TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852)

VOL. LXXXV

1922

Edited by the Secretary, under the direction of the Committee on Technical Activities and Publications.

NEW YORK
PUBLISHED BY THE SOCIETY

1922

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1496

THE CONTINUOUS TRUSS BRIDGE OVER THE OHIO RIVER AT SCIOTOVILLE, OHIO, OF THE CHESAPEAKE AND OHIO NORTHERN RAILWAY*

By Gustav Lindenthal,† M. Am. Soc. C. E.

With Discussion by Messrs. C. A. P. Turner, T. Kennard Thomson, Charles Evan Fowler, J. E. Greiner, D. B. Steinman, Henry H. Quimby, and Gustav Lindenthal.

Synopsis

The peculiar construction of the Sciotoville Bridge has been the subject of frequent inquiries, and the following detailed, although somewhat belated, description will serve as a permanent record, useful for similar bridge construction elsewhere.

The distinguishing features of the design are four: Continuous trusses over two long spans; floor-beams, acting as inverted arches and braced against tractive forces; erection with the minimum of falsework and without extra material in the trusses; and riveted connections to the limit of the largest rolling mill and shop facilities. The subject-matter is presented herewith under the following heads:

1.—General Conditions and Selection of Design.
2.—History and Characteristics of Continuous Truss Bridges.
3.—Substructure.
4.—Steel Superstructure.
5.—Fabrication and Erection of Steelwork.

1.—General Conditions and Selection of Design

Location and Grades.—When it was decided by the Chesapeake and Ohio Northern Railway Company to bridge the Ohio River and connect with the

* Presented at the meeting of April 5th, 1922.
† Cons. Engr., New York City.

main line of the Chesapeake and Ohio Railway, on the Kentucky side (the southern or left bank) of the Ohio River, the crossing was selected so as to form a direct route from the junction to Columbus, Ohio, along the valleys of the Little Scioto and the Scioto Rivers, with exceptionally favorable grades for the heavy coal transportation to the Great Lakes *via* the Hocking Valley Railway. Going north, the grades are 0.2% and, going south, 0.3%, and are compensated on curves. The line has a single track for the present, but the Ohio River Bridge and the masonry piers of the approach viaducts are built for two tracks. With the coal traffic steadily growing, two tracks for the entire line will be necessary before long. The double-track bridge was built at a period of low prices, thus enhancing its economic advantage in the development and growth of that important traffic.

River Conditions.—At the site of the bridge, the Ohio River has a width of about 1 600 ft. between embankments and forms a sharp bend. The river channel is near the inner or Kentucky shore, but at high-water stages, the traffic shifts toward the Ohio shore, along which also, at times, considerable ice and drift are carried.

The river channel is 5 ft. deep at low water and 72 ft. deep at high water. The bottom which is practically bare rock, with a slight slope from the Ohio toward the Kentucky shore, afforded a solid foundation for the piers.

The requirements of the War Department called for a minimum clear height under the bridge of 90 ft. above low water and 40 ft. above high water. The navigation interests demanded large openings, owing to the danger by obstructing piers to the descending coal tows some of which are 150 ft. wide and 700 ft. long and (at this sharp bend of the river, at the head of the shoals of the Little Scioto River) are difficult to control, particularly on account of the dense smoke and fog prevalent in this locality. On the Kentucky side, it was also necessary to keep the river channel open for navigation during the erection of the bridge.

Selection of Design.—After several layouts with different span lengths, it appeared that with two spans of 750 ft. each in the clear and a pier in the middle of the river, the requirements of the navigation interests and of the War Department would be satisfied. This made possible a symmetrical and sightly structure with two spans 775 ft., center to center, of bearings. (Fig. 1.)

The rock foundations were favorable to a continuous truss bridge, which also offered the advantage of erection with a minimum of falsework. Two simple truss spans of 775 ft. each, would have been from 15 to 20% more expensive for metal and erection.

The span of 775 ft. exceeds in length the longest existing simple span bridge, namely, the 720-ft. span of the Ohio River Bridge, at Metropolis, Ill., built in 1917.

It will thus be seen, that the selection of continuous trusses was primarily indicated in this case by reasons of economy in metal and by facilities of erection.

2.—History and Characteristics of Continuous Truss Bridges

In view of the fact that this bridge has the longest spans of the continuous truss type, and thus comes into competition, for long spans, particularly with the simple span and the cantilever type, it appears appropriate to review briefly its history and some of its characteristics not generally appreciated by American bridge engineers.

History.—The Britannia Bridge in England, built by Stephenson in 1848, marks a milestone in bridge construction, not only because it was the first important iron bridge of the beam type, but also because it was the first representative of the continuous girder type. It is a tubular plate girder bridge of four spans, two of which are 230 ft. and two of 460 ft. The girders were proportioned as simple beams, but the designer, realizing that continuity increased the carrying capacity, regarded this feature as an additional safety. To gain more information, he made tests with a model, and thus found the points of contraflexure of the elastic line, which he regarded as "fixed" and as dividing the span into two "cantilevers" and a "central beam". This laid the foundation for the later development of the modern cantilever bridge by the introduction of hinges at the points of contraflexure.

Too much credit cannot be given to that galaxy of early English bridge engineers of nearly one hundred years ago—Stephenson, Fairbairn, Telford, Tierney Clark—for the originality and daring of their plans and constructions. They did their own thinking; they did not wait for precedents, but created them. Theirs was the genius that originates as distinguished from routine which merely imitates.

The Britannia Bridge was followed by several similar bridges, among which may be mentioned the Torksey Bridge, built in 1849, over the Trent, with two spans of 130 ft., and the Bryne Bridge, built in 1855, with a central span of 267 ft. and two side spans of 141 ft. each.

In the latter part of the Nineteenth Century, continuous truss bridges were extensively built on the Continent, principally in France, and were usually of the lattice truss type with parallel chords, with from three to five spans. The Fades Viaduct over the Sioule River, in France, built from 1905 to 1908, with a central span of 472 ft., was the longest continuous span previous to the building of the Sciotoville Bridge.

The Lachine Bridge, built in 1888, by the late C. Shaler Smith, M. Am. Soc. C. E., over the St. Lawrence River, near Montreal, Que., Canada, with two side spans of 269 ft., and two middle spans of 408 ft., was, for 29 years, the only continuous bridge in America. It was built as a cantilever and then converted into a continuous truss for the live load. It was replaced in 1910 by a simple span bridge.

There has been always more or less prejudice against continuous trusses, because cantilever trusses offer the alleged more accurate computation of stresses on purely statical principles. No continuous girder as far as known has ever failed in the trusses, whereas the largest and most discreditable bridge failure belongs to the supposedly accurate cantilever type.

It appears now, however, as if the continuous truss would again come into its own, for not only was it adopted for the Sciotoville Bridge, but also for the bridge of the Bessemer and Lake Erie Railroad built in 1918 over the Allegheny River near Pittsburgh, Pa., with spans of from 272 ft. to 520 ft., and for the bridge of the Hudson Bay Railway, across the Nelson River, built in 1918, with spans of 300 ft. and 400 ft.

FIG. 1.—SCIOTOVILLE BRIDGE OVER THE OHIO RIVER.

Characteristics of Continuous Bridges.—The continuous truss type has nowhere met with more indiscriminate and unqualified condemnation than by engineers in the United States, who have alleged three principal objections against it, none of which is novel or decisive:

1st.—That it is statically indeterminate, that is, its reactions and stresses are dependent on the elasticity of its members;

2d.—That it is subject to stresses from unequal settlements of its supports; and,

3d.—That it is affected by temperature changes.

Although these characteristics may be objectionable in certain cases, it can be readily shown that they are entirely unobjectionable in others, and they will be briefly discussed.

Static Indeterminateness

The argument that the stress computations are complicated can be readily dismissed. For preliminary and comparative designs, the simplifying assumption of constant moment of inertia of the vertical section through the girder or truss and the neglect of the elastic deformation of the web members, furnish sufficiently accurate and quick results. For the final design, the slight additional time and labor involved in the accurate calculation is of no account as continuous bridges will never be a common type.

The stress calculation, of course, is the more complicated the greater the number of spans, but, for other reasons, it is not advisable to have more than three or four spans in a continuous bridge.

The fact that the stresses are, to a certain extent, dependent on the elastic deformation of the members, is characteristic of all statically indeterminate structures, but in a properly designed continuous bridge the effect of even large variations in the elastic behavior is insignificant when compared with other uncertain stresses, such as secondary stresses, etc., which exist also in statically determinate structures.

Variation between the actual and the calculated elastic deformation may be due, first, to a difference between the assumed and the actual modulus of elasticity; and, second, to the influence of the details, such as gussets, splice and tie-plates, rivets, etc., which can only be roughly estimated.

A proportional change in the modulus of elasticity or in the sectional area of all members due to allowance for details has no influence on the reactions and stresses, but only on the deflections which change in the same proportion with the moduli.

The effect of non-proportional changes is well illustrated by a comparison of two sets of calculations made for the Sciotoville Bridge. It was planned that, during the various stages of erection, the trusses would pass successively through static conditions of a beam on two, three, four, and five supports (see Plate XII). The calculation of the reactions, for the purpose of determining the necessary jacking forces at the temporary supports, and of the deflections, for determining the jacking heights, were made, first, on the basis of the gross area of the main section of the members without allowance for details and, second, on the basis of gross area of the main section plus the area of a section equivalent to 75% of the weight of the details. This additional area, due to details, varies for different members from 5 to 25% and averages about 20% of the gross area of the main section. For these two extreme assumptions, certain deflections differ by 25 to 60%, whereas the reactions differ by not more than 0.2% in the two-span condition, 465 ft.-775 ft., 10% in the three-span condition, 155 ft.-465 ft.-775 ft., 60% in the four-span condition, 155 ft.-155 ft.-465 ft.-775 ft., 11% in the three-span condition, 310 ft.-465 ft.-775 ft., and 0.7% in the final two-span condition, 775 ft.-775 ft.

