by Mr. Buck in the erection of the first Verrugas Viaduct, in Peru, but owing to the large cost of a plant suitable for handling heavy and unwieldy pieces, and their limited experience in using it for such purposes, it was proposed by the contractors to use travelers resting on the top chords of the arch, and the change was sanctioned by the Chief Engineer. The anchorages were strengthened to accommodate the additional weight. This erecting plant proved very safe and efficient. The two sides were entirely independent of each other, furnishing two points of progress, and when there was no outside cause of delay, the erection proceeded rapidly. There were two engines to each traveler, placed in the towers of the old bridge at the level of the railway floor, this being a good point of observation, and well out of the way.

Travelers.—The metal travelers required considerable special treatment, to clear the cables, and furnish the necessary clearance for trains. They are shown on Plates VII, VIII and X. The heaviest piece handled on this work weighed 32 tons, but the capacity of the travelers was considerably greater. For handling the rib members, special clamps were used to make them lie at the angle of the batter.

Progress of Work.—The false-work for the end spans was first erected, and the travelers raised on the outer bents, in which position they handled the skewback castings and first panels of the arch. The first sections of the rib rested on light false-work until the end posts and the braces in the first panel were placed. This much of the first panel was then lifted and held clear of the false-work, by ordinary tackle attached to the tops of the end posts. The false-work is shown in Fig. 1, Plate VII.

When the first top chord sections and the second pair of posts were placed, the pins were driven at the top of the end posts connecting with the anchorages. The end posts were then given the right inclination by means of the adjusting toggle. The traveler was then moved forward on the first panel of the arch, and in this position was ready for the erection of the second panel.

The material was conveyed to the travelers by means of trucks running on tracks on each side of the bridge. These tracks rested on the false-work as far out as the end posts of the arch span, and from

there to the center on the sidewalk brackets. The track stringers for the railway floor were used to carry these temporary tracks, being placed at their proper panels, ready for raising to final position when the railway floor should be put in.

The erection proceeded in this manner to the center. The lower floor system was put in, along with the trusses and lateral bracing. It was dropped below its normal position sufficiently to avoid the possibility of the weight of the old bridge coming on it, when deflected under passing trains, and thereby putting undue stress on the anchorages.

The closure at the middle was anticipated with considerable interest and some anxiety. The absence of the center hinge rendered great accuracy in laying out the work necessary, in order to secure proper closure and distribution of load between the top chord and the rib.

As a safety provision, the center panel top chord sections were not planed to length until six panels of the arch had been completed on each side, and a check measurement had been taken across the intervening space of about 134.5 ft. This measurement was not very satisfactory on account of the difficulties in the way of securing accurate results. The half spans were leaning back from their normal positions, their set and deflection could not be accurately accounted for, and the weather conditions were generally unfavorable. However, it was decided, after taking the measurement, to plane the center chord sections to theoretical length. When the center panels were erected, there remained an opening at the center of 8 ins. due to the two halves of the arch being drawn back, to secure the necessary clearance for placing these panels.

When all was ready, the adjusting toggles were slackened away together. In the proper order of events the top chords should have met first, and then, as those passed from tension to compression, the ribs should have met. But the reverse was the case, the ribs met first, and when the anchorages were entirely slackened off, there was an opening at the center of the top chord of $\frac{1}{2}$ in. This indicated no compression in the top chords at the center, whereas there should have been about 350 tons. The cause or causes of the failure to close, were not, at the time, very obvious, but it was decided that the adjustment could be duly effected after casting off the anchorages. The

FIG. 1.

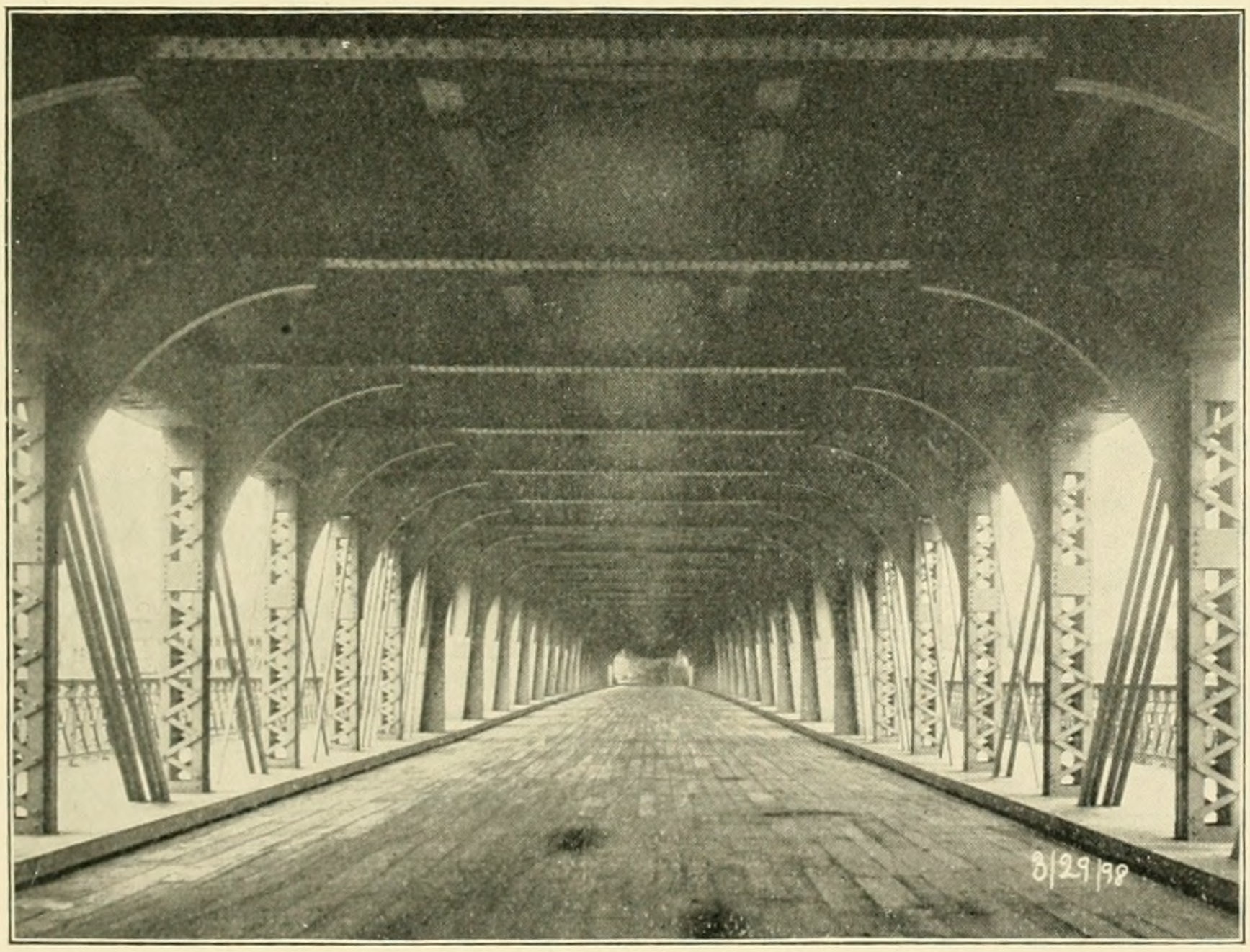
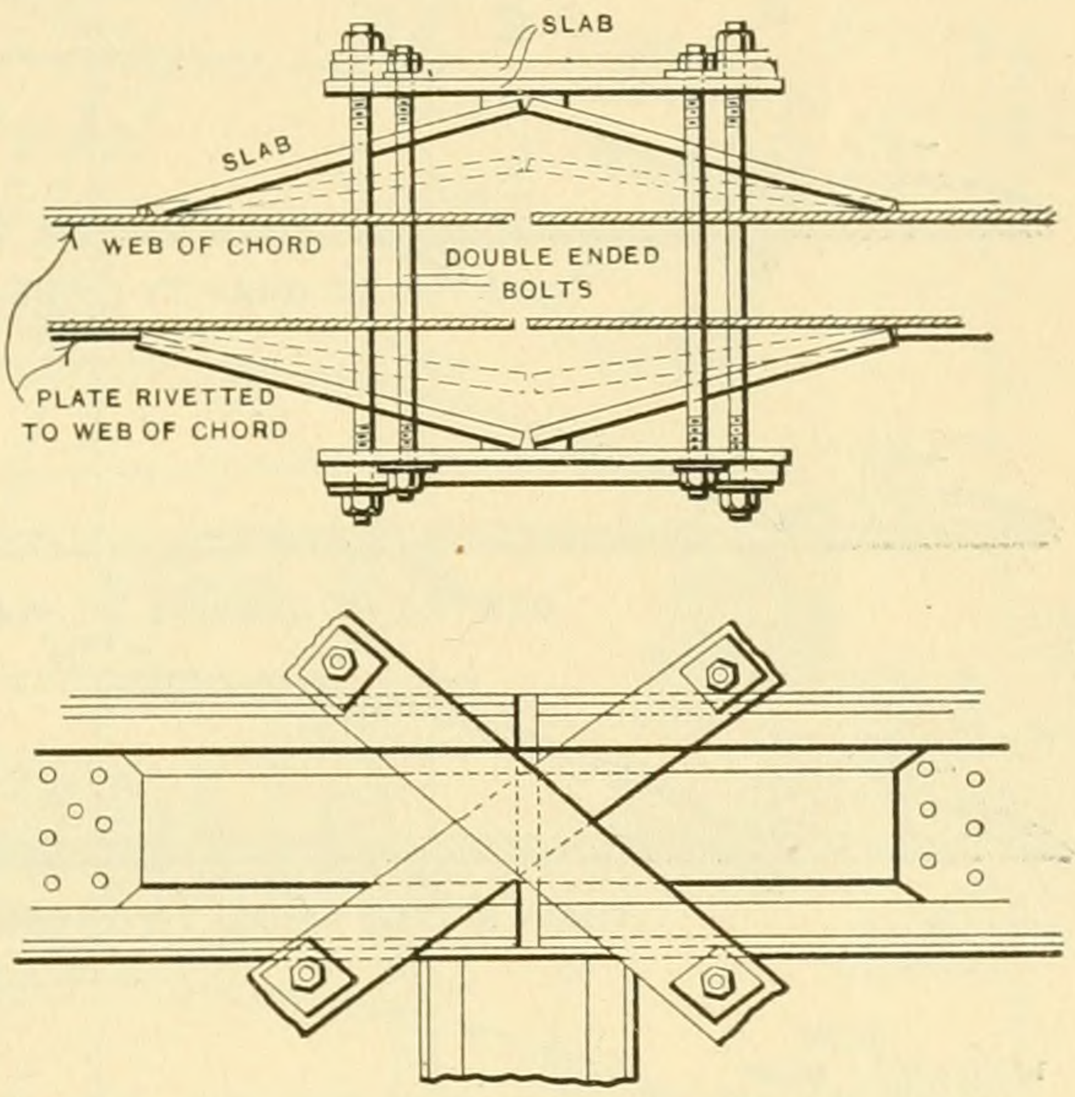


FIG. 2.

anchorages were cast off and taken apart. None of the joints were riveted up at this time, but almost all holes were filled with drift pins and bolts. Certain of the rib joints were open, the bearing faces being held apart by the drifts and bolts. When these were removed, so as to allow the bearing faces to come together, the opening in the top chord at the center was reduced to $\frac{1}{4}$ in.

It then became necessary, in order to secure the required compression in the top chord, to force it apart at the center and insert a shim. This was done by means of a compression toggle, shown roughly in Fig. 11. This toggle was improvised largely from material on the ground. The chords were forced apart until the opening was 1 in. wide, and a shim conforming to the section of the chord and of this thickness was inserted.



SKETCH OF
COMPRESSION TOGGLE USED
TO FORCE APART TOP CHORD
OF ARCH AT CENTER.
FIG. 11.

Both before and after the top chords were forced apart at the center, levels were carefully taken at each panel point for the purpose of obtaining the exact camber. The results indicated a slight elevation of the camber over the whole span after the adjustment, and in a closer conformity to the theoretical camber.

After the adjustment was effected, and the end spans completed, the lower floor system was raised to its final position. Timbers were laid crosswise on top of the roadway stringers, and when all was ready the stiffening truss was blocked up for its whole length on the new work. This was done between trains. The suspenders were then detached from the cables, and the cables were taken down. The wrapping was cut from the cables with axes, and the strands were cut at the shoes and lowered down, one at a time, on the bridge, where they were cut up for scrap.

After the removal of the cables, the upper floor was put in. In order to do this the upper floor and top chords of the old bridge had

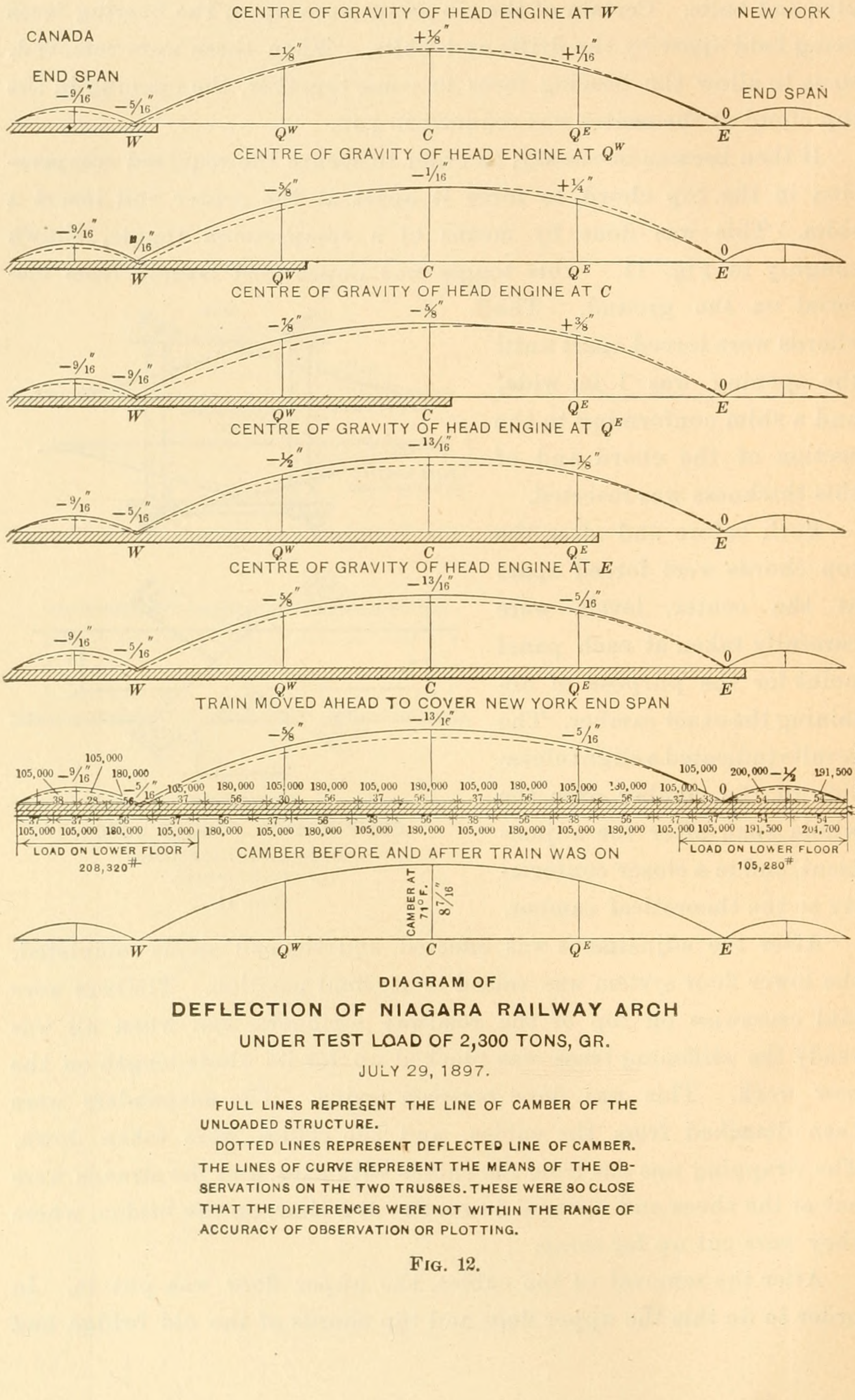


DIAGRAM OF

DEFLECTION OF NIAGARA RAILWAY ARCH

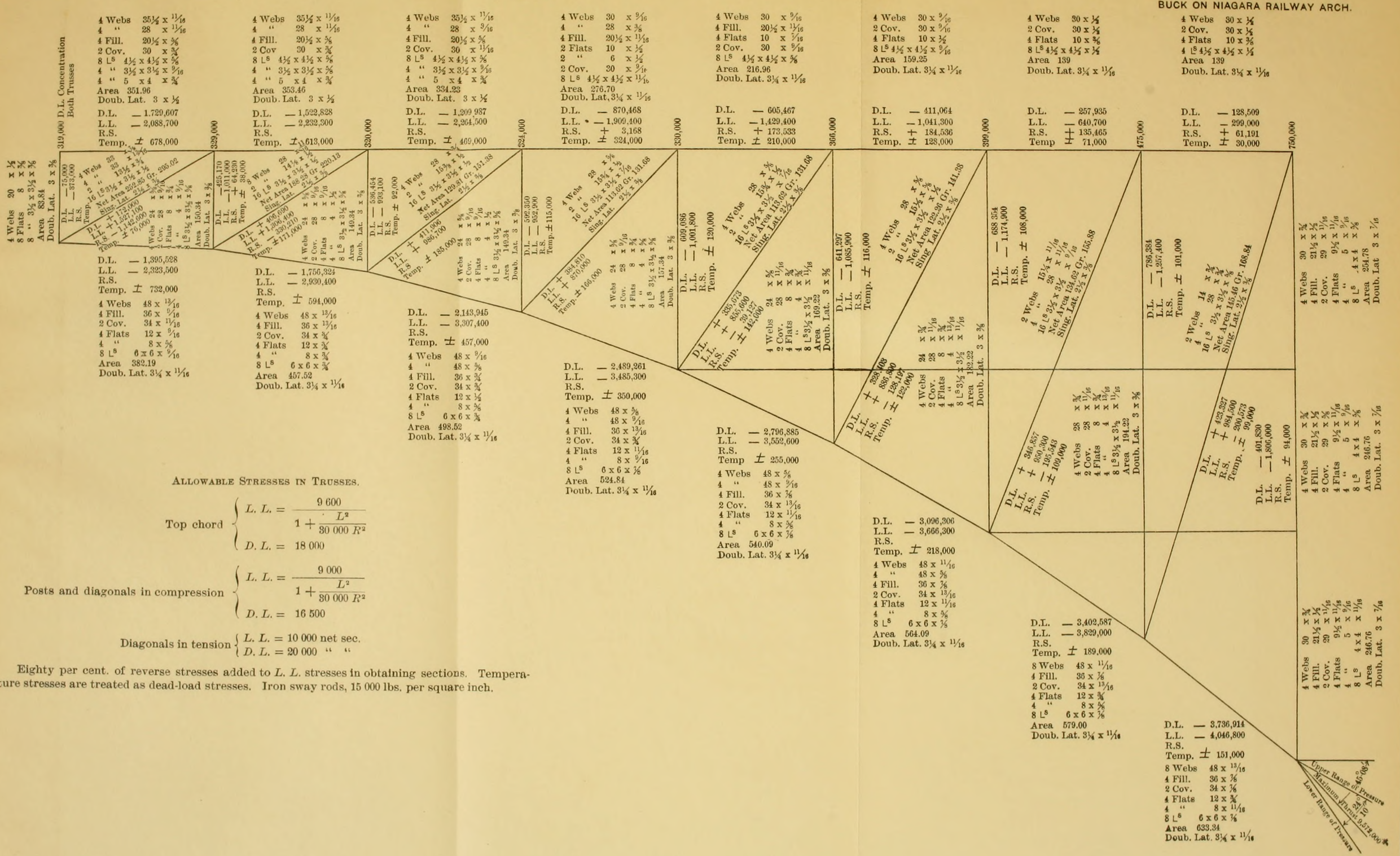
UNDER TEST LOAD OF 2,300 TONS, GR.

JULY 29, 1897.

FULL LINES REPRESENT THE LINE OF CAMBER OF THE UNLOADED STRUCTURE.

DOTTED LINES REPRESENT DEFLECTED LINE OF CAMBER. THE LINES OF CURVE REPRESENT THE MEANS OF THE OBSERVATIONS ON THE TWO TRUSSES. THESE WERE SO CLOSE THAT THE DIFFERENCES WERE NOT WITHIN THE RANGE OF ACCURACY OF OBSERVATION OR PLOTTING.

FIG. 12.



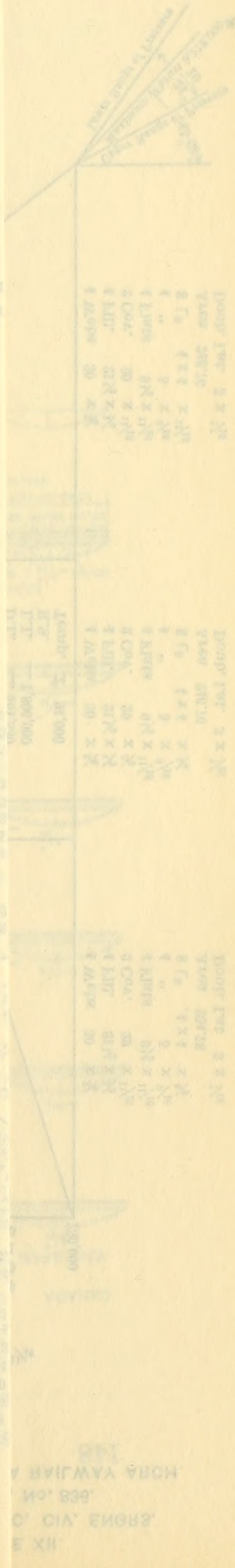
Allowable Stresses in Trusses.

Top chord $\begin{cases} L.\,L. = \dfrac{9\,600}{1 + \dfrac{L^2}{80\,000\,R^2}} \\ D.\,L. = 18\,000 \end{cases}$

Posts and diagonals in compression $\begin{cases} L.\,L. = \dfrac{9\,000}{1 + \dfrac{L^2}{80\,000\,R^2}} \\ D.\,L. = 16\,500 \end{cases}$

Diagonals in tension $\begin{cases} L.\,L. = 10\,000 \text{ net sec.} \\ D.\,L. = 20\,000 \text{ " " } \end{cases}$

Eighty per cent. of reverse stresses added to *L. L.* stresses in obtaining sections. Temperature stresses are treated as dead-load stresses. Iron sway rods, 15 000 lbs. per square inch.



to be removed. In order not to stop traffic this had to be done between trains, two panels at a time. The top chords and track stringers of the stiffening truss were cut into sections, conforming as closely as possible to the panel lengths of the new bridge, and the panels of the new bridge were put in as the sections of the old bridge were taken out.

Operations began at the middle, and after the first day, when only one panel was placed, two panels a day were put in until all were in place. The time allowed for this work was about two hours each day, and the work was always done within the time limit. The same track alignment was preserved, and the same rails and ties were used temporarily after the new floor beams and stringers were in place. When this work was completed as far back as the shore ends of the end spans, the towers were taken down, a high gin pole being used to remove the caps and upper sections, and the traveler to remove the lower sections.

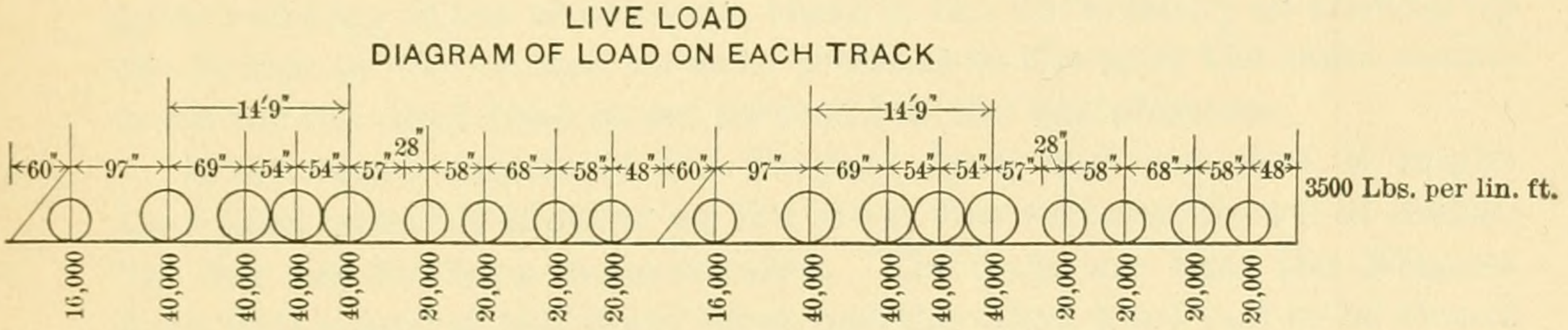


FIG. 13.

After the removal of the towers, the new plate-girder approach spans were substituted for the old ones. The maintenance of traffic here, while shifting, was more difficult than on the main and end spans, on account of the switches to be taken care of; but the change was accomplished without mishap. Plates VII to XI show various views of the bridge during construction.

Test.—On account of the difficulty of securing the full load of 10 000 lbs. per running foot, it was decided to make up two test trains as heavy as were available, and to observe the deflections under this loading. Each train consisted of two heavy Lehigh pushers, four of the heaviest Grand Trunk locomotives at hand and nine coal cars. The cars were of 30 tons capacity, loaded with coal, and had as many rails piled on top as was deemed safe for the cars. The loading is given in detail in Fig. 13. As indicated, some load was put on the lower floor, chiefly on the end spans.

The deflections under the test load are shown in Fig. 12.

The apparent slight irregularities in deflection are probably due more to inaccuracies of observation on account of the humid atmosphere and consequent refraction, than to any real irregularity of settlement of the structure under the load. The arch assumed exactly the same camber after the removal of the load as it had before the load was put on.

Ground was broken for the foundations of the arch span April 9th, 1896, and these were completed September 28th, 1896. The contract for the superstructure was let June 15th, 1896. The work of erection began September 17th, 1896, and the bridge was ready to test and was tested July 29th, 1897. All work on the bridge was completed August 27th, 1897.