The large differences in the reactions in the three and four-span conditions are due principally to the great variation in the length of the spans and to the comparatively great height of the short spans. Continuous trusses of great height and greatly different span lengths are, therefore, rightly objectionable, the more so, because they are also sensitive to settlements of the supports and to temperature changes. During erection, such a condition is of no consequence, because the reactions can be measured and the height of the bearings promptly adjusted if necessary. Where the spans are more nearly equal, and the trusses not unusually high in comparison with the span length, the effect on the reactions and stresses from a variation in the elasticity of the trusses is practically insignificant.

As a precaution, and to obviate any uncertainty of stress action, from dead load at least, it is always possible and advisable, as was done in the case of the Sciotoville Bridge, to measure certain reactions, and, if necessary, to adjust the height of the bearings until the reactions are correct.

In no case should the variation in span length in a continuous truss be so great that the live load on any span will cause a reversal of the dead load reactions of the adjoining span. For all these reasons, continuous trusses over several spans and on metallic towers, or on steel arches, should preferably be shallow in depth, a rule first practiced by French engineers.

It is interesting to note that the actual deflections of the Sciotoville trusses, as observed in the field, were nearly midway between the values computed under the two previously mentioned assumptions; in other words, an average addition of about 10% to the gross areas of the sections, to allow for details, should be made when calculating elastic deformations.

Effect of Settlements or Compressibility of Supports

Where considerable settlements of the foundations are to be anticipated, or where the supports are high elastic towers, the continuous type of bridge is not advisable, unless care is taken to eliminate the effect of inconstant levels by means of adjustable bearings.

The stresses caused by the ordinary compressibility of the supports can be computed and, if they are not unduly large, can be neglected or provision can be made in the sections.

Settlements of the foundations are less objectionable the longer the spans, as already mentioned. For similar continuous trusses with equal proportion of height to span length, the same settlement of a support causes stresses approximately in inverse proportion to the span length. As settlements may be assumed as proportional to the foundation pressure and as the latter is about the same for long and short spans, under the same soil conditions, it follows that, in general, the danger of excessive stresses due to settlements is less the greater the span lengths. In special cases, it may be advisable to design the permanent details of the truss bearings on the piers and abutments so that they may be raised or lowered by hydraulic jacks at any time, as needed to maintain the original levels.

In the case of the Sciotoville Bridge, a settlement of 2 in. in one of the end piers, if it was possible, or a settlement of 1 in. in the middle pier, the others remaining undisturbed, would change the reactions by only 0.6 per cent. It is evident that even a considerably greater settlement, which would seriously disturb the vertical alignment of the track, would not objectionably affect the stress condition in the trusses. When the Kentucky end of the bridge was raised to its final position, after erection had been completed, difference in the jacking force for the last 3 in. of jacking was noticeable. This shows that for spans of this length continuous trusses are unobjectionable, even if the foundations do not rest on solid rock.

As already mentioned, the first continuous trusses had solid webs (Britannia Tunnel Bridge) and on the Continent small mesh lattice webs. The secondary stresses in the web are of no importance in the first type and, in the latter type, they may be considered rather beneficial, since they contribute to stiffness and absorb some of the bending stresses on the trusses, proven by the fact that the lattice can bear some load when the chords are cut away, only the chord sections over the bearings may need reinforcement against bending. In some instances, the continuous small mesh web girders over several spans were erected on falsework with a camber of $\frac{1}{500}$ to $\frac{1}{300}$ of the entire length of bridge and then let down on the pier bearings, to produce in the trusses initial bending stresses opposite those from live load. Such was the procedure of erection of the continuous truss viaduct of five spans on high iron towers over the Thur at Ossingen, Switzerland, in 1873. The writer was on the Engineering Staff for that structure, for which the statical computations were considered quite a feat at that time.

Temperature Effects

Temperature stresses in continuous trusses may be caused, first, by the expansion or contraction of intermediate supports, particularly high steel towers, and, second, by unequal temperature changes in different parts of the trusses themselves (uniform temperature changes in all members cause no stresses).

The first effect is similar to that of settlement of the piers and is greater, the higher the intermediate supports and the shorter the spans. In some existing bridges, it amounts to as much as 25% of the stresses from dead and live load. It is, however, insignificant and negligible in the case of long spans resting on shallow piers. As the stresses can be calculated, it is easy to make the necessary provision in the sections.

The second effect is neglected, as a rule, although in a case where the bottom chords are protected from the direct rays of the sun by a solid floor, the temperatures in the top and bottom chords may differ considerably. The effect of such a difference is similar to that of a variation in the elasticity of the truss members and may be serious in a case where the lengths of spans vary considerably.

In the case of the Sciotoville Bridge an average difference in temperature of 10° Fahr., between the top and bottom chord (the effect of the web members is insignificant), would change the end reactions by only 1.5%, which is negligible.

Summing up, one arrives at the well established conclusion that continuous trusses are generally unobjectionable under the following conditions:

1st.—Where the lengths of the spans do not vary greatly, and the trusses are not unusually high as compared with the span length.

2d.—Where the foundations rest on fairly solid ground, compressibility of the soil being less objectionable the longer the spans.

3d.—Where the intermediate supports are not excessively high in comparison with the span length.

In every continuous truss there are a few members, such as the chords near the points of contraflexure, or web members near points of maximum moment, which are most sensitive to the effects enumerated, because their sections, as required by the dead and live load stresses, are comparatively small. Such members, as a rule, will require for fabrication or erection purposes a section somewhat in excess of that required by the stresses, to provide a certain margin for possible variation. It is advisable always to investigate such members and proportion them so that they are strong enough under reasonably extreme assumptions.

Advantages of Continuous Trusses.—Against the disadvantages mentioned, as far as they can be classed as such in any given case, must be weighed the advantages which the continuous type possesses over the simple span or the cantilever.

As regards economy, it is not feasible to make a general comparison between the continuous bridge and the cantilever since that depends largely on the arrangement of the spans, which is usually governed by local conditions and of necessity must be different for the two types, owing to their different character. In comparison with a series of simple spans of the same length, however, the continuous truss shows a decided economy which is greater the longer the spans and, up to a certain limit, greater the number of spans. For long spans, the saving in cost may be as much as 25 per cent. For short spans, the economy over simple spans is not important, but greater rigidity under passing loads is an advantage.

In point of rigidity, as measured by the deflections, the continuous truss compares favorably with the simple span and shows a decided advantage over the cantilever. Its deflections and amplitude of vibrations are smaller, and the elastic line smoother and devoid of local kinks such as occur at the hinges of cantilevers. Both rigidity and economy gain from the fact that


DETAILS OF RIVER PIER
NO. 16
TOP VIEW
Base of Rail Elev. + 590.0
Elev. + 571.23
38′ 9″ c. to c. Trusses
56′ 0″ Length of I Beams
13′ 0″ c. to c. Tracks
Rad.
Center Line of Pier & Railroad
Symmetrical abt. C. L.
Batter 1:16
Elev. + 474.5
ELEVATION
SECTION ON CENTER LINE
1″ ◻ Hor. Rods 4′ 0″ apart
1″ ◻ Hor. Rods 2′ 0″ apart
1″ ◻ Vert. Rods
1″ ◻ Vert. Rods 2′ 0″ apart
1″ ◻ Hor. Rods 4′ 0″ apart
Alternate Vertical Rods stop here
Others run to bottom of foundations
21 @ 4′ 0″ = 84′ 0″
Rods 1″
SECTION C-C
1″ ϕ Rods 2′ 0″ Staggered
1″ ◻ Rods 2′ 0″ apart
20 I 80 lb. x 56′ 0″
Symmetrical abt. C. L.
Batter 1:16
DETAIL OF TOP AND COPING
14 - 1″ ◻ Vert. Rods spaced equally
1″ ◻ Hor. Rods 2′ 0″ apart
1″ ◻ Vert. Rods
1″ ◻ Hor. Rods 4′ 0″ apart
1″ ◻ Hor. Rods 2′ 0″ apart
1″ ◻ Vert. Rods spaced equally
7 - 1″ ◻ Vert. Rods spaced equally
22 spaces @ 2′ 0″ = 44′ 0″
SECTION D-D

Fig. 2.

truss members subject to reversion of stress (tension and compression) can be riveted members proportioned only for the larger stress.

As is characteristic of all statically indeterminate structures, the continuous truss has the advantage in that when a member is seriously weakened, as may happen in the case of the derailment of a train and collision with the trusses, the probability of failure of the whole bridge is less than in the case of the statically determinate simple span or a cantilever. This greater safety is still more pronounced when the truss webs consist of small mesh lattice.

The continuous bridge presents no greater difficulties in erection than the cantilever, but it offers the advantage over the simple span in that it can be erected on the cantilever principle without, or with little, additional material. This was one of the governing factors in the case of the Sciotoville Bridge.

From the esthetic point of view, the continuous bridge can well compete with the simple span or cantilever, if properly designed, but not with the more artistic arch or suspension bridge.

3.—Substructure

Foundations.—All the piers rest on solid shale rock which has nearly vertical stratification. The maximum foundation pressure is 9.5 tons per sq. ft. from vertical loads only and 12.5 tons per sq. ft., with longitudinal force acting.

The foundations presented no unusual problems, except that the center pier required a very heavy coffer-dam to obviate the danger of the coffer-dam being washed away by flood, as it had to rest on bare rock bottom.

Piers.—The piers are of concrete, reinforced with steel rods to prevent shrinkage and temperature cracks. Both the up and down-stream ends of the piers are semi-circular. The tops are marked by octagonally shaped massive copings which give a neat appearance at no additional expense.

The surfaces are plain, having been wetted and rubbed to a smooth finish with concrete bricks immediately after the forms were removed. The top surfaces were troweled to a smooth finish before the concrete had set and, after hardening, received two coats of cement filler.

The center pier which resists the longitudinal force from the entire bridge and carries a vertical load of 16 400 tons, is 18 by 63 ft. under the coping and has a batter all around of 1: 10. The shore piers which carry only vertical loads of 5 100 tons, are 12 by 57 ft. under the coping and are battered 1: 20.

The concrete is mixed in the proportion of 1 part Lehigh Portland cement, 2 parts sand, and 4 parts gravel. Sand and gravel of excellent quality were obtained from a river bank a few miles below the bridge site. The reinforcement consists of 1-in. square steel rods, arranged as shown on Fig. 2. The copings are reinforced longitudinally by 20-in. by 80-lb. beams, spaced 2 ft. apart, and running the full length of the pier. They serve also to distribute the load from the bearings.