In conclusion it should be stated that much credit is due to the Pennsylvania Steel Company, contractor for the superstructure, for care and efficiency in the prosecution of the work.

DISCUSSION.

J. M. KNAP, M. Am. Soc. C. E.—The speaker can urge but one objection to the paper. A good many years ago he had the honor of preparing a thesis on Roebling's Bridge, and predicted that it would stand fifty years, and he thinks that it would have done so had it not become necessary to replace it. Mr. Knap.

J. W. SCHAUB, M. Am. Soc. C. E.—The arch, by the use of curved instead of straight lines, at all times appeals to the eye through its beauty, and so does the Niagara Arch appeal to the eye, provided the eye is fortunate enough to get a proper view of it. The upper arch-bridge now being built is more fortunate in this respect, as, next to the falls themselves, it is now the most striking feature at Niagara Falls. Mr. Schaub.

In erecting an arch on the cantilever principle, without a hinge at the center joint, so many difficulties of vital importance to the integrity of the structure present themselves that the writer is surprised that such methods should be used. In addition to the internal stresses produced in the arch by changes of temperature and any possible yielding of the abutments, there is the uncertainty of closing up the bridge at the center, so as to produce in the arch the same conditions for the dead load as are assumed in the calculations.

If the upper chords of the Niagara Arch had met first, it might have been possible to close up the lower ribs without the use of shims, but this has not been demonstrated. The facts are that the Niagara Arch first closed at the lower joint, and the other joint had to be closed by means of shims; and it is a curious fact that the same thing happened over twenty-five years ago during the erection of the arch-bridge, commonly known as the Eads Bridge, at St. Louis. There, after repeated trials and failures to properly close the upper rib, the lower rib having closed first, the junction was finally made by means of an adjustable member, or practically by the use of a shim, just as was done at Niagara. It should be explained that the St. Louis Arch was a true arch, being composed of two parallel, curved ribs, braced, and was hingeless, with fixed ends. In both cases the dead-load stresses are vitiated to an unknown extent. In the case of the Niagara Arch the stress diagram calls for a dead-load stress at the center top joint of 864 800 lbs. for each truss. The actual dead-load stress now existing in the bridge is practically nothing, compared with the above. It is actually what was put there by screwing up four bolts on a toggle joint, and nothing more. In the St. Louis Arch the case was not so bad, for at the time of closing up the arch the entire superstructure which rests on the parallel ribs was not in place. At St. Louis the lever arm used in screwing up the first closing tube broke, dropping one or two men into the river. The remaining tubes were not screwed

Mr. Schaub. up so tight, and they let it go at that. In attempting to close up the arch one of the members in the bridge was actually ruptured, so that it had to be replaced. This will give some idea of the uncertainty involved in erecting a true arch on the cantilever principle,* In the Niagara Arch the entire dead load was on the bridge, and yet the top joint failed to close. In both cases all this uncertainty could have been avoided by the use of a center hinge, if properly used. The speaker does not recommend a center hinge for an arch similar to that used at St. Louis.

It is argued that a center hinge destroys the stiffness of the arch, and so it does. The difficulty is that the lack of stiffness of the center-hinged arch is not so much due to the hinge as to the fact that the lower chords, or ribs, are not straight. The greater the versed sine of this curvature, the greater will be the moment producing the deformation of the unloaded arm; or, in other words, if only one arm is loaded the other arm will have an amplitude of vibration directly proportional to the curvature of the lower rib. Hence, it follows, that if this curvature is made zero the amplitude of vibration in the unloaded

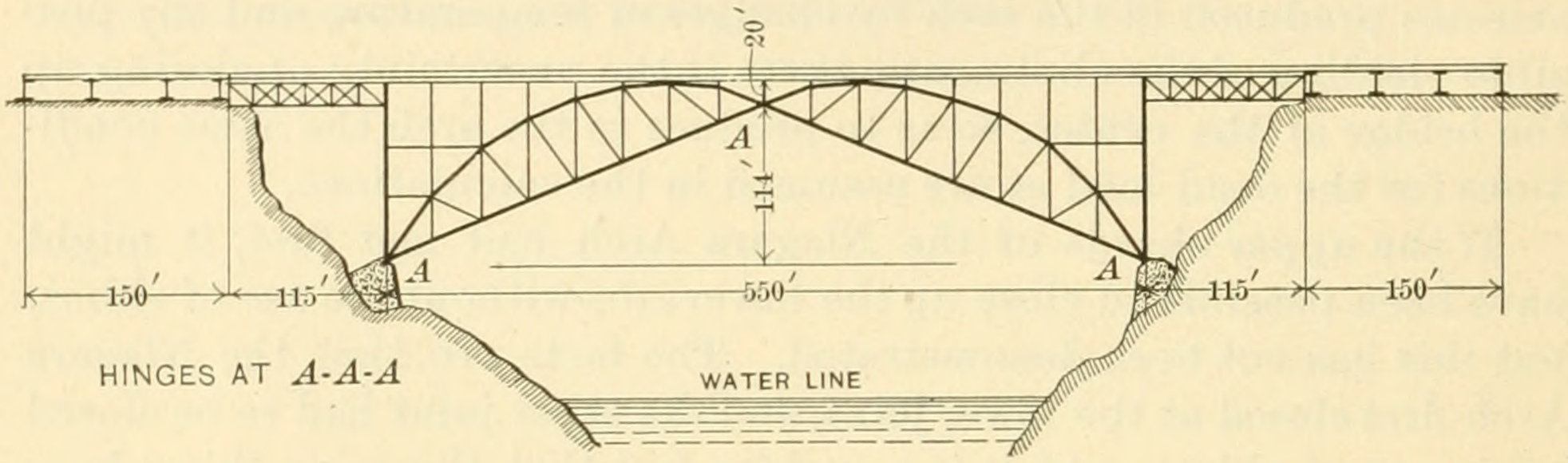


FIG. 14.

arm will be zero. This means that if the lower rib is made a straight line between hinges the deformation of the unloaded arm will be a minimum, and no reversal of stresses will take place.

It is even possible to design an arch so that no reversal of stresses will take place in either upper or lower ribs; then the only reversal of stresses would occur in the web members, and here they cannot be eliminated unless adjustable counter-rods are used. This would give the ideal arch, as far as stiffness is concerned, and, at the same time, eliminate all uncertainty due to the yielding of the abutments and to temperature. Above all things, the stresses would be clearly defined by statical methods, and, after the arch was closed at the center, there would be no question as to the final dead-load stresses.

The speaker begs to submit herewith a sketch, Fig. 14, showing an arch with a hinge at the center, and with the lower ribs composed of straight lines. This form of arch the speaker has had occasion to compute, and finds it a much more economical design than the arch

* "St. Louis Bridge," by Prof. C. M. Woodward, p. 190.

with the lower rib curved. This follows from inspection. The loads travel to their corresponding reactions by the shortest routes, and, by the theory of least work, this must give the stiffest and, at the same time, the most economical structure. Mr. Schaub.

GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The author states that when the cables of the old suspension bridge were taken down the wires curled up, showing that they had not lost the original set received from the wire drum. The same fact was observed by the speaker when he rebuilt the old Roebling suspension bridge over the Monongahela River, at Pittsburg, Pa., which fact he recorded at the time. * In the Monongahela Bridge the wires had been strained almost daily to half their breaking strength. It would be interesting to learn from the author what the greatest strain had been in the wires of the Niagara Bridge, and how frequently it occurred. Mr. Lindenthal.

That the natural rock abutments of the Niagara Cañon make the arch the proper type for bridging it was long self-evident.

The pains taken in this case to make the bridge very rigid deserve special notice, since this most desirable quality, great rigidity, is too often neglected from notions of false economy. In this respect a comparison with the widely-known cantilever structure of about the same span, and only a few hundred feet above the arch bridge, will not be amiss. The test load for the arch bridge is given at about 6 500 lbs. per lineal foot of bridge, while that for the cantilever bridge was about 4 500 lbs. † Yet the cantilever bridge deflected $7\frac{3}{8}$ ins., as against $\frac{13}{16}$ in. in the arch bridge. On this basis the arch bridge is theoretically thirteen times more rigid than the cantilever bridge. The meaning of this is that the arch bridge will be much more durable.

Several features contribute to its great vertical and lateral rigidity. First of all is the inclination of the arches from the vertical, which greatly lessens the disagreeable lateral swaying, so noticeable in bridges in which the arches are not inclined. Thus, nearest in length of span, and likewise of high rise, is the Washington Bridge, in New York, having six solid-webbed ribs and two spans of 510 ft. each. It is, next to the Niagara Bridge, the heaviest arch bridge in existence; proportioned, namely, for 8 000 lbs. live load per lineal foot as against 10 000 in the Niagara Bridge. Although the Washington Bridge is much wider (80 ft.) than the Niagara (about 55 ft.), yet the former is subject to such considerable lateral vibration as to have caused public comment on its supposed weakness, for which, of course, there is no foundation.

Another instance is the Margarethen Bridge in Budapest, which is a heavy structure of several spans, carrying a wide avenue over the

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* *Transactions*, Am. Soc. C. E., Vol. xii, p. 353.

† *Transactions*, Am. Soc. C. E., Vol. xiv, pp. 534–542.

Mr. Lindenthal. Danube. Like the Niagara Bridge, it is of the spandrel-braced type, without the center hinge, and has a buckle-plate floor and a stone pavement. The vibration is very marked, although not so perceptible as in another large arch bridge—the well-known St. Louis Bridge. All iron or steel arch bridges, having the arches in vertical planes, show the same peculiarity of lateral vibration.

On the other hand, arches with inclined planes, such as the Douro Bridge in Portugal, the fine bridge over the Adda, at Paderno, in Italy; the Gruenenthal Bridge over the Baltic Canal, and others like them, show most remarkable lateral stiffness. The bridges referred to have no middle hinge.

That arches without a middle hinge are stiffer than those having one requires no argument. It is true that the calculation of the strains in the arch without the center hinge is laborious, particularly so for the spandrel-braced type; but the labor is well spent for the advantage of a more rigid structure.

The use of riveted connections in the Niagara Bridge is a remarkable deviation from American practice; but that it was a proper choice cannot be questioned, and it is the more creditable to the engineer as the temptation to use pin connections, for greater ease of erection, was one not easily ignored. Increasing experience shows that pin connections should be used in bridges only for members subject to single stresses, either in tension or compression. Since in this bridge almost all members are subject to reversal of stress, riveted connections were the best.

The roller bearings under the arch footings is the one feature which the speaker could not approve; it will hardly find imitators. The author states that the roller bearings under the cables of the old suspension bridge were corroded and had become inoperative. The same thing will happen with the roller bearings of the arches which, however, will do no harm in this case, as the rotation of the arch footings from any cause is nearly *nil.* The speaker would have preferred pin bearings as a simpler and better construction.

The author would add greatly to the value of his description of the work if he would give, in the closing discussion, a synopsis of the calculations and the assumptions on which they were based. Among these are the moduli of elasticity; the panel loads for dead load; whether the top chord at the middle temperature is assumed to be without compression in the middle panels; and whether the calculated deflection (of 1 to $1\frac{5}{16}$ ins. for 10 000 lbs. live load per lineal foot) was obtained by taking account also of the changes of length in the web members. This would be particularly instructive, because the observed deflection ($\frac{13}{16}$ in. for 6 500 lbs. per lineal foot), compared with the calculated deflection, would then furnish a valuable index as to other results of calculation.

In a case like the present, widely different cross-sections for the same design and for the same loads, may be obtained by different computers, unless they are agreed on the premises from which the computations are started. Mr. Lindenthal.

The speaker confesses to be somewhat puzzled by the statement that in closing the arches during erection there should have been a pressure of 350 tons in each top chord. The author does not state that the arches were closed in the winter, or that the temperature was exceptionally low, although the views illustrating the paper show snow and ice on the ground. Even then the necessity for a pressure of 350 tons in the top chord is not clear, without a fuller explanation.

M. Lewinson, M. Am. Soc. C. E.—It would seem from the author's hinge design that as the reaction cannot be assumed to pass through one point, the result will be a rocking arch rather than a hinged arch. In the calculation of an arch like that at Niagara the first condition is that all reaction must pass through one point; that is, through the intersection of the lower chord with the vertical (or sometimes inclined) member. The line found by connecting the common center of gravity of the upper and lower chord sections, taken at various laminæ of the arch, is the line by which the arch should be calculated. The author does not state how he calculated it; therefore, assuming that the reaction passes through the pin, as designed, uneven stresses on the rockers result. It is a question whether the rockers are strong enough to withstand the strain. That the author has not been confident of his calculation of this problem is shown by the fact that he also calculated the arch as a three-hinged arch, and made corrections accordingly in the upper-chord stresses, which is not at all warranted. In a three-hinged arch, if it is advisable to put more compression in the upper chord, it is necessary only to locate the hinge in this chord. Assuming *a priori* all the points as correct, there is no necessity for trying to fill up the gaps in the theory of an arch which is entirely unlike an elastic arch. A three-hinged arch is a static arch, and that under discussion is an elastic arch. The stresses in an elastic arch are determinable, and the addition of material at the inflection points of such an arch is superfluous. Mr. Lewinson.