The pier on the Ohio shore intersects the ground surface at mid-height of the embankment, about 30 ft. above low-water level. The foundation was placed during low water by dry, open excavation through about 10 ft. of clay and 8 ft. of loose disintegrated shale rock. This pier was started in November, 1914, and completed in March, 1915.

The pier on the Kentucky shore is near the bottom of the embankment, where the ground surface is only about 15 ft. above low-water level and the soil is mostly sand. These conditions made it desirable to sink the pier as a concrete caisson with a steel cutting edge, a working chamber 6 ft. high, and four shafts 7 ft. 3 in. in diameter. It was not necessary, however, to resort to air pressure. All excavation down to the solid rock, about 15 ft. below low water, was done by open dredging with orange-peel buckets while the caisson was flooded. About 2 ft. of disintegrated rock was removed by hand, for which purpose the caisson was kept dry by pumping, having been carefully sealed with empty cement bags rolled up behind the cutting edge. As there was considerable seepage through the rock bottom, drain sumps were provided from which the water was pumped during the concreting of the working chamber. These sumps were finally filled by forcing cement grout down through pipes by compressed air. The cutting edge was set on June 22d, and the pier completed on September 14th, 1915.

At the middle or river pier, the rock surface is practically level and barely exposed at low water. The rock was excavated to a depth of 10 ft. to give the pier a firm hold against dislocation under possible ice pressure. This depth also proved necessary for the removal of all disintegrated and seamy rock.

Excavation and concreting were carried on within a wooden coffer-dam set on the rock bottom. This coffer-dam was 16 ft. high, 81 ft. wide, and 129 ft. long, and had double walls 10 ft. apart. The space between the walls was filled with dirt dredged from the river channel. The top was capped with 2-in. planking to prevent the washing out of the fill. Beams 8 ft. high were placed outside and inside, along the dam. Although submerged for four months during the high-water season, the coffer-dam remained intact.

Construction of the coffer-dam was started in November, 1914, and the pier was completed in May, 1915, after an interruption of four months in the foundation work during high water.

The three piers contain approximately 15 000 cu. yd. of concrete and 250 tons of steel reinforcement and cost $165 000, or $11 per cu. yd. They were built on contract by the Dravo Contracting Company, of Pittsburgh, Pa., with an average daily force of 105 men.

4.—Steel Superstructure

Rigidity was one of the main considerations in working out the design of the steel superstructure. With a few modifications, the rules of design were the same as those for the Hell Gate Arch Bridge and Approaches, and are contained in detail, with explanatory remarks, in the paper by O. H. Ammann, M. Am. Soc. C. E., on that bridge.* They are, therefore, only briefly quoted here insofar as they apply specifically to the Sciotoville Bridge.

* *Transactions,* Am. Soc. C. E., Vol. LXXXII (1918), p. 852.

Loads and Stresses.—All stresses except those from wind and lateral forces were computed by the accurate theory of elasticity, considering the elastic deformation of all members after their section had been determined. The stresses from the wind and lateral force were computed by assuming a constant moment of inertia of the lateral truss. The assumed loads were as follows:

The dead load (D) was carefully calculated for each panel point after the general details had been fully worked out. A re-calculation of the weight from the shop drawings showed the assumed weight to be 3% in excess and, therefore, no re-calculation was made. The average dead load is 18 200 lb. per lin. ft. of bridge, which includes 1 400 lb. per lin. ft. assumed for the two tracks.

The live load (L) on each track was as shown on Fig. 3 (Cooper's E-60 loading).

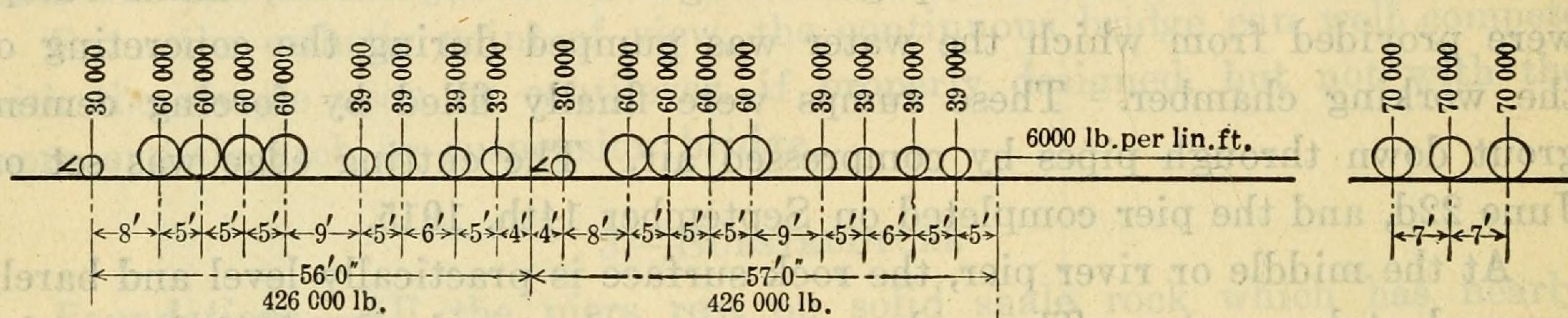


DIAGRAM OF SPECIFIED LIVE LOAD

Fig. 3.

In cases where the live load had to be separated, in order to produce maximum stress, the engine load was placed on the stretch producing the greater portion of the stress and a uniform load of 6 000 lb. per ft. of track on the stretch contributing the smaller portion.

The impact (I) was calculated according to the writer's formula, as follows:

$$I = \frac{L}{D + L} \times \frac{1\,200 + \frac{a}{n}}{600 + 4\,a}$$

in which,

D = dead load stress, in pounds.

L = live load stress, in pounds.

a = length of train behind locomotive tender for position of maximum stress, in feet.

n = number of tracks loaded for maximum stress.

The lateral force (Lat.) from trains was 10% of the live load on one track, or Cooper's E-6 loading acting horizontally 5 ft. above the rail.

The wind pressure (W) was 800 lb. per lin. ft., stationary along the top chord, 700 lb. per lin. ft., stationary along the bottom chord, and 500 lb. per lin. ft., moving 7 ft. above the rail.

The longitudinal force (Br.) from braking or traction on one track only was 60 000 lb. on a wheel-base of 15 ft. for each locomotive, or 1 000 lb. per lin. ft. of track for the whole train.

The total stress was as follows: Members with dead and live load stress were proportioned for a total stress of $D + L + I +$ Lat. + Excess, in which

Excess = (W and Br.) — 20% ($D + L + I +$ Lat.). Members free from dead or live load stress were proportioned for a total stress of W + Br. + Lat.

The permissible unit stresses, in pounds per square inch, were as follows:

Tension, net section	20 000
Compression, net section	20 000
Gross section, $20\,000 - 100\left(\frac{l}{r} - 20\right)$ rounded up to the nearest 500 lb., in which l = the length and r = the radius of gyration of the whole section.	
For latticed members, a further deduction of $100\left(\frac{l_1}{r_1} - 20\right)$ was made, in which l_1 = the length and r_1 = the radius of gyration of the unsupported portion of the section between lattice connections.	
Bending on rolled shapes, girders, and steel castings, net section	20 000
Bending on pins	30 000
Shearing on webs, shop rivets, and pins	15 000
Shearing on field rivets and bolts	12 000
Bearing on shop rivets	25 000
Bearing on field rivets, turning-bolts and pins	20 000
Pressure on concrete masonry	600

The specification for proportioning compression members differs somewhat from that used in the Hell Gate Bridge. The specified compression stress is applied to the gross instead of the net area of the section with the provision, however, that the stress applied to the net area shall not exceed 20 000 lb. per sq. in. The net area usually governed for members with $\frac{l}{r}$ less than 50.

Further, instead of different sets of unit stresses for different types of sections (closed section and sections with one or two open sides), the same basic stress was used for all sections, subject, however, to the previously stated deduction for latticed members.

General Proportions.—The steel superstructure, as built, is a double-track, two-span, continuous bridge, with a total length of 1 550 ft. between centers of end piers, or two equal spans of 775 ft. and two clear openings of 750 ft. at low-water level.

To obtain ample lateral rigidity, the width between centers of trusses was made 38 ft. 9 in., or one-twentieth of the span length, although at first a somewhat smaller width was considered sufficient, in view of the continuity of the lateral truss. The height at the middle of each span is 103 ft. 4 in. between centers of chords. To fix this height, the portion of the span between the end pier and the point of contraflexure was considered as a simple span. This portion varies from three-fourths of the span length for uniform load on both spans to seven-eighths of the span length for uniform load on one span only and averages 630 ft. in length. The height of the truss was chosen approximately one-sixth of this length.

The height over the center pier is 129 ft. 2 in., or one-sixth of the span length; this is the proper height for a simple span of the same length, which has the same maximum moment at the center as the two-span continuous bridge over the middle support.

The height at the end was made 77 ft. 6 in., or equal to a double panel, in order to give the end posts an inclination of not less than 45 degrees. These heights also secured a pleasing outline for the top chord. The web system is of the Warren type, with subdivided panels of a uniform length of 38 ft. 9 in., which was found to be the most economical.

Truss Members and Connections.—Two preliminary designs of the trusses were made, one with tension members built up of pin-connected, 16-in. eye-bars and one with riveted members and riveted connections throughout. In the first design, all riveted members had riveted connections as most of them had to be designed for reversal of stress and no pin connections were allowed for such members. This design was unusual, in that the eye-bars in the heavier tension members were arranged in two chains, one above the other, each consisting of two sets of bars corresponding to the two webs or gussets of the riveted members.

The principal panel points were carefully detailed for both designs, and it was found that they were feasible, although requiring unusual connections and gussets of the largest practicable sizes.

Bids were asked on both designs. The eye-bar design, although about 200 tons lighter, proved to be only slightly cheaper, according to the lowest bid. The riveted truss design was finally adopted, in view of its superior rigidity, durability, and safety. The Sciotoville Bridge is, therefore, the largest truss bridge with completely riveted connections.

The advantages of pin-connected bridges, namely, cheaper fabrication and quicker erection, are not as important to-day as they were formerly. With the present improved facilities for punching, drilling, riveting, etc., the cost of manufacture and erection of riveted work has been greatly decreased, and the time of erection is a less important factor than in the pioneer days of rapid railroad construction.

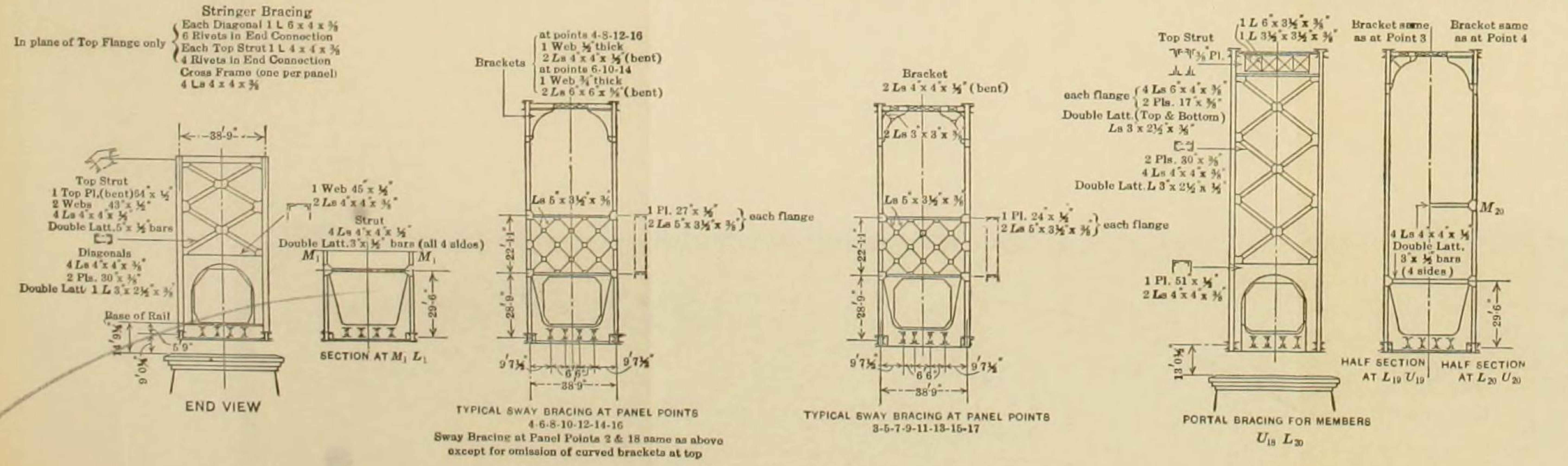
Typical sections of the truss members are shown on the stress sheet (Plate XI). All members have double webs and all chords and main diagonals have inside and outside flange angles. Both top and bottom chords and the inclined end posts have one solid cover-plate on top. The bottom flange angles of the same web are connected by a flange plate which distributes the stress from the latticing to both angles. The inclined posts at the center pier have a solid cover, top and bottom, and thus form completely closed boxes.

All the open sides of the members have exceptionally strong latticing, ranging from 3 by 2½-in. by ⅜-in. angles, with two rivets, to 12-in. by 30-lb. channels with six rivets. For compression members, the latticing has been proportioned for a transverse shearing force, in pounds, equal to three hundred times the gross area of the member, in square inches. The members are stiffened against distortion by transverse diaphragms about 15 ft. apart.

The gusset-plates are in the plane of the web-plates of the members, the latter being cut at the edge of and spliced to the gussets in such a manner that the rivets are in double shear. The flange angles extend in all cases over the gussets.

Note: All Top and Bottom chord Members. End Posts and Diagonals to have transverse Diaphragms about 15' apart.
Gusset Pls. at all Main Panel Points 1⅝" thick except at U_{18} & L_{20}
Gusset Pls. at all Secondary Panel Points 13/16" thick.
Gusset Pls. at U_{18} & L_{20} 3⅛" thick.

Material for Diaphragms
½" Web
4 x 4 x ½ Ls



Assumed Dead Load Panel Concentrations per Truss in Units of 1000 lb.

Panel Points	Sum of Concentrations for Half Truss	20	19	18	17	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
Steelwork at Upper Chord	2021.3	63.5	111	300	53.5	61.9	53	185	68.4	80	68.4	159.5	83.7	96.4	84	158.3	77.5	90	78	149.2	0	0
Steelwork at Middle Points	1154.5	11	219.8	0	148.5	24.4	132.3	0	102.6	22.9	89.6	0	54.3	22.6	49.8	0	59.5	21.2	66	0	130	0
Steelwork at Lower Chord	3552.2	176	153.2	199	158	251.7	132.7	168	133	225.1	157	196.5	157	200	157.2	196.7	157	187.8	126	148.8	125	146.5
Total Steelwork	6728	250.3	484	499	360	332	318	353	304	328	315	356	295	319	291	355	294	299	270	298	255	146.5
Flooring	540	13.5	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	27	13.5
Total Steelwork and Flooring	7268	264	511	526	387	365	345	380	331	355	342	383	322	346	318	382	321	326	297	325	282	160
Bearings																						

General Notes:
All Stresses are given in Units of 1000 lb.
D. denotes Dead Load
L. " Live Load
I. " Impact ———— $I = \frac{L^2}{D+L} \times \frac{1200+\frac{a}{n}}{600+4a}$
Lat. " Lateral force
W. " Wind
Br. " Braking
Excess = (W + Br.) − 20% (D + L + I + Lat.)

a = length in ft. of train behind Locomotive tender for position causing max. Stress
n = number of tracks

All field connections of stringers to floorbeams shall not be riveted until span is completely erected and self supporting

Size of Rivets

Trusses:
- All main members { shop = 1" dia. / field = 1" dia. } except at Points U_{18} L_{20} = 1⅛" dia.
- Sub diagonals, main & sub hangers { shop = ⅞" dia. / field = 1" " }
- Long secondary posts { shop = ⅞" dia. / field = ⅞" " } except in connection for Bott. Chord Gussets = 1" dia.
- For balance of truss members & bracing { shop = ⅞" / field = ⅞" }

Floorsystem { shop = ⅞" dia. / field = ⅞" dia. } except for the end connections of Floorbeams and stringers = 1" dia.

Computation of Stresses

All stresses determined by assuming main trusses and bottom lateral truss as continuous over three supports and top lateral trusses as simple spans between U_2 and U_{18}

STRESS AND SECTION SHEET


DETAILS OF PANEL POINT L_0
END FLOORBEAM
Same for Kentucky and Ohio Ends
14'-8⅜" (Kentucky End)
14'-9¼" (Ohio End)
Elev. 575.25
Elev. 590.02 Ky.
Elev. 590.08 Ohio
C.L. of Truss
C.L. Stringer
Base of Rail
C.L. Stringer
C.L. of Bridge
9'-7½"
19'-4½"
6'-6"
3'-3"
7'-0"
9'-0"
2½"
3"
4 Flange Ls 8"x 8"x 1"
1 Web Pl. 88¼"x ¾"
8 Cover Pls. 17"x ¾"
4 Pls. 14½"x 2"
8 " 12"x 2"
1 4"
5'6¼" b. to b. Sr.
6¼"
5¼" Ohio
4⅝" Kentucky
7'-0¼"
7'4½" b. to b. Fb.
8'10¾"
5'-0"
1'8½"
1'8"
2'0"
1'4"
6" Φ Pin
L_0
Pier No. 15 Kentucky
Pier No. 17 Ohio
4'-4"
6'-6"
Anchor Bolts 1¾" Φ
Elev. 575.25
Bottom Chord L_0 L_1
1 Cover Pl. 48"x ⅝"
2 Flange Pls. 14"x ⅝"
4 Webs 48"x 13/16"
2 Side Pls 16"x 13/16"
8 Ls 8"x 6"x 13/16"
Lacing Channels 12"-25 lb
Diaphragms
4"x 4"x ½" Ls
Web Pls. ½"
4'0½"
End Post L_0 L_1
1 Cover Pl. 48"x ⅝"
2 Flange Pls. 16"x 9/16"
4 Web " 48"x 13/16"
2 Side Pls. " 32"x 13/16"
2 " " 32"x ½"
6 Ls 8"x 6"x 13/16"
2 Ls 8"x 8"x 13/16"
Lacing Channels 12"-30 lb
Diaphragms
4"x 4"x ½" Ls
Webs ½"

Fig. 4.

The gusset-plates at the main panel points are 1⅝ in. thick, except at panel points, U_{18} and L_{20}, where they have a maximum thickness of 3¼ in. The gussets at the secondary panel points are 13/16 in. thick. Correspondingly,