F. W. Skinner, M. Am. Soc. C. E.—This bridge is very interesting in comparison with the other equally large and important bridges of the present era, as embodying in itself a sort of epitome of the advance of long-span bridge construction from the first instances of it up to the present design for heavy railroad work. It represents four stages: First, there was the simple suspension bridge, without any pretense of stiffening. Next, the John A. Roebling bridge with its stiffening truss, imperfectly designed and more imperfectly constructed. Third, the development of that bridge into a thoroughly stiffened suspension bridge, and last, its entire replacement by a Mr. Skinner.

Mr. Skinner. wholly different type of construction. There is also to be noted the not less radical and striking advance in materials of construction and methods of work. The fact that owing to its location it has been visited by more engineers and others than almost any other bridge on this continent, and that since the first construction of the Roebling bridge the developments have been entirely under the direction of one man who has undertaken some of the most difficult, delicate and hazardous tasks, materially adds to its interest.

In the design of the present bridge the features that seem most striking are the excellent arrangements for the skewbacks, the elimination of the excessively large pins, the extension of the bearing surface at skewback joints and the avoidance of friction, and the portal bracing in the cross-bents in the upper floor-beams. These are most excellent features and are not seen often enough in similar work.

Mr. Lindenthal adverted to the riveted connections, and it may be apposite in that connection to refer to another arch bridge, lately built in Shenley Park, Pittsburg—the Panther Hollow Bridge. This bridge, which is of large span, was built with both pin-connected and riveted joints, was assembled on falsework and the primary connections made with pins. After the bridge was in position, the center panel connected and the arches swung, so that the pins were assumed to carry all the dead-load strains of the bridge, riveted gusset-plate connections were made at all the panel points; the intention being to so proportion and drive them that they should carry only live loads.

The speaker hopes that the author, in closing the discussion, will tell how many men were required to operate the toggle joint for lowering the bridge, how many were required for raising it, and about what speed was obtained; also why it was inadmissible, if it was inadmissible, to use that method of opening the top chord for inserting the key —why it was not used instead of the compression toggle.

It would also be interesting if he could add some data regarding the location of the pier centers; the measurement of the gap in the bridge; if there was anything in particular regarding the making of the core for the casting which he assumes may have had something to do with its shrinking; and whether it was a baked sand core or not.

As the completion of this bridge concludes so interesting a series of constructions, it seems proper to refer to some of the more important features of its predecessors on the site of this structure. It is true that they have have been explained briefly in two papers by Mr. L. L. Buck,* but an extremely brief reference to some of the undertakings involved in the reconstruction will go well with this description of the erection of the new bridge. In the first place, in the re-

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*"The Re-enforcement of the Anchorage and Renewal of Suspended Superstructure of Niagara Railroad Suspension Bridge," *Transactions*, Am. Soc. C. E., Vol. x, p. 195. "Replacing the Stone Towers of the Niagara Railway Suspension Bridge with Iron Towers," *Transactions*, Am. Soc. C. E., Vol. xvii, p. 204.

newal of the wires of the anchorage, the ends of the cables, where they unite with the shoes and anchor chains, were exposed and a large number of the outside wires were cut out under strain. Small sections were removed, the ends were scarfed and slightly notched, spliced pieces a little longer than the gap were added, and the wires were put under strain. The strain was measured by deflecting the uncut wires an inch or an inch and a half, and using this as a test for deflecting other wires to get a corresponding strain. The wires were so carefully spliced that they developed almost or quite the maximum strength of the original wires. The old anchorages were found to be somewhat deficient, both in the sections of the members of the chains and in the sizes of the anchor plates, and they were re-enforced by the building of an additional new anchorage for each end of each pair of cables. The new anchorage was built and sets of chains were attached to the shoes of the old anchorages at their connection with the cables, and the anchor chains were adjusted. They were put on by expanding them by heat until they were of a sufficient length, and the strains were measured and adjusted by an ingenious application of the principles of the modulus of elasticity, the bars having first been pulled in machines and the extensions for given strains noted. Both of these works, involving the exposure of the most critical part of the bridge, were carried on without material interruption to traffic and without in any way impairing the integrity of the bridge. Later on, in 1879–80, Mr. Buck proceeded to a still more delicate feat in the renewal of the suspended superstructure. In this case the old suspended superstructure amounted practically to a stiffened tube, a sort of combination truss with wooden compression members and iron rods. The new truss was designed on lines enabling it to practically fit inside of the old one. First the lower floor beams were all renewed. The bridge was then stripped as much as possible of unnecessary weight. Then 150 or 200-ft. lengths of the new lower chord were put in. Fortunately the old bridge members were nearly all double, so that half of them could be cut out without entirely destroying the efficiency of the bridge, and this was done in the floor beams, and later, after the lower chords were placed, in setting up the new vertical posts. Then the upper floor beams were placed, the new top chords were put in position, and half of the diagonal members. The trusses were connected up, and spliced at the ends to the old trusses. The intermediate part of the old trusses was removed, and the process was repeated, the entire old suspended structure being thus removed. After the trusses, the new track stringers were put in. After that the bridge was screwed up to the required camber, which, if the speaker's memory serves him right, was a circular arc of a radius of about 5 000 ft. Then the strains in the six hundred and thirty-six suspenders were all weighed,

Mr. Skinner.

Mr. Skinner. each of them carrying about 3 200 lbs. A hydraulic dynamometer was placed on each one and the strain carefully adjusted to correspond with the varying adjustments of the adjacent ones, and it was found that the suspensions in the suspender rods could be estimated by the touch within the limits of the dynamometer readings (25 lbs.).

The work of the replacement of the new truss was carried on under many disadvantages. It was difficult to place the top floor-beams, on account of the cramped position and the interference with the other work, and particularly difficult to place the lower lateral rods. In order to do it the workmen had to sit on the top flanges of the 8-in. lower floor-beams, working under the floors with very small headroom. There were only two men on the job who were willing to do this work, and one of those being discharged, it was a troublesome matter to get all the lower lateral rods in place.

The conditions of building the bridge were far different from those of the present day. The structure was built in a bridge shop lately converted from an old brewery, but with some special tools, and, under expert mechanical guidance, and by zealous and experienced contractors, it was erected without any steam power whatever. The work was done wholly by hand. Notwithstanding this, there was no hiatus in the work, no discrepancy, no trouble, no particular delay, and not a single serious accident.

The further reconstruction of the bridge is outlined clearly in the paper, but following it from the beginning, through the different changes to the present completed structure, with its extreme rigidity, it is a wonderful piece of work, and is a tribute to the courage, watchfulness, persistence and genius of its author, and a great credit to the able and successful execution of this last enterprise by his resident engineer.

From the first repairs of the anchorage, 20 years ago, no part of the complex and unprecedented work was undertaken until every possible contingency was provided for; remedies were devised in advance, not only for the difficulties that did arise, but that might possibly have arisen through mischance and did not, and well-placed confidence was inspired in all who were associated in the work.

The stiffening trusses of 1880 were the first, or among the very first, large bridge trusses in which riveted structural steel was used, and the character of the work throughout the whole period has been at once conservative and advanced practice.

Mr. L. L. Buck. L. L. Buck, M. Am. Soc. C. E.—The speaker's work at Niagara began 21 years ago. Four years after he went there, there was talk of another bridge just above the railroad suspension bridge, and it was proposed to put in a cantilever. It always seemed as though that was the place for an arch, and a spandrel-braced arch was the one that seemed best adapted to the place, because it was the most convenient

to erect without scaffolding under the main arch; which would, of course, be impossible in a place with a 20-mile current underneath and from 75 to 140 or 150 ft. depth of water. So the speaker designed an arch very similar to that which has now taken the place of the suspension bridge, but did not get a chance to compete on the design, though some parties had said that no doubt there would be an opportunity to do so, but that they would not build the bridge for some time to come. The "some time to come" proved to be just about a week, as in that time, they had let the contract, and were going right ahead. An arch bridge crossing the Genesee river at Rochester, designed by the speaker, was spandrel-braced and had three hinges, but, very much to the speaker's surprise, when the bridge was empty, with the exception of a double team of large horses, which kept time pretty well at a slow jog-trot, the bridge would teeter near the quarters. Mr. Alden, who did the work, said one day, "We must get rid of that teeter." "How much do you suppose it is, Alden?" "Well," said he, "three-quarters of an inch." He was told that if the whole range of the movement was $\frac{1}{8}$ in. it was as much as the speaker expected. A level and rod were procured, and the rod was laid on the sidewalk so that it had an inclination, and the level set up on the shore so that a sight could be taken at the rod. As it was raised up and down it would, of course, make a considerable movement transversely on the horizontal hair of the level, and in this way would indicate pretty fine readings of the amount of the up-and-down movement from the normal position. The movement was less than $\frac{1}{16}$ in. Mr. L. L. Buck.

In Brock's Monument—a stone tower—during a high wind, there is considerable motion which feels as though it was 3 or 4 ins., though probably it is not nearly as great an amount as that.

After building the three-hinged arch the speaker decided that the center hinge, although theoretically giving less temperature strain, was in many respects a mistake, as the reversals in the diagonals and in the top chords are greater, and the top chords give no assistance in supporting the load, their only office being to stiffen each half of the rib. The speaker's idea in this design was that if the stresses in a two-hinged arch were carefully calculated through all the members when the dead load alone was on, and the shortening of members in compression and the lengthening of members in tension accurately worked out, and the increase or decrease in the length of each as indicated by these calculations made; the top chords being in tension and the lower chords in compression in erection, the top chords would naturally meet first when the bridge came together. They would have done so, except for the fact that the connections being riveted, and the sheets held in position by drift pins and bolts, each gave a little at either end; the braces being in tension pulled slightly out of position, and the total movement amounted to considerable when the middle of the

Mr. L. L. Buck. span was reached, while the posts had just the opposite effect. This, the speaker thought, was the reason the top chords did not come together without a shim; but the shim did no harm at all. It required a little more work, but by turning one nut the toggle was compressed on both sides and with a given pull on the wrench, and a careful estimate of friction, the required pressure could be very closely obtained. The result proved the work, because if the pressure had not been right in the top chord a good many of the very long diagonals would have shown more or less tendency to spring sideways. This has never been shown, at any temperature, and it is safe to say that the work is practically correct, and that, there being no pin connection, every member of those trusses is helping to sustain the load. When the full load is on, the diagonals are in tension and the top and bottom chords in compression.

In making the calculations the lower chords at the middle point were lighter than the speaker liked to have them, and the middle portion of the top chords was very heavy; consequently, he increased the middle portion of the bottom chord by one-half of the difference between the three-hinged and the two-hinged arch, for which he is not sorry.

The speaker wishes to say that in addition to the valuable assistance the author rendered in the calculation of stresses, he has filled the position of resident engineer with great energy, excellent judgment and strong loyalty to his chief, and has contributed greatly to the success of the work.

CORRESPONDENCE.

HENRY GOLDMARK, M. Am. Soc. C. E.—There is so much that is interesting and valuable in this paper that the writer cannot help regretting its comparative brevity. Mr. Goldmark.

Among other points, a more detailed statement as to the method used in determining the stresses would be of much interest. It is to be hoped that the author may give some further information on this subject in closing the discussion.

While all arches with less than three hinges depend on the theory of elasticity, as well as on statics for their stress determinations, this particular form reduces the difficulty of computation to a minimum. Furthermore, a single system of bracing is used, with riveted connections at all points, so that the changes in the lengths of the separate members are purely elastic, with no play due to pin fittings.

The details of the bearings at the skewbacks are also very skillfully designed to reduce friction as much as possible, so that the usual assumption of frictionless hinges is very nearly justified.

Under these conditions, the only indeterminate quantity involved is the magnitude of the horizontal thrust H at the abutments. When this is known the stresses in all the members can be very easily found by simple statical methods.

In this respect the design compares very favorably with ribs having solid web-plates, and also with all forms of arches in which the bracing is arranged in multiple systems. In all of these the theory of stresses is more uncertain and the arithmetical computations more complicated.

The best method of solving all indeterminate frames is, beyond doubt, that known as the Method of Least Work, or Virtual Velocities. This method is mathematically exact, being in fact merely an extension to framework of the well-known mechanical theory of Virtual Displacements.

The first application of this theory to engineering structures was made by Clerk-Maxwell in 1865,* but it has subsequently been developed almost entirely by Continental writers, and made a useful instrument for solving all the more complex questions involving the stresses and deformations of structures. Reference, however, should be made to a valuable paper by G. F. Swain, M. Am. Soc. C. E., which is almost the only contribution to the Method of Least Work which the author has met with in the English language.†

In calculating a two-hinged spandrel arch, such as that treated of in the paper, the only assumption involved in determining the stresses

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* *Philosophical Magazine*, 1865.

† *Journal of the Franklin Institute*, Feb., Mar., April, 1888.

Mr. Goldmark. due to external loading is that of a Constant Modulus of Elasticity. For modern structural steel the error introduced by this assumption ought not to exceed 2 or 3 per cent.

The effect of temperature changes, is, of course, more uncertain, as their variation must necessarily be somewhat arbitrarily assumed. There is, however, no difficulty in adopting an extreme range, which will provide for all possible cases that can occur in practice, and even under this assumption the economy of omitting a third hinge is considerable.

The use of a central hinge at the crown of the arch is commonly assumed as obviating the necessity of taking temperature stresses into account. From the observations he has been able to make of a number of large arches, used either as bridges or roofs, the writer is convinced that this assumption is, in most cases, unwarranted. In every arch he has yet examined, including several designed by himself, the friction in the pin bearings was clearly sufficient to stop all rotation and hence to overthrow the condition of frictionless hinges, on which the entire theory of the three-hinged arch is based. In addition to the friction in the usual pin connections, the necessary floor fastenings in both highway and railroad bridges, and the roof covering in buildings, are usually sufficient to prevent all motion.

As an instance of this, the three-hinged arches in several of the World's Fair Buildings at Chicago may be mentioned, in which the two halves of the arch were quite rigidly connected by iron and wooden purlins and lantern girders. As these were entirely continuous across the central hinge, all rotation was effectually prevented.

Mr. Moncrieff. J. M. MONCRIEFF, M. Am. Soc. C. E.—It would add still further to the usefulness of an already valuable paper if some details could be given of the weights of steel in the various parts forming the main span, such as:

The combined weight per lineal foot of the arch frames (ribs, top-chords and web-bracing).

The weight per lineal foot of span of the lateral and sway-bracing.

The weight per lineal foot of the steel floor systems.

The weight per lineal foot of the remaining permanent load of timber, permanent way, etc., etc.

These details, together with a short statement of the allowed unit stresses, would enable one to make a comparison with other types, as to the economy of the structure in carrying the usual running load of 10 000 lbs. per lineal foot.

The writer notes also that no reference is made to wind pressure, either as to its amount, or influence on the structure.

With regard to the erection, the insertion of the shim in the top chords, by forcing the latter apart by means of the compression toggle shown in Fig. 11, appears to the writer to give no guarantee that the

required compression of 350 tons was either reached or not greatly exceeded, and as the permanent stresses in the various members depend, to some extent, on the stress in the top chords, there is, evidently, some uncertainty as to their actual value in the finished structure, over and above the uncertainties due to the assumptions usually made in the stress calculations for such structures as well as to conditions of erection, temperature, etc. That this uncertainty was present in the mind of the designer of the Niagara Arch is evidenced by the sections of the ribs having been increased to be a mean between those required for a two-hinged arch and a three-hinged arch. Mr. Moncrieff.

It is to be noted that the required compression of 350 tons forms an appreciable amount in relation to the total stress on the top chords, as given on the stress sheet accompanying the paper, assuming that the 350 tons and the figures on the stress sheet both refer to stress on two arch frames together.

With regard to the adjusting toggles in the anchor chains used for erection, the writer devised a similar means of adjustment in 1896 for the erection of the new Redheugh Bridge across the Tyne (England), now in course of construction from his designs.

This bridge is to consist of two spans of 248 ft., two spans of 168 ft., and a number of short plate-girder approach spans. The larger spans are to rest on three braced steel piers in the river and on two masonry piers, one on either river bank, and the erection, at a clear height averaging 87½ ft. above high-water level, is to be carried out without staging or falsework other than is afforded by the old bridge which is being replaced by the new structure. The old bridge is, however, in such a condition that no part of the new structure is to be allowed to rest on it during erection, and it will simply serve as a platform for conveying the material to the point of erection.

The main trusses are to be of the Petit type, and the heads of the inclined end posts of adjoining spans are to be connected together over each river pier by adjustable toggles. The trusses are to be erected as cantilevers on either side of the piers meeting at the center of each of the 248-ft. spans, and in the case of the 168-ft. spans from the river piers for the full span to the masonry shore piers.

As the erection of the main girders and their toggle adjustments is not yet commenced, it is a satisfaction to know that the toggles at the Niagara Arch bridge were so effective, both in raising and lowering the load.

The roller bearings at the heels of the Niagara Arch are, in their earlier life, no doubt, free from some of the objections accompanying the use of large pins, but unless means are provided for periodical cleaning, and proper attention is paid to this duty, there may be a positive disadvantage in having adopted rollers in preference to the much simpler pin.

Mr. Moncrieff. The writer notes that the material of which the rollers is made is not stated in the paper.

The engineer, and all concerned in the work, are to be congratulated on the successful completion of so handsome a structure under difficult conditions of erection.

Mr. Johnson. J. B. JOHNSON, M. Am. Soc. C. E.—The great amount of labor involved in the computation of a spandrel arch of the kind here given creates a demand for a simpler method. The writer therefore offers the following approximate method, and hopes it may have a fair trial at the hands of those who may be called upon in future to design such structures.

Since the great problem is to find the horizontal components of the reactions for the several joint loads, and since all full-spandrel trussed arches would be very much alike, as to general relative dimensions, the writer has taken the reactions as found for this Niagara arch (as kindly sent to him by the author), and has simply drawn lines through the springing representing their several directions, and extended these to intersections with their respective verticals through the several joints of the arch. The broken (dashed) line joining these intersections, b', c', $\ldots\ldots k'$, may be called the locus of the reaction intersections. If now a parabola be made to fit this true locus as nearly as may be, and its equation found in terms of the constants, span, l, rise of arch, r, and depth of crown, D_c, this may be called the "parabolic intersection locus," and it could be constructed in place, as soon as the general dimensions of the arch were known. The reaction lines could then be drawn to the intersections of the verticals with this curve, and the directions, positions and amounts of these reactions thus at once determined; that is to say, all the peculiar difficulties of the analysis for such a structure would disappear if this intersection locus were known.

In the present example, it can be seen by inspection of Fig. 15 that the error in the horizontal or vertical components due to an erroneous direction of the reactions, if these were drawn to the parabolic locus instead of to the true one, would be very small. Taking joint E as the worst case, the horizontal component would be in error about 2%, the vertical component about $2\frac{1}{2}\%$ and the resultant reaction by less than 2 per cent. The average single error for all the joints would be less than 1%, and these would be compensating, as they are of opposite signs. The resulting effect upon the dimensions of the members, for any combination of loads, would therefore be practically zero. For this one case, therefore, the parabolic locus would have served as well as the true locus, and another curve could be found which would fit the true locus still closer if it were thought necessary.

As to using the equation of this parabolic intersection locus for another bridge, as in a new design, there is only this to be said: The

intersection locus for a two-hinged arch of constant depth (parallel ribs) is not very different from the parabolic locus here found. The equation of this latter, when referred to an origin at the center of the line joining the two hinges, or springing points, is Mr. Johnson.

$$y = \frac{32\, l^2\, r}{25\, l^2 - 20x^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where

y = ordinate to locus curve,
l = span of arch,
r = rise of arch,
x = distance out to right or left from the center.

The curve marked a'', b'', c'' ..... k'' is this locus.

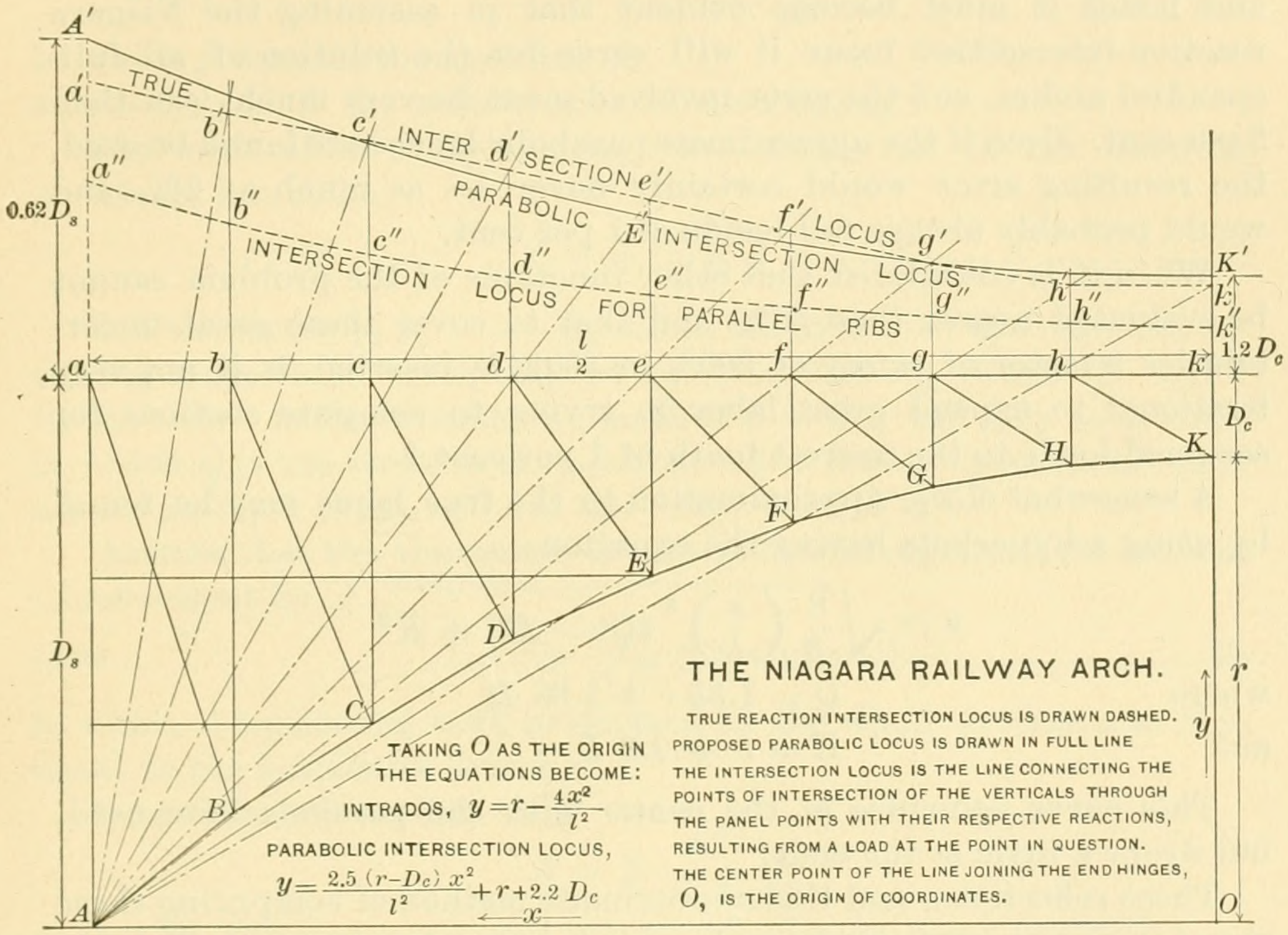

FIG. 15.

The equation of the parabolic locus here used for a full spandrel arch, referred to the same origin, is

$$y = \frac{2.5\,(r - D_c)\,x^2}{l^2} + r + 2.2\,D_c \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

The full-line curve, A', E', K' is this locus. It will be seen that if reaction lines were drawn through the springings to the locus, a'', b'' ..... k'', these would all have greater horizontal components than those drawn to either of the others. That is to say, the horizontal components of the reactions for an arch of constant depth (but whose

* See *Modern Framed Structures*, p. 210; Eq. (19); also, the works of Greene and Du Bois.

Mr. Johnson. moment of inertia increases with the secant of the angle with the horizontal from the crown to the springings, this condition having been assumed in the derivation of this locus), are all greater than the corresponding horizontal components for a full-spandrel arch; but the difference between actual stresses in a full-spandrel arch and the stresses which would result from using the locus a'', b'', ... k'' would be less than 5% on the average. That is to say, if the reactions were assumed to be the same for a full-spandrel two-hinged arch as they are for such an arch of constant depth, or having parallel ribs, the error involved could not exceed 5% on any member.

It is evident, however, that all full-spandrel arches are much more nearly like each other than they are like arches having parallel ribs, and hence it must become evident that in assuming the Niagara reaction-intersection locus it will serve for the solution of all full-spandrel arches, and the error involved must be very much less than 5 per cent. Even if the approximate parabolic locus here found be used, the resulting error would certainly never be as much as 2%, and would probably always fall inside of 1 per cent.

When it is considered that other functions of the problem cannot be evaluated nearer than 50%, and that to cover these great uncertainties a factor of safety of 300% or 400% is inserted, is it not very irrational to expend great labor in trying to compute sections for assumed loads to the nearest tenth of 1 per cent.?

A somewhat closer approximation to the true locus may be found by using a hyperbola having the equation

$$y = \sqrt{\frac{8}{3}\left(\frac{x}{l}\right)^2 (Q^2 - K^2) + K^2}$$

where $$Q = 1.35\, r + 1.85\, D_c$$

and $$K = r + 2.2\, D_c$$

This curve coincides at the center with the parabola here used, but drops a little at the ends.

These reflections, and this approximate method of computing these structures, are submitted to the good judgment of practicing engineers, rather than to the equally worthy members of the Society whose principal business is to apply exact theories to very definite but assumed problems.

Mr. Braune. G. M. Braune, Jun. Am. Soc. C. E.—The writer fully agrees with the author in his praise of the steel arch. In beauty, the steel arch is entitled to first place among the different types of bridges. Were it not for that statically indeterminate force—the horizontal thrust—it would, no doubt, appear more frequently in America.

The following equations for determining the horizontal thrust for the two-hinged arch with web members were taken by the writer from the lectures of the late Dr. Fraenkel, of the Technical High School of

Dresden, Saxony, and from Mueller-Breslau's Graphical Statics,* which work treats of the arch very fully, and in such a manner that the theory is easily understood by the reader. Mr. Braune.