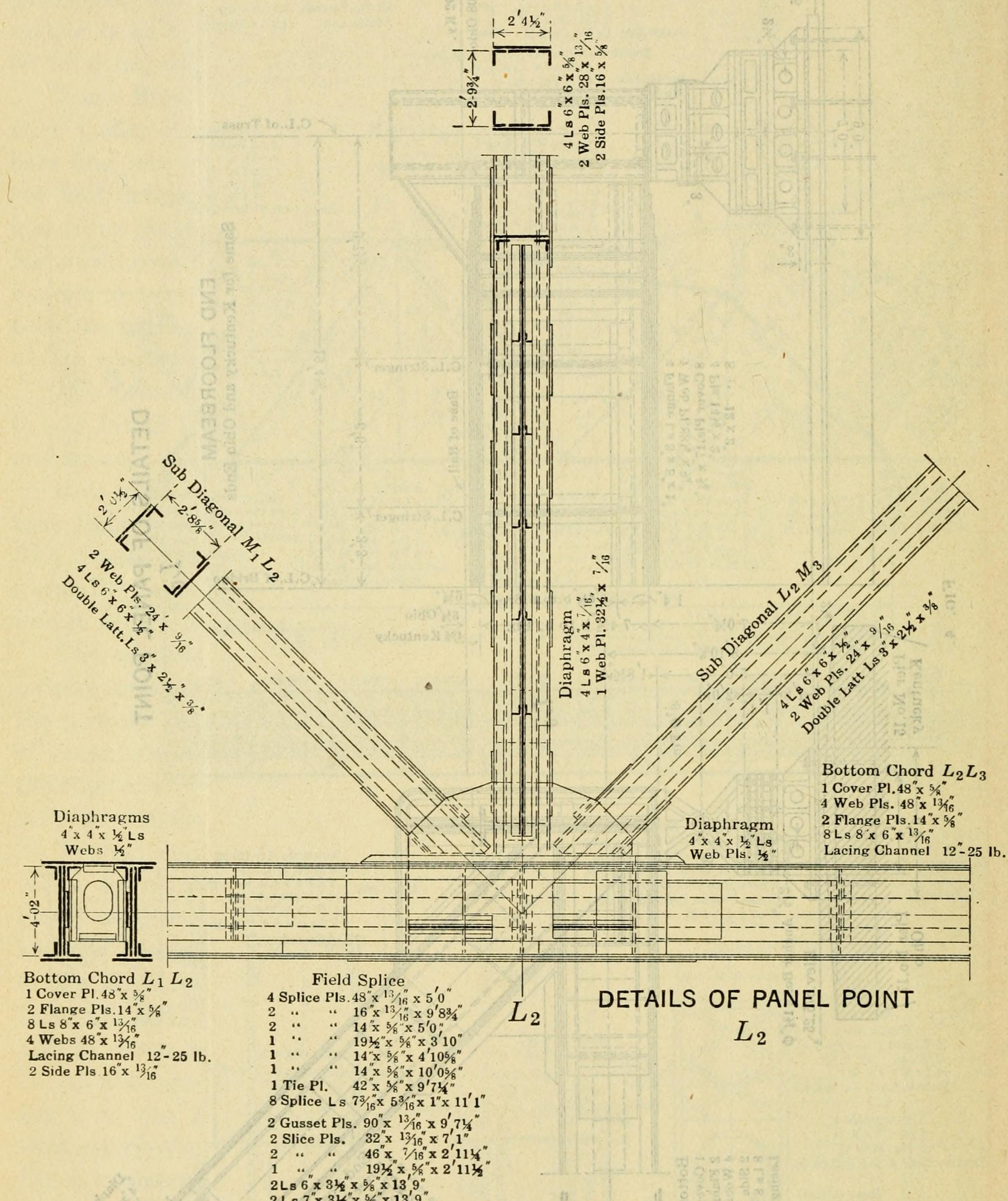


FIG. 5.

the webs of the main members are 13/16, 1⅝, and 3¼ in. thick, all being made up of individual plates, 13/16 in. thick. This uniformity greatly facilitated the detailing of the connections and the avoiding of fillers.

As the full thickness of the gussets was not required for the full size, at some of the largest connections the gussets were split into two to four plates, $\frac{13}{16}$ in. thick, of variable size, in order to save weight. The largest gusset-plates used are 135 in. by $1\frac{5}{8}$ in. by 14 ft. 9 in., and 140 in. by $\frac{13}{16}$ in. by 18 ft. 2 in.

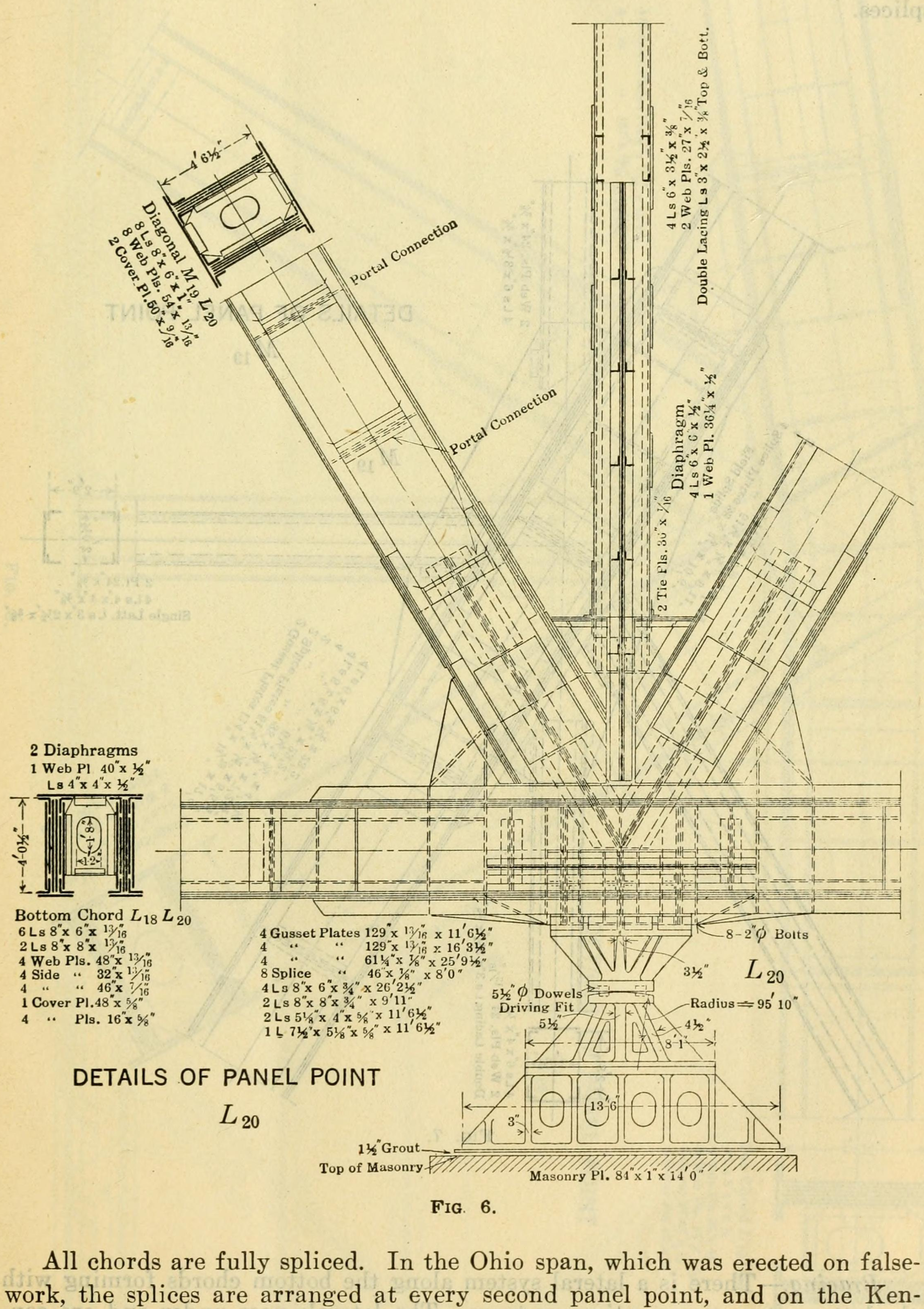


FIG. 6.

All chords are fully spliced. In the Ohio span, which was erected on false-work, the splices are arranged at every second panel point, and on the Kentucky side at every panel point for convenience in the cantilever erection.

All shop and field rivets of the main truss members are 1 in. in diameter, except at panel point, U_{18} and L_{20}, where they are $1\frac{1}{4}$ in. in diameter and up to $7\frac{5}{8}$ in. long between heads. The secondary members have $\frac{7}{8}$-in. shop and 1-in. field rivets. Figs. 4, 5, 6, 7, and 8 show typical connections and splices.

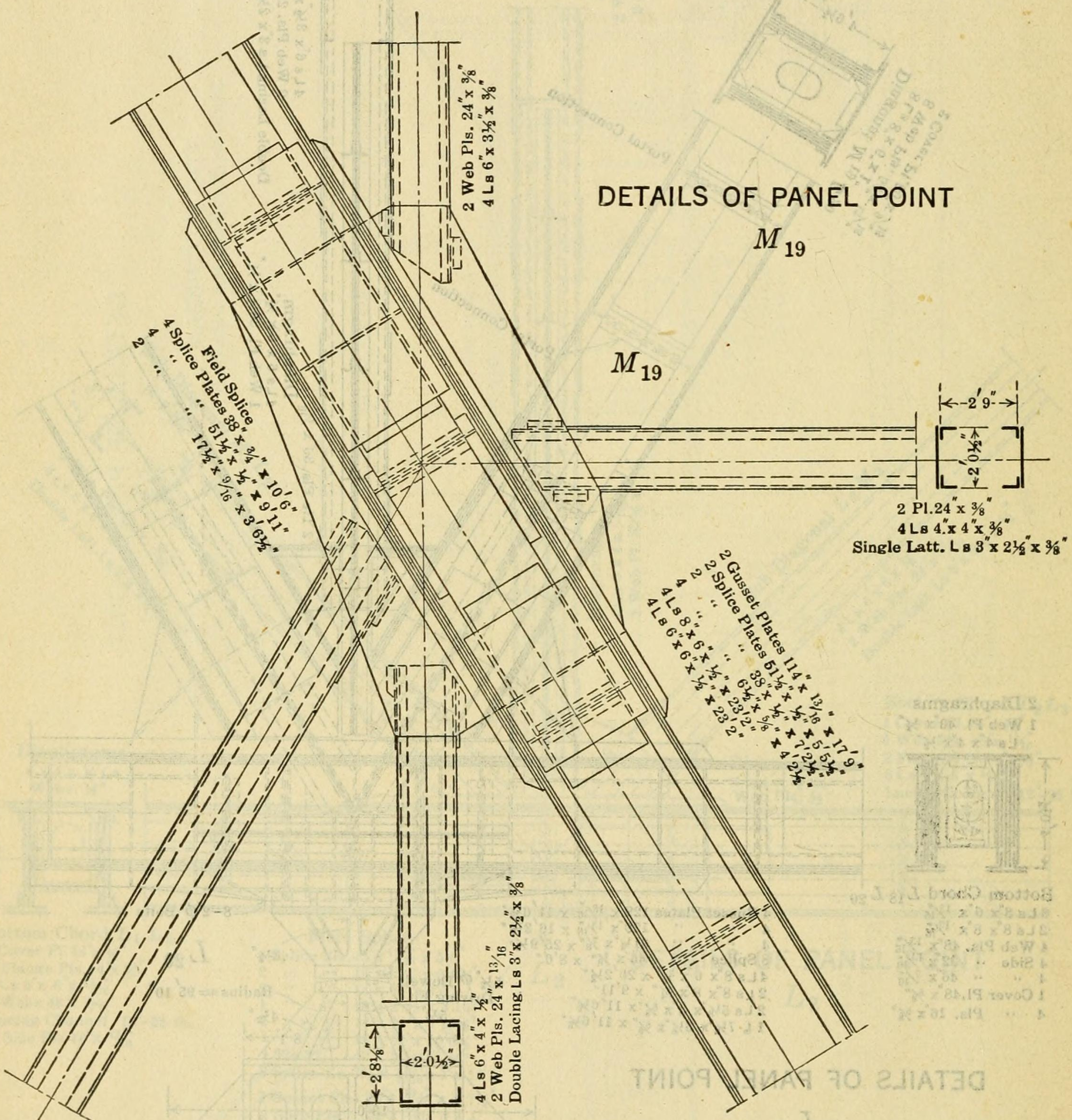


FIG. 7.