If S represents a force working in the axis of a member m—m whose length is u; $\underset{m}{\longleftarrow}\!\!\overset{u}{\!\!\!\!\!\!\!\!\!\!\!\!\!\!\!\!}\!\!\underset{m}{\longrightarrow}\!\!\overset{S}{\!\!\!\!\!\!\!\!\!\!\!\!\!\!\!\!}$ a certain amount of work will be performed, which (assuming that the material of which the member is composed possesses the virtue of elasticity) will be equal to

$$A = \frac{S^2\, u}{2\, E\, F} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

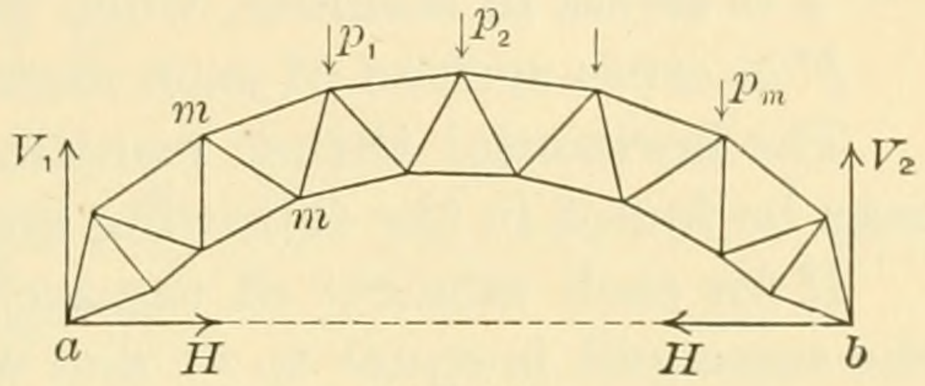

FIG. 16.

in which E denotes the modulus of elasticity, and F the cross-section of the member.

The work of a whole system, when forces act in the different members, would be:

$$A = \Sigma \frac{S^2\, u}{2\, E\, F} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Let the forces $p_1\, p_2$ ——— p_m act on the arch a–b; the reactions resulting from these forces may be resolved into V_1, V_2 and H. Let S be the stress in the member m—m, then $S = Q + s\, H$ may be written, in which Q is the stress in the member when $H = 0$, or when the arch acts as a simple beam, and s is the stress when $H = 1$.

Assume that the abutments are movable, and represent the amount of movement by $\Delta\, l$,

then $$-\Delta\, l = \frac{d\, A}{d\, H} \text{ (Castigliano)} \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which A equals the work performed in the whole system, and H is equal to the horizontal thrust.

It has been shown that

$$A = \Sigma \frac{S^2\, u}{2\, E\, F},$$

so that by differentiating, and substituting in (3), there results:

$$-\Delta\, l = \Sigma \frac{S\, u}{E\, F} \frac{d\, S}{d\, H} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

From $S = Q + s\, H$, is obtained $\frac{d\, S}{d\, H} = s$; and substituting these values in (4):

$$-\Delta\, l = \Sigma \frac{(Q + s\, H)\, u\, s}{E\, F},$$

and hence

$$H = \frac{\Sigma \frac{Q\, u\, s}{F} + E\, \Delta\, l}{\Sigma \frac{s^2\, u}{F}};$$

* "Statik der Baukonstruction," by Mueller-Breslau.

Mr. Braune. or, when the abutments are regarded as fixed,

$$\Delta l = 0 \Bigg\} \quad H = \frac{\Sigma \frac{Q\ u\ s}{F}}{\Sigma \frac{s^2\ u}{F}} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Q = stress in member when $H = 0$.
u = length of member.
s = stress in member, when $H = 1$.
F = cross-section of each member.

The horizontal thrust resulting from the change in temperature may be found in the following manner.

If for each member of the arch truss the change in temperature is the same and is equal to t°, and were the reactions removed, the arch would expand and assume a shape similar to its original form, and the line of horizontal thrust would lengthen itself, $a\ l\ t^\circ$, l being the span, a the coefficient of expansion, and t° the difference in temperature. Now, the abutments would prevent this expansion through two equal forces H_t working in the direction of the line of horizontal thrust.

Now, $$-\Delta l = \frac{d\ A}{d\ H_t},$$

therefore $$-a\ l\ t^\circ = \frac{d\ A}{d\ H_t} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Let S be the stress working in any member of the truss caused by H_t, then $S = H_t\ s$, and $\frac{d\ S}{d\ H_t} = s$ (and s = stress for $H_t = 1$).

It has been shown that $d\ A = \Sigma \frac{S\ d\ S\ u}{E\ F}$; substituting these values in equation (6) there results:

$$-a\ l\ t^\circ = \Sigma \frac{S\ u}{E\ F} \frac{d\ S}{d\ H_t} = H_t\ \Sigma\ \frac{s\ u}{E\ F}\ s$$

and, $$H_t = \pm \frac{E\ a\ t^\circ\ l}{\Sigma \frac{s^2\ u}{F}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which

E = the modulus of elasticity,
a = the coefficient of expansion,
t° = difference in temperature,
l = the length of span,
s = stress when $H = l$,
u = length of member,
F = cross-section of member.

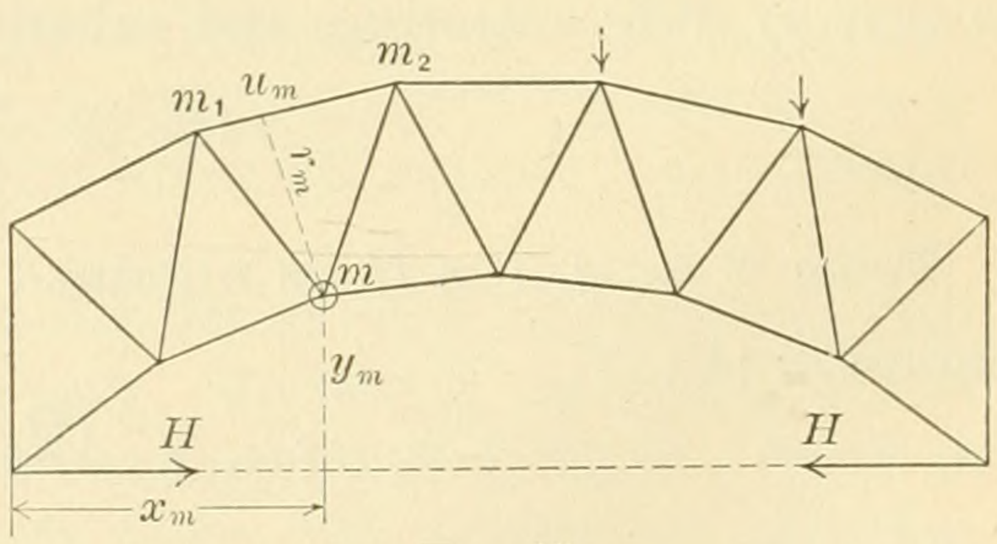

FIG. 17.

By disregarding the change in form (Formaenderung) of the web members, when forces act upon the truss,* equations (5) and (7) may be simplified in the following manner:

* Mueller-Breslau : Zeitschrift der A. & I. Verein, 1884, p. 581.

Mr. Braune.

Let u_m = length of member m_1—m_2.
r_m = perpendicular from m to u_m,
y_m = ordinate of panel point m,
F_m = section of u_m,
F_c = the most frequently occurring section.

$$H = \frac{\Sigma \frac{Q\ s\ u}{F}\ F_c}{\Sigma \frac{s^2\ u}{F}\ F_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5^a)$$

$$H_t = \frac{E\ a\ t^\circ\ l\ F_c}{\Sigma \frac{s^2\ u}{F}\ F_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7^a)$$

Let M_m be the moment of all the external forces about the point m and $M^1{}_m$ the moment about the point m of a simple beam, then

$$M_m = M_m{}^1 - H\ y_m,$$

and the stress in the member is:

$$S_m = \frac{M_m}{r_m},$$

or,
$$S_m = \left(\frac{M_m{}^1}{r_m} - \frac{H\ y_m}{r_m}\right).$$

It has been shown that $S_m = (Q_m + s_m\ H)$, in which Q = stress when $H = 0$, S = stress when $H = l$.

By comparing these two equations, it is found that

$$Q_m = + \frac{M_m{}^1}{r_m};\ \text{and}\ s_m = - \frac{y_m}{r_m};$$

hence,

$$Q_m\ s_m\ u_m \frac{F_c}{F_m} = + \frac{M_m{}^1\ s_m\ u_m}{r_m} \frac{F_c}{F_m} = \frac{M_m{}^1\ y_m\ u_m}{r^2{}_m} \frac{F_c}{F_m};$$

and,

$$S^2{}_m\ u_m \frac{F_c}{F_m} = - \frac{y_m\ s_m\ u_m}{r_m} \frac{F_c}{F_m} = - \frac{y_m{}^2\ u_m}{r^2{}_m} \frac{F_c}{F_m}$$

and, by substituting in equations (5^a) and (7^a),

$$H = \frac{\Sigma\ M_m{}^1\ w_m}{\Sigma\ Z_m} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5^b)$$

and,
$$H_t = \frac{E\ a\ t^\circ\ l\ F_c}{\Sigma\ Z_m} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7^b)$$

$$W_m = \frac{y_m\ u_m}{r^2{}_m} \frac{F_c}{F_m}$$

in which
$$Z_m = - y_m\ w_m = - \frac{y^2{}_m\ u_m}{r^2{}_m} \frac{F_c}{F_m}.$$

Mr. Braune. Now let a force P act at a point $x = a$ of the truss $t - t$, then the moment about a panel-point to the left of P will be:

$$M^1{}_m = \frac{P\,(l - a)}{l}\, x_m,$$

and for a panel-point to the right of P:

$$M^1{}_m = \frac{P\,a}{l}\, x^1{}_m.$$

If all the moments of the panel-points to the right and left of P are found and substituted in equation (5^b), there obtains:

$$H = P\,\frac{\frac{l-a}{l}\,\Sigma^{a}\, w_m\, x_m + \frac{a}{l}\,\Sigma^{l-a}\, w_m\, x^1{}_m}{\Sigma\, Z_m} \quad \ldots\ldots\ldots \quad (8)$$

Now if the values of w_m are multiplied by the force $P = l$, and the moments of these forces about the point $x = a$ are found (allowing the forces w_m to act, of course, in the respective panel-points for which they were calculated) there results:

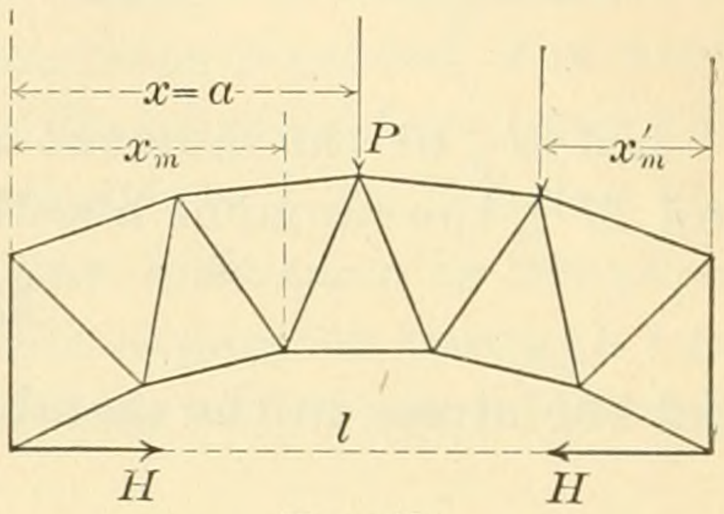

FIG. 18.

$$M\,w = \frac{l-a}{l}\,\Sigma^{a}\, w_m\, x_m + \frac{a}{l}\,\Sigma^{l-a}\, w_m\, x^1{}_m.$$

The right side of this equation is identical with the numerator of equation (8), and, by substituting, it is found that,

$$\left.\begin{aligned} w_m &= -\frac{Y_m\, u_m}{r^2{}_m}\,\frac{F_c}{F_m} \\ Z_m &= -\,Y_m\, w_m \end{aligned}\right\} \quad H = \frac{M\, w_m}{\Sigma\, Z_m} \quad \ldots\ldots\ldots\ldots \quad (9)$$

Therefore, to find the horizontal thrust which a force $P = l$ exerts at a point $x = a$ of the arch truss, the values $W_m = \frac{Y_m\, u_m}{r^2{}_m}\,\frac{F_c}{F_m}$ must be calculated and multiplied by $P = l$, and the bending moment of these forces (allowing them to act at the respective panel points, for which they were figured) found about the point $x = a$.

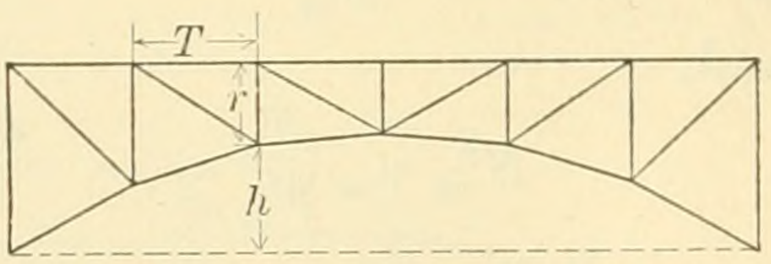

FIG. 19.

These equations may, at first, appear rather tedious, but after using them a few times they are solved very readily.

By putting $\frac{F_c}{F_m} = 1$, the equations are much simplified, and the results thus obtained (Mueller-Breslau) are very satisfactory.

The value F_c of the equation (7^b) may be calculated as follows: After having found the stress acting in a member T due to the live and dead loads, an equation,

$$S\,F_c = Q_L + Q_D + Q_t,$$

may be written, in which $S =$ the stress allowed per square inch, Mr. Braune.

$Q_L =$ the stress resulting from live load,

$Q_D =$ " " " dead "

$Q_t =$ " " " change of the temperature.

Having calculated the expression $\frac{E\,a\,t^{\circ}\,l}{\Sigma\,Z_m}$ from (7^b), let it be represented by B;

then, $$S\,F_c = -\,Q_L + Q_D + \frac{h}{r}\,B\,F_c,$$

and, $$F_c = \frac{Q_L - Q_D}{s - B\,\frac{h}{r}}.$$

For the case where the top chord is horizontal, and the panel lengths are equal, there may be written:

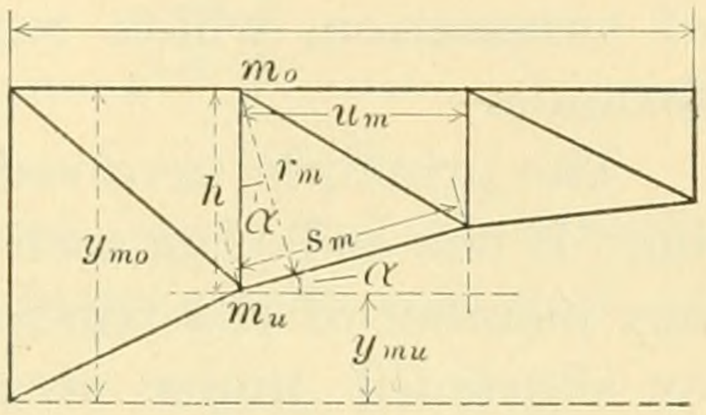

FIG. 20.

$$W_{mo} = \frac{S_m\,y_{mo}}{r^2{}_m}\,\frac{F_c}{F_m}$$

$$= \frac{U_m}{\cos\,\alpha}\;y_{mo}\;\frac{1}{h^2\,\cos^2\,\alpha}\,\frac{F_c}{F_m}$$

$$= \frac{Y_{mo}\,U_m}{\cos^3\,\alpha\;h^2}\,\frac{F_c}{F_m}$$

and $$W_{mu} = \frac{y_{mu}\;U_m}{h^2}\,\frac{F_c}{F_m}$$

combining these two

$$W_{mu} + U_{mo} = W_m = \frac{U_m}{h^2}\left(y_{mu}\,\frac{F_c}{F_m} \times y_{mo}\,\frac{F_c}{F_m}\,\frac{1}{\cos^3\,\alpha}\right)$$

For all practical purposes, since the panel-lengths are equal, these equations may be written:

by putting $$\left(\frac{F_c}{F_m} = \frac{F_c}{F_m}\,\frac{l}{\cos^3\,\alpha} = l,\right)$$

$$W_m = \frac{Y_{mu} + Y_{mo}}{h^2};\; Z_m = \frac{Y^2{}_{mu} + Y^2{}_{mo}}{h^2}$$

R. S. BUCK, M. Am. Soc. C. E.—In response to several requests, the author gives herein the outline of the method of calculation used in this design, and, in order to make clearer the essential points involved, has added some details and explanations not given in Professor Greene's book. Mr. R. S. Buck.

The removal of the center hinge eliminates the point that admits of an easy graphical determination of the proper force polygon for any loading, and the equation of condition for invariability of span is not applicable. This equation,

$$\Sigma\,\frac{E\,F\,.\,D\,E}{E\,I} = 0,$$

becomes $$\frac{1}{E}\,\Sigma\,\frac{E\,F\,.\,D\,E}{I} = 0$$

Mr. R. S. Buck. when I becomes a variable, as it does with the spandrel-braced arch with horizontal top chord.

In this equation $E\ F$ is the vertical intercept between the force polygon for any loading and the axis of the rib; and $D\ E$ is the ordinate to the same point on this axis (see Fig. 21). E is the modulus of elasticity of the material in the arch, which is usually constant. I is the moment of inertia of the truss, which varies rapidly, and cannot be determined at any point until the sections are known. It must, therefore, come within the sign of summation, which renders it impracticable to apply the above formula.

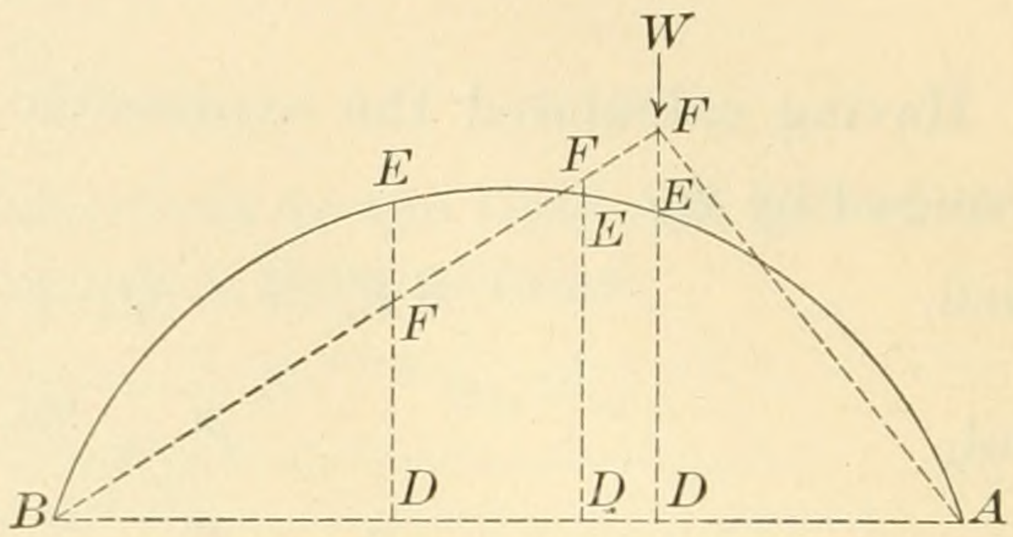

FIG. 21.

The principle involved in the Clerk-Maxwell method is the following: If one end of an arch is fixed and the other end free to move, and if any member of the truss is lengthened or shortened under stress, the horizontal movement of the free end of the arch due to this change of length will be proportional to the horizontal thrust at this point, if the movement were prevented. In other words, internal work of strain equals external work of resistance.

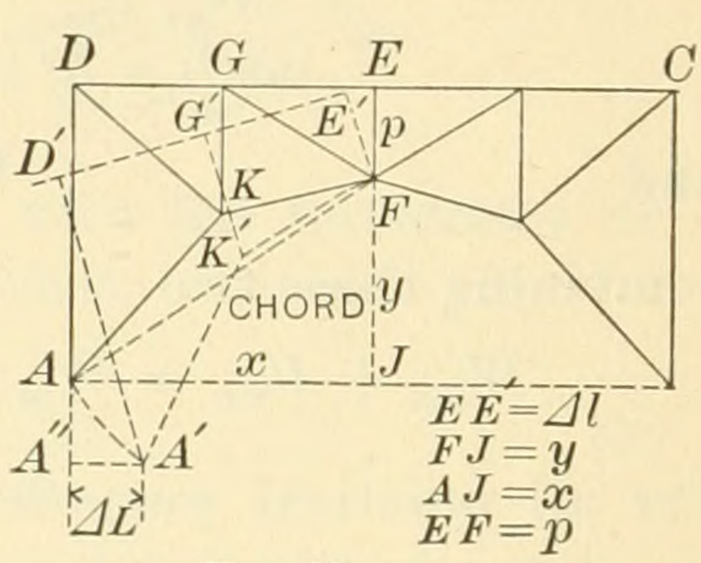

FIG. 22.

To demonstrate more fully (see Fig. 22):

Let T = stress in any member $G\ E$, no other member being under stress.

" A = area of cross-section of $G\ E$,

" E = modulus of elasticity "

" l = length of $G\ E$,

" Δl = elongation or compression of $G\ E$ under stress T.

Then $$\Delta l = \frac{T}{E\ A} \times l \ldots\ldots\ldots \quad (1)$$

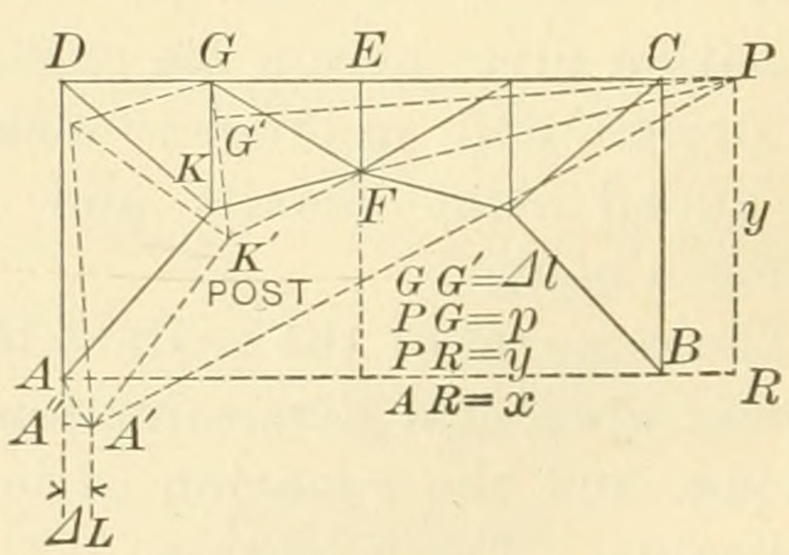

FIG. 23.

Now, if we consider the position of the arch on the right of $G\ E$ to be held rigidly and this member lengthened or shortened an amount Δl, for a slight distortion motion of the free end of the arch, F can be considered as taking place about F as a center, this being the center of moments of the piece in question. We then have

$$A\,A' : E\,E' :: A\,F : E\,F \,.\cdot.\, A\,A' = \frac{E\,E' \times A\,F}{E\,F} = \frac{\Delta l \times A\,F}{p}$$

Again $$A\,A' : \Delta\,L :: A\,F : F\,J \,.\cdot.\, \Delta\,L = \frac{A\,A' \times F\,J}{A\,F}$$

$$= \frac{y}{p}\,\Delta l \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

In other words, the horizontal component of the movement of the free end of the arch is to the change of length of the member stressed as the vertical ordinate of the center of moments of the member is to the perpendicular to the member dropped from the center of moments.

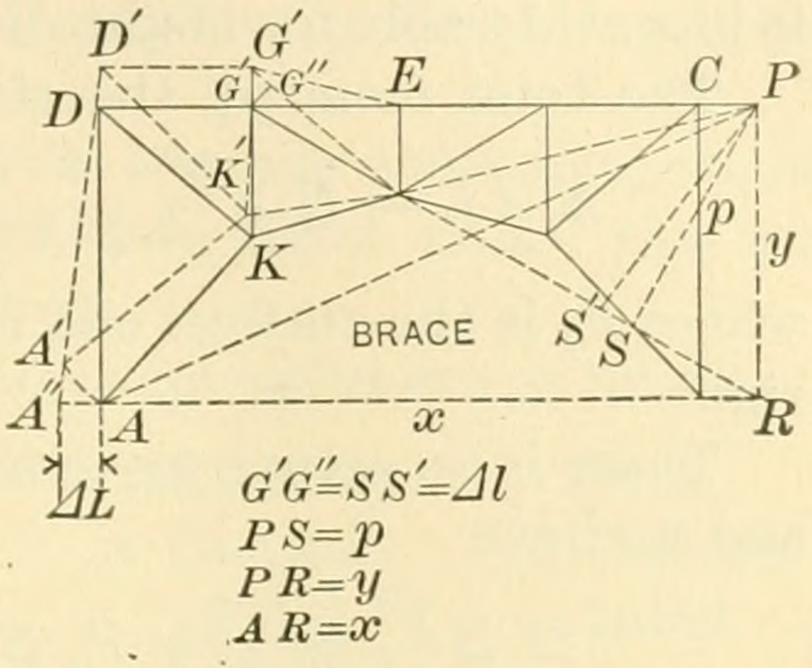

FIG. 24.