Bracing.—There is a lateral system along the bottom chords forming with the latter a two-span continuous truss. The laterals pass under and are connected to the bottom flange of the stringers.

DETAILS OF PANEL POINT U_{18}
4 Gusset Plates 114¼" x 13/16" x 13'4"
4 " " 137" x 13/16" x 17'6½"
U_{18}
Top Chord $U_{18}U_{20}$
8 Ls 8" x 6" x ¾"
4 Side Pls. 38" x ¾"
8 Web Pls. 54" x 13/16"
1 Cover Pl. 50" x ⅝"
2 Cover Pls. 16" x ⅝"
4'-6½"
Diagonal $U_{18}M_{19}$
8 Ls 8" x 6" x ¾"
8 Web Pls. 54" x 13/16"
2 Cover Pls. 50" x 9/16"
4'-6½"
Diaphragm "Z"
1 Web Pl. 54" x ⅜" x 3'2"
4 Ls 4" x 4" x ⅜" x 1'6"
4 Ls 4" x 4" x ⅜" x 2'6¾"

FIG. 8.


19' 4½"
3'3"
6'6"
9'7½"
19' 4½"
4-Ls-6"x 4"x 3/8"
Double Latt. Ls-3"x 2½"x 3/8"
4-Ls-7"x 3½"x ½"
1-Pl.-15"x 7/16"
1-L-3½"x 3½"x 3/8"
1-Pl.-18"x 7/16"x 2'1"
1-L-3½"x 3½"x 3/8"
U14
L14
Symm.abt.C.L.
4-Ls-6"x 6"x 7/16"
2-Pls.-36"x 7/16"
Double Latt. Ls 3"x 2½"x 3/8"
4-Ls-7"x 3½"x 9/16"
1-Pl.-15"x 9/16"
4-Ls-6"x 4"x 3/8"
Double Latt. Ls-3"x 2½"x 3/8"
2-Pls.-43"x 7/16"
TYPICAL DETAILS OF TOP AND BOTTOM LATERALS
HALF PLAN STRINGERS, FLOORBEAMS & BOTTOM LATERALS
HALF PLAN OF TOP LATERALS
KENTUCKY SIDE SHOWN
38' 9" Panel Length
U15
L15
Symm.abt.C.L.
4-Ls-6"x 6"x 7/16"
Double Latt. Ls-3"x 2½"x 3/8" Sides
Single " Ls-3"x 2½"x 3/8" Top & Bott.
4-Ls-7"x 3½"x 11/16"
1-Pl.-15"x 7/16"
19'4½"
4-Ls-6"x 4"x 3/8"
Double Latt. Ls-3"x 2½"x 3/8"

FIG. 9.

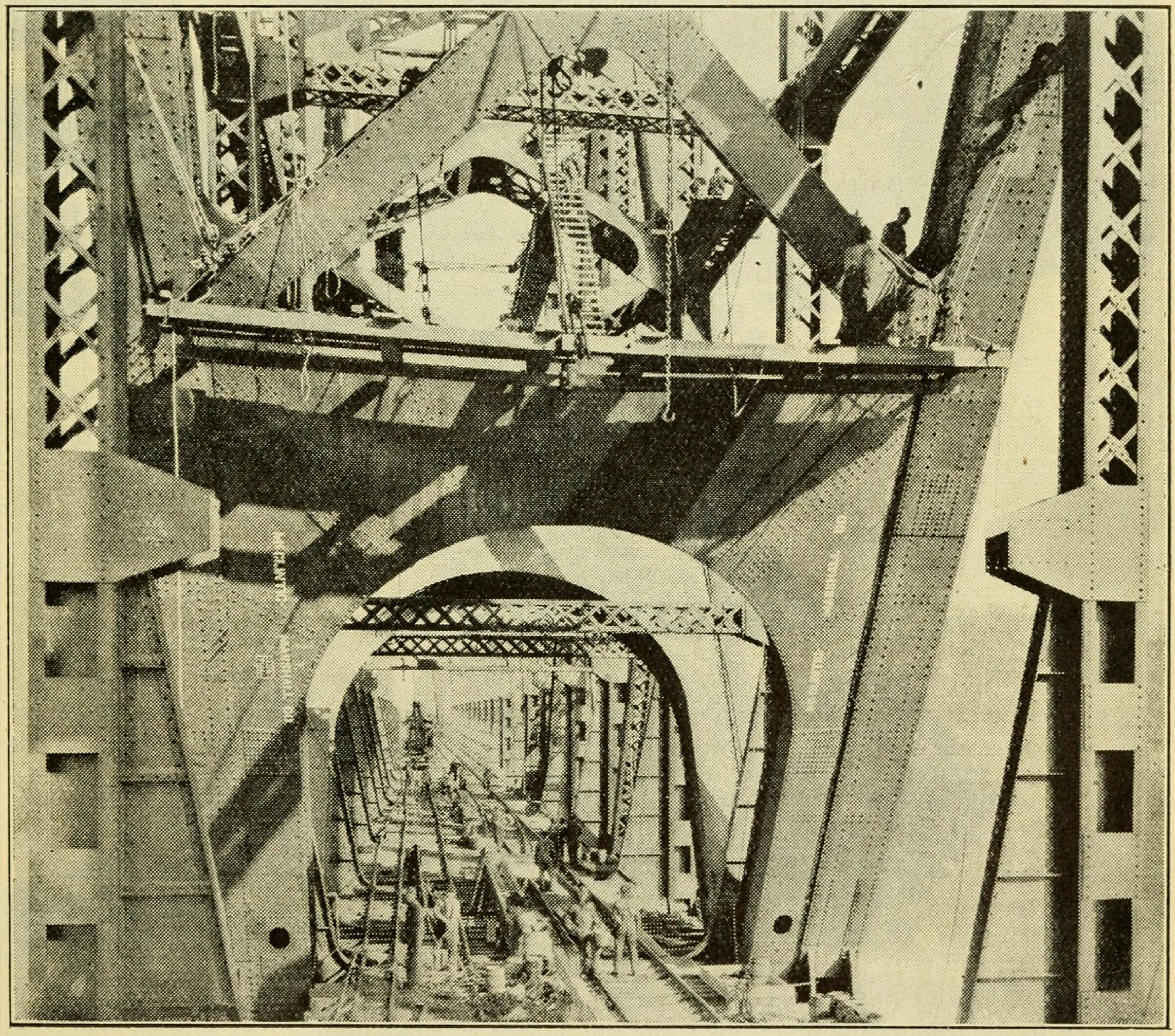


FIG. 10.—BOX GIRDER PORTALS BETWEEN DIAGONALS OVER CENTER PIER, SCIOTOVILLE BRIDGE.

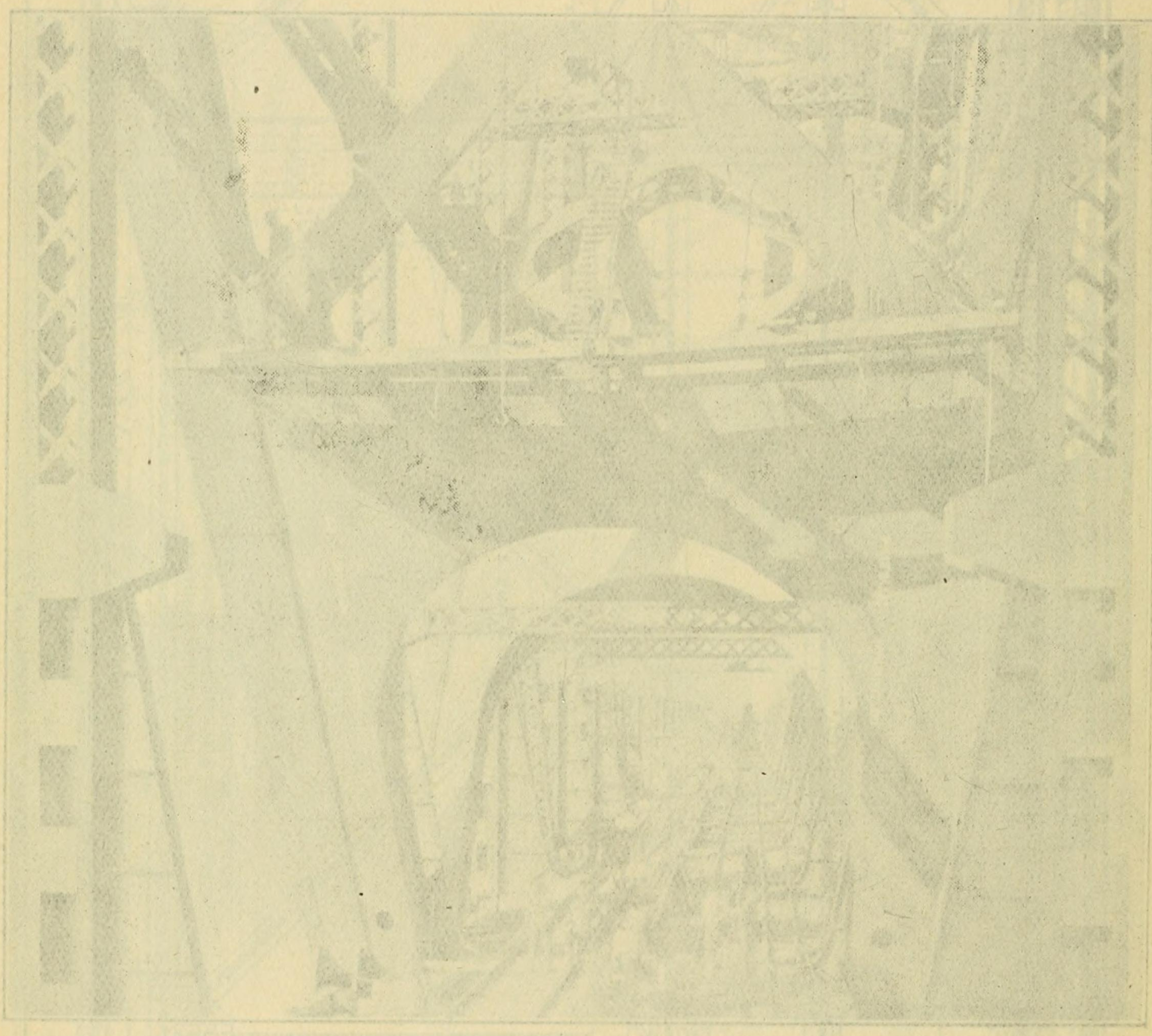


FIG. 10.—BOX GIRDER PORTALS BETWEEN DIAGONALS OVER CENTER PIER, SCIOTOVILLE BRIDGE.

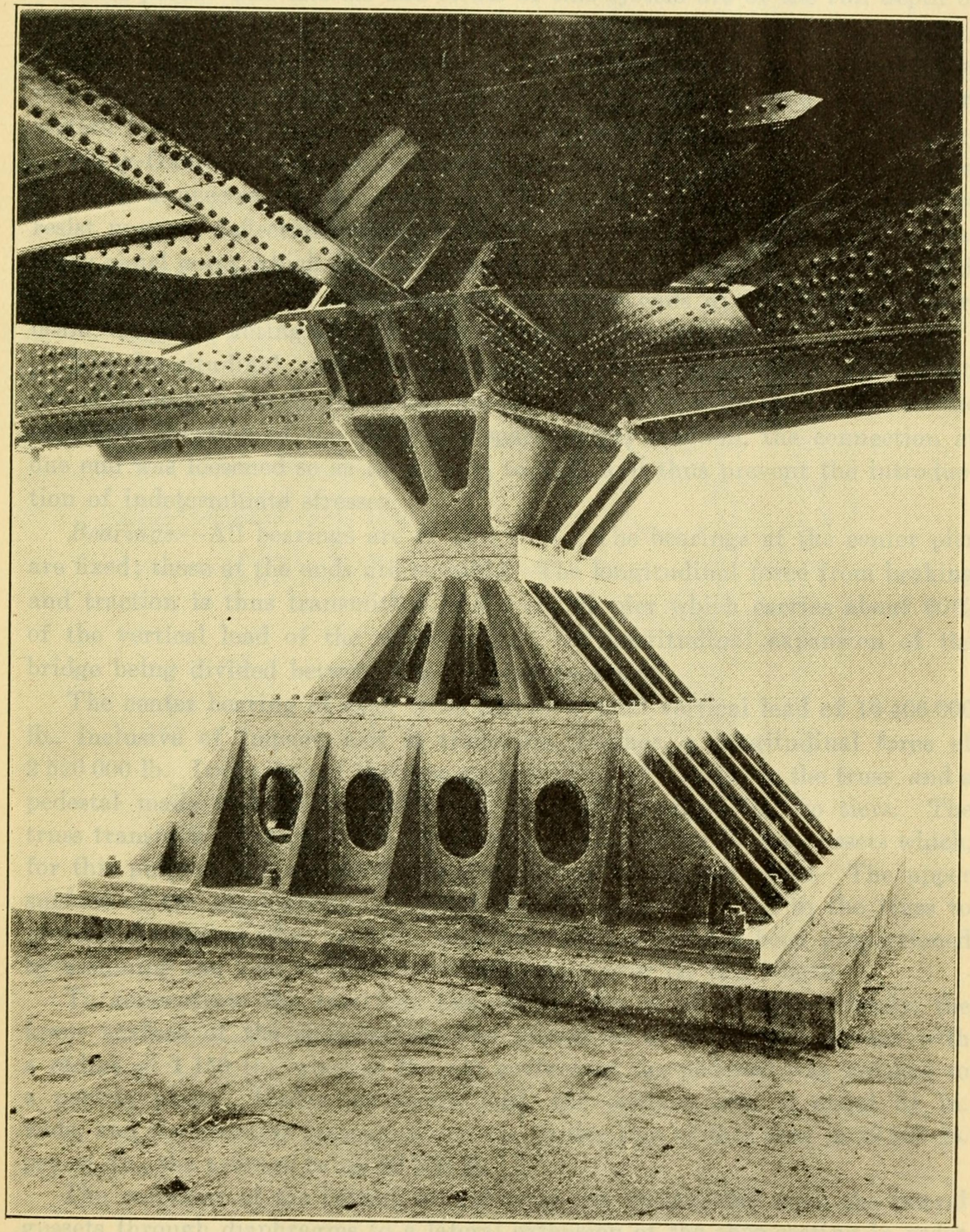


FIG. 11.—STEEL CASTING ON CENTER PIER, SCIOTOVILLE BRIDGE.

There is also a lateral system between the top chords from end to end. The portions between panel points, U_2 and U_8, are assumed to act as simple spans, transmitting the end shears at these points to the portals between the inclined posts. The laterals and struts of this system are of the full depth of the top chord (Fig. 9).

The portals between the end posts and between the inclined posts at the center pier are very rigid. The upper part consists of rigid single intersection braces and the lower one of a solid plate-girder arch (Fig. 10).

Sway-frames between the other web members have been purposely omitted as being unnecessary and as they would have had to be made very strong to resist unequal deflections of the trusses. Instead, there is a lattice frame at every panel point which acts as a lateral brace for the long web members and, at the same time, as a strut between the upper ends of the U-shaped floor-beam, the latter acting as an inverted arch, as described later.

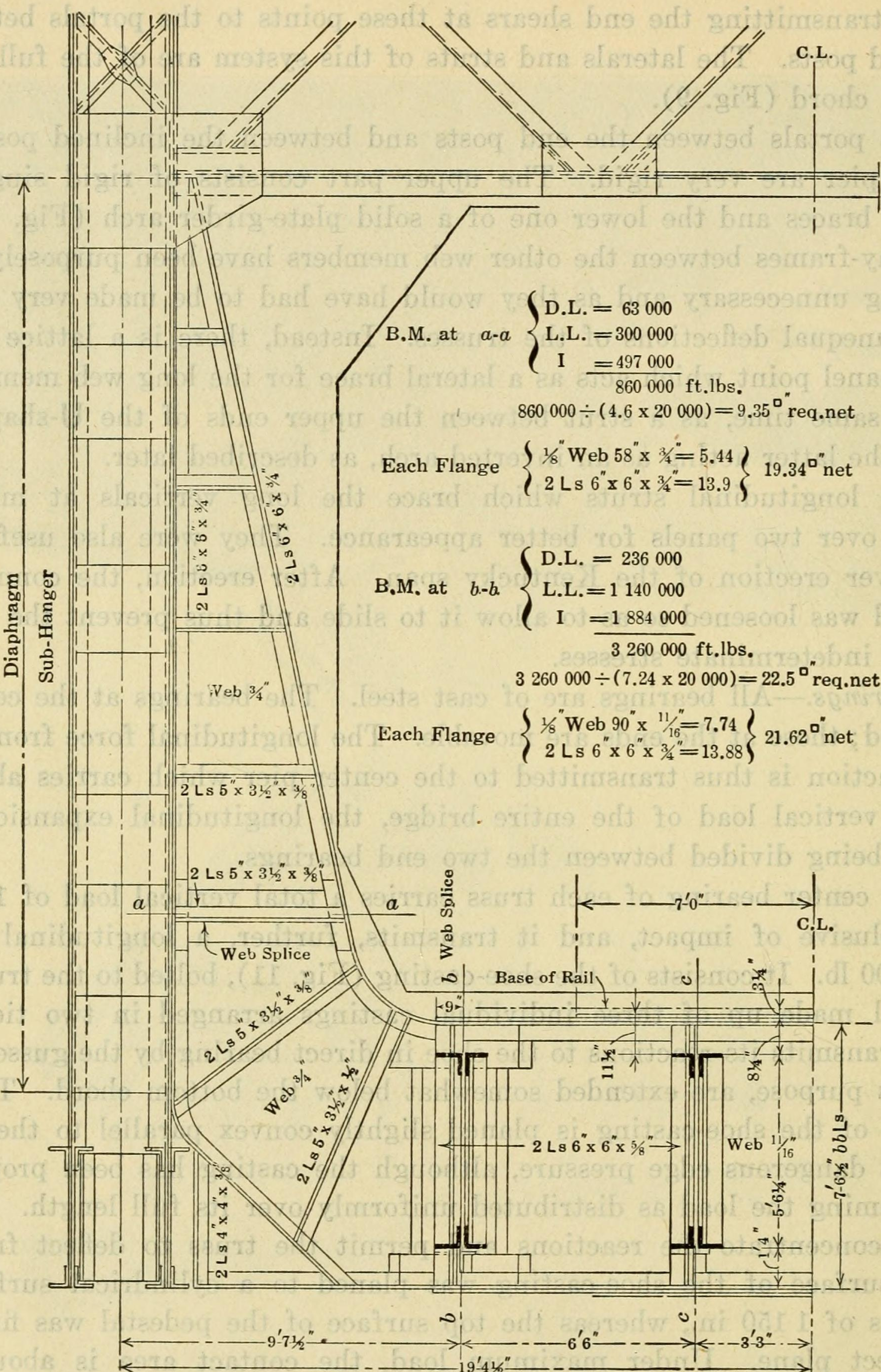
The longitudinal struts which brace the long verticals at mid-height, extend over two panels for better appearance. They were also useful in the cantilever erection of the Kentucky span. After erection, the connection at one end was loosened so as to allow it to slide and thus prevent the introduction of indeterminate stresses.