This is more readily apparent in the case of the top chord than of the vertical and inclined members. The distortion of the truss for change of length of post, brace and rib is given in Figs. 23, 24 and 25; and the operations of obtaining the values of ΔL for these cases are as follows:

$$\textit{Post.}—A\,A' : G\,G' :: A\,P : P\,G \,.\cdot.\, A\,A' = \frac{G\,G' \times A\,P\,.}{P\,G}$$

$$A\,A' : \Delta\,L :: A\,P : P\,R \,.\cdot.\, \Delta\,L = \frac{P\,R \times A\,A'}{A\,P} = \frac{\Delta l \times A\,P}{p}$$

$$= \frac{y}{p}\,\Delta l$$

$$\textit{Brace.}—A\,A' : S\,S' :: A\,P : P\,S \,.\cdot.\, A\,A' = \frac{\Delta l \times A\,A'}{P}$$

$$A\,A' : \Delta\,L :: A\,P : P\,R \,.\cdot.\, \Delta\,L = \frac{y \times A\,A'}{A\,P} = \frac{y}{p}\,\Delta l$$

$$\textit{Rib.}—A\,A' : K'\,K'' :: A\,G : G\,S \,.\cdot.\, A\,A' = \frac{\Delta l \times A\,A'}{P}$$

$$A\,A' : \Delta\,L :: A\,G : G\,R \,.\cdot.\, \Delta\,L = \frac{y \times A\,A'}{A\,G}$$

$$= \frac{y}{p}\,\Delta l$$

Now substituting in equation (2) the value of Δl given in equation (1) we get,

$$\Delta\,L = \frac{T}{E\,A}\,\frac{l\,y}{p} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

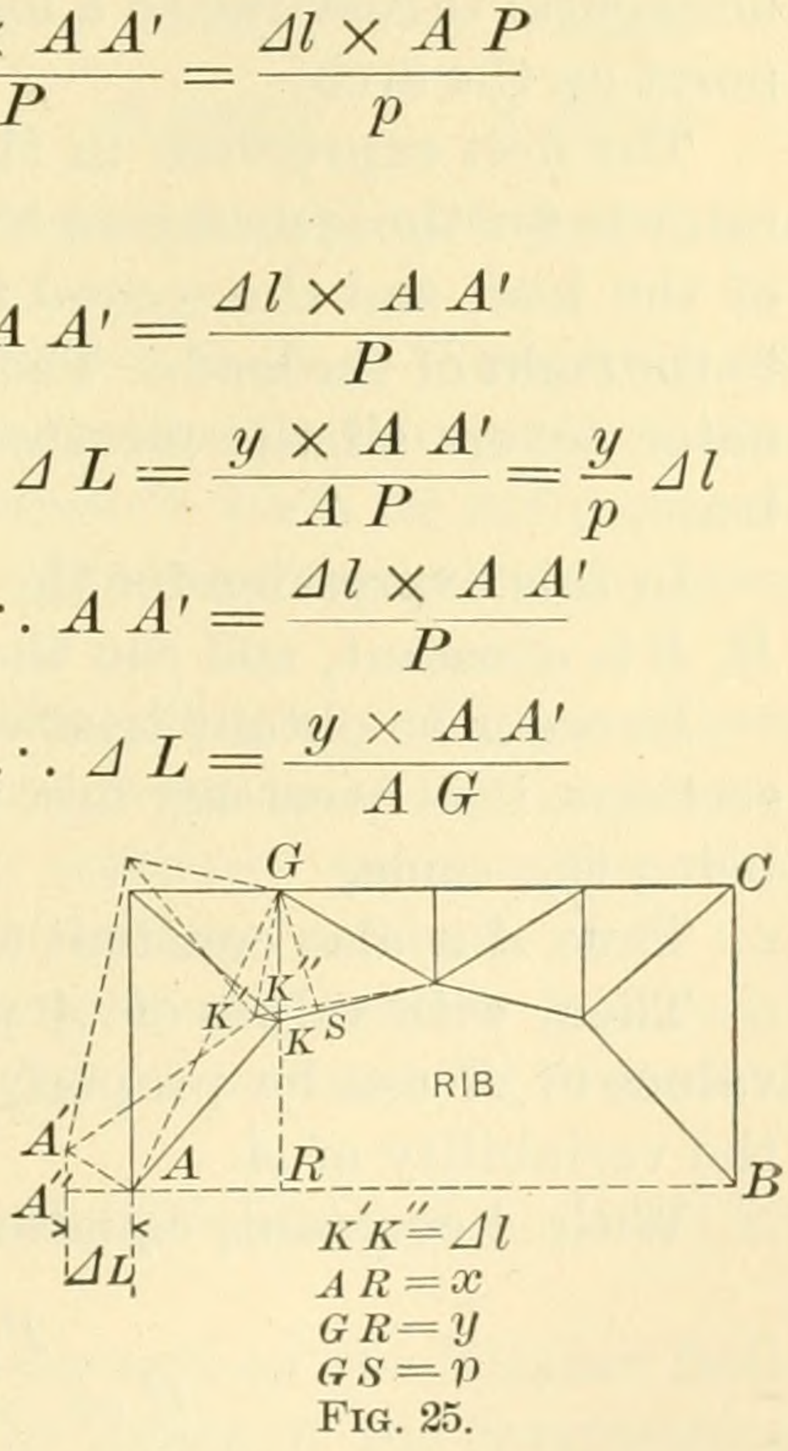

FIG. 25.

If the arch be acted upon by two forces H and V (Fig. 26), by the method of moments, the stress in any member as $G\,F$ can be obtained.

$$T = \frac{H \times P\,R + V \times A\,R}{P\,S} = \frac{H\,y + P\,x}{p} \ldots\ldots\ldots\ldots \quad (4)$$

Mr. R. S. Buck. Substituting this value of T in equation (3), there results

$$\Delta L = \frac{H y^2 + P x y}{p^2} \frac{l}{E A} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Now, this value of ΔL can be calculated for the change of length of each member in the arch, and the error of considering each member as changing its length independently of the changes in the other members is inconsiderable for slight distortions.

The total moment, therefore, is

$$\Sigma \Delta L = H \Sigma \frac{y^2}{p^2} \frac{l}{E A} + P_1 \Sigma \frac{x y}{p^2} \frac{l}{E A} + P_2 \Sigma \frac{x y}{p^2} \frac{l}{E A} \ldots\ldots (6)$$

where P_1 is the vertical end reaction at the left, and P_2 the same at the right.

There is no actual movement of the point A, therefore, $\Sigma \Delta L = 0$, and we have

$$H \Sigma \frac{y^2}{p^2} \frac{l}{E A} + P_1 \Sigma \frac{x y}{p^2} \frac{l}{E A} + P_2 \Sigma \frac{x y}{p^2} \frac{l}{E A} = 0 \ldots\ldots\ldots (7)$$

$$H = \frac{P_1 \Sigma \frac{x y}{p^2} \frac{l}{E A} + P_2 \Sigma \frac{x y}{p^2} \frac{l}{E A}}{\Sigma \frac{y^2}{p^2} \frac{l}{E A}} \ldots\ldots\ldots\ldots\ldots (8)$$

which is the general value for the horizontal thrust due to a load at any point on the arch.

The first expression in the numerator is for those members to the left of the load and the second for those to the right of the load. The denominator covers all the members in the truss.

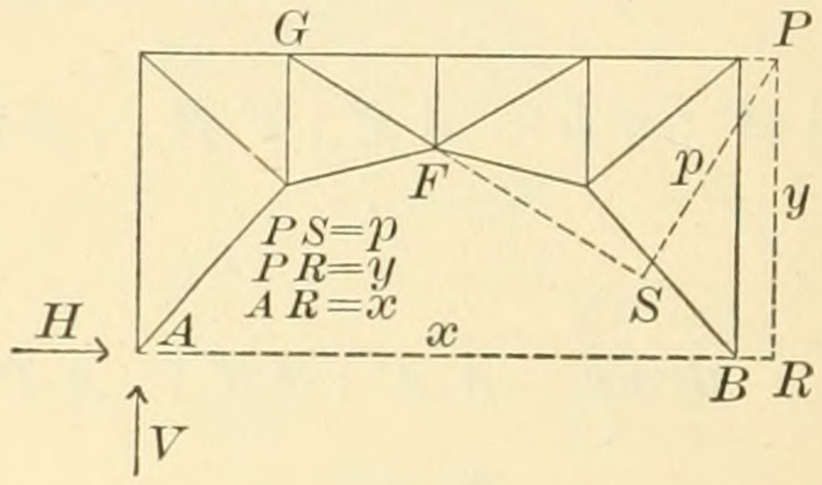

FIG. 26.

In this expression for the value of H, E is constant, and can therefore be cancelled out.

In order to obtain trial values of H and from these to obtain trial sections, it is necessary first to consider the sections of all members as being the same.

Thus A is also constant and cancels out.

Then, with values of A proportionate to the unit load stresses, the values of H can be properly corrected by giving due consideration to the variability of A.

With A constant, equation (8) becomes

$$H = \frac{P_1 \Sigma \frac{x y}{p^2} l + P_2 \Sigma \frac{x y}{p^2} l}{\Sigma \frac{y^2}{p^2} l} \ldots\ldots\ldots\ldots (8^a)$$

As can be readily seen, $\frac{x}{p}$ is simply an expression for the stress in any

member due to a vertical end reaction of unity, and $\frac{y}{p}$ the same for a horizontal thrust of unity. Mr. R. S. Buck.

Therefore, the first steps in the solution of the above expression for H is to ascertian these stresses in all members. These can be obtained by any of the ordinary methods, preferably perhaps by means of moments for the chord members, and graphically for posts, braces and ribs.

A combination of the two methods serves as a very convenient check.

With values of H for load unity on each panel point, the unit stresses of all the members can be obtained.

The values of H for load unity on each panel-point are as follows:

	Preliminary. *A Constant.*	Final. *A Variable.*
Load on 1	.15554	.16459
" " 2	.29824	.31400
" " 3	.43439	.45411
" " 4	.56250	.58499
" " 5	.67596	.70175
" " 6	.77945	.79974
" " 7	.85051	.86702
" " 8	.87614	.88540
(16 Panels Loaded).....	8.39652	8.65840

The results show only a variation of from 1 to 6%, and therefore a second approximation is unnecessary.

There is marked similarity between this method of Clerk-Maxwell and that given by Mr. Braune as taken from the German authorities. Applied to the truss given in Professor Greene's book on arches to illustrate the Clerk-Maxwell method, Mr. Braune's equation (5) gives values of H from 1 to 9% smaller.

Professor Johnson's views concerning the sufficiency of approximate methods in such designs as require so much work to compute are unquestionably sound. His deductions are certainly serviceable. One or two applications, however, of the more tedious methods show numerous short cuts within the limits of theoretically correct treatment that deprive them of much of their capacity for consuming time and grey matter.

Mr. Schaub's desire to launch a stringent criticism rather clouds his observation of facts.

The full amount of dead load was not imposed on the Niagara Railway Arch when the adjustment was made by means of the compression toggle. None of the upper floor system, and only part of the lower floor system, was placed at the time the adjustment was made. In fact, when due consideration was given to temperature conditions, only

Mr. R. S. Buck. 350 tons had to be imposed in both top chords to secure proper adjustment, instead of 850 tons, as imagined by Mr. Schaub.

If Mr. Schaub examines the toggle a little more closely he will find that it is capable of much more push than may at first appear. The bolts are $2\frac{1}{2}$ ins. in diameter, and with ordinary wrenches the nuts could be drawn up to produce a tension of 4 tons in each bolt. This means 16 tons applied to both sides of a double toggle, or 32 tons effective lateral pressure. The angle of inclination of the toggle proper multiplies this by 8, and gives an available pressure of 256 tons for each toggle, or 516 tons for both chords to take care of 350 tons needed.

So far from having practically nothing in the way of dead load stress in the top chord of the center panel, there is a reasonably close approximation to the 350 tons due at the time of adjustment, and the balance due to subsequent loading is taken care of in the regular way.

The openings at the center of the top chords were increased by means of the toggles from $\frac{1}{4}$ in. to 1 in., and the line of camber, according to close level observations, raised over the entire span in a manner to indicate closer conformity to a correct distribution of stress.

The author takes exception to the proposition that the two-hinged arch is incapable of proper adjustment, and insists that such adjustment is not only possible, but easily applicable. He would, however, recommend a more sensitive means of measuring the stress applied at the center, such as was used in the case of the Niagara Falls and Clifton Arch, viz., hydraulic jacks with gauges attached. The device, however, used in the case of the Railway Arch was designed to meet an unexpected contingency, and was not, as in the latter case, carefully designed beforehand.

Mr. Lewinson's proposition that the form of end bearing here adopted is improper, on account of the theoretical center of the end bearing being 4 ft. away from the bearing face of the joint, is readily answered.

Eccentricity and consequent appreciable unfair distribution of the pressure in the rollers are not possible so long as the line of pressure does not depart considerably from the center of the roller bearing. The extreme range of the line of pressure is only 24° 10′, and the usual range not more than a quarter of this; therefore, the rollers are not, as Mr. Lewinson imagines, very seriously abused. No bending can occur at this joint unless the line of pressure actually falls outside of the roller bearing, which is manifestly impossible.

The case there, in effect, is simply that of a pin 8 ft. in diameter with roller bearings.

The rollers are easily accessible and can be cleaned readily, therefore danger of their becoming ineffective from rust is not at all serious. As far as can be seen, there is not the least indication that there is any unfair distribution of stress on the rollers.

Mr. R. S. Buck.

Replying to Mr. Lindenthal's request for further information on certain points, the dead load concentrations and the allowable stresses for the truss have been added to the stress diagram.

The method adopted to secure the deflection under a uniform load of 10 000 lbs. per lineal foot was to compute the shortening of the rib and the consequent reduction of versed sine under this load. This dealt with the rib members only, and gave $2\frac{5}{16}$ ins. as the deflection. By using the "Pull over E" formula, $2\frac{1}{8}$ ins. are obtained as the deflection.

Theoretically, therefore, under a load of 6 500 lbs. per lineal foot, the deflection should be $1\frac{7}{16}$ ins. instead of $1\frac{3}{16}$ in. as actually obtained under this load. The difference, $\frac{5}{8}$ in., is proportionally considerable, but the actual amount of deflection is too small, and existing conditions too far removed from those necessarily assumed in theoretical treatment, to use the deflection as a close check on the stresses. All that can be accomplished is to see whether or not the deflection comes within the theoretical limit.