Bearings.—All bearings are of cast steel. The bearings at the center pier are fixed; those at the ends are movable. The longitudinal force from braking and traction is thus transmitted to the center pier which carries about 60% of the vertical load of the entire bridge, the longitudinal expansion of the bridge being divided between the two end bearings.

The center bearing of each truss carries a total vertical load of 16 406 000 lb., inclusive of impact, and it transmits, further, a longitudinal force of 2 520 000 lb. It consists of the shoe-casting (Fig. 11), bolted to the truss, and a pedestal made up of three individual castings arranged in two tiers. The truss transmits its reactions to the shoe in direct bearing by the gussets which, for this purpose, are extended somewhat below the bottom chord. The upper surface of the shoe-casting is planed slightly convex parallel to the truss to prevent dangerous edge pressure, although the casting has been proportioned by assuming the load as distributed uniformly over its full length.

To concentrate the reactions and permit the truss to deflect freely, the lower surface of the shoe-casting was planed to a cylindrical surface with a radius of 1 150 in., whereas the top surface of the pedestal was finished to a perfect plane. Under maximum load, the contact area is about 8½ in. wide, and the greatest pressure along the center line of this area is 29 900 lb. per sq. in., the average being 23 300 lb.

The reactions of the bottom lateral truss are transmitted from the lateral gussets through diaphragms to a lateral extension of the shoe-casting in order to relieve the gussets of the main trusses of transverse bending. The longitudinal force is transmitted from the main gussets to vertical lugs of the shoe-casting by means of turned bolts 2½ in. in diameter. To prevent horizontal motion, the shoe-casting is secured to the pedestal by four steel dowels 6 in. in diameter.



INTERMEDIATE FLOORBEAM

Assumed Wind, Lateral & Braking Force

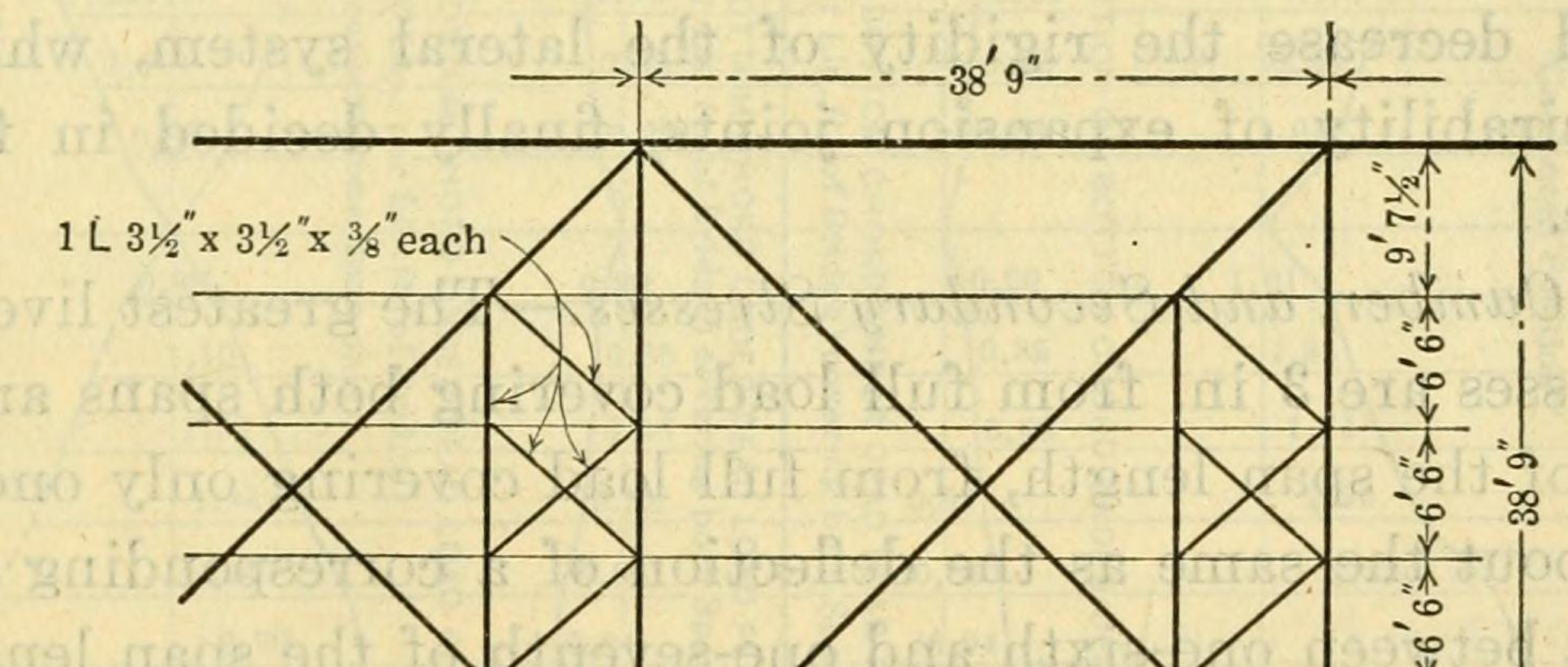
Wind along Top Chord = 800 lb. per lin.ft. of Bridge
" " Bottom Chord = 700 " " " " "
" on Train = 500 " " " " "
Lateral Force from Train = 10% of Live Load (E.60) on one Track
Braking Force = 1000 lb. per lin.ft. of Bridge on one Track
or: $1000\ \frac{25.875}{38.75}$ = 670 lb. per lin.ft. of one Truss

FIG. 12.

The complete bearing weighs 75 tons and the heaviest individual casting, 21 tons. The end bearings are much lighter and are of the ordinary type with a pin, 6 in. in diameter, and a nest of rockers, 16 in. high. Each end bearing weighs, complete, 20 tons.

Floor System.—The tracks are carried by four lines of stringers of the usual plate-girder type, which are framed into the floor-beams.

The floor-beams, however, are of exceptional design. They form U-shaped frames (Fig. 12), extending up to the bottom of the overhead struts. The available height of the floor was too shallow for an economical and stiff floor-beam of the ordinary type computed as a simple span between centers of trusses. There was, however, sufficient room available outside the train-clearance line for wide, deep brackets. By making the latter continuous with the floor-beam proper, it became admissible to compute the stresses by assuming the frame as an inverted two-hinged arch. This reduced the bending moments in the horizontal portion considerably and effected a substantial saving in weight. The overhead strut takes up the horizontal thrust of the inverted arch. The whole arrangement is contributive to stiffness and resistance to vibration, and the bridge behaves most satisfactorily in that regard under heavy trains.



BRAKING TRUSS
(one in each Panel)
(in Plane of Bottom Flange of Stringers)

FIG. 13.

Braking Trusses.—In large bridges, traction or braking trusses are necessary to avoid excessive horizontal bending of the floor-beams by the longitudinal forces. In the Sciotoville Bridge, this has been solved in a novel way by providing such a truss in every panel in the plane of the bottom laterals (Fig. 13). In this manner, the stresses in the stringers from the longitudinal force are reduced to a negligible minimum and the horizontal bending of the floor-beams from that force and from the deformation (changes in length from tension and compression) of the bottom chords, is avoided.

No expansion joints are provided in the floor. This feature introduces high longitudinal stresses in the stringers from the deformation (extension and compression) of the bottom chords. To reduce these stresses to the effect of the live load, the length of the stringers was made to correspond with the length of the bottom chords under full dead load.

The stringer connections were not riveted until the spans had been swung. Even so, it was found necessary to increase the number of rivets in the stringer connections above that required for the vertical shear, by about 20%, or rather the diameter was increased from $\frac{7}{8}$ in. to 1 in. To avoid these high stresses in the stringer connections altogether, provision for an expansion joint in each panel was considered, but it was found that the additional expense for the expansion seats for the stringers would have been considerable and, moreover, the rigidity of the floor would have been greatly impaired.

An arrangement with only four expansion joints, eight panel lengths, or 310 ft., apart, with a braking truss in the middle between two joints, was also considered. Its advantage was a slight saving in weight of braking trusses and comparatively small stresses in the stringers and floor-beams, both from the longitudinal forces and the deformation of the bottom chords. In this case, however, it was not proper to connect the floor laterals rigidly to the stringers, because the deformation of the chords would be transmitted to the stringers through the laterals, and thus severely strain the latter and their connections to the stringers. To omit these connections would increase the weight and decrease the rigidity of the lateral system, which together with the undesirability of expansion joints, finally decided in favor of the adopted scheme.

Deflections, Camber, and Secondary Stresses.—The greatest live load deflections of the trusses are 3 in. from full load covering both spans and $4\frac{5}{8}$ in., or about 1 : 2 000 of the span length, from full load covering only one span (Fig. 14). This is about the same as the deflection of a corresponding simple span having a height between one-sixth and one-seventh of the span length.

As was expected, the deflection polygon showed a rather sharp upward kink at the middle support and indicated high secondary stresses in the vicinity of this support. This was corroborated by a calculation of these stresses, which was made both for full load on one span only and for full load on both spans.

Without provision for reducing them, the largest secondary stresses would be 13 300 lb. per sq. in. from dead load, and 8 100 lb. per sq. in. from live load, or a total of 21 400 lb. per sq. in. in the bottom chord next to the center bearing, and 5 500 lb. per sq. in. from dead load, and 3 300 lb. per sq. in. from live load, or a total of 8 800 lb. per sq. in. in the inclined posts at the center pier. In view of this, and on account of the rigid truss connections, it was considered advisable to reduce the secondary stresses as far as possible, not only near the center support, but throughout the truss, in the chords as well as in the web members. This was done by cambering the trusses for full dead load plus one-half the live load, covering both spans, but assembling and erecting them so that the angles between the members and the bevels of the joints would correspond to the geometric form of truss. In other words, under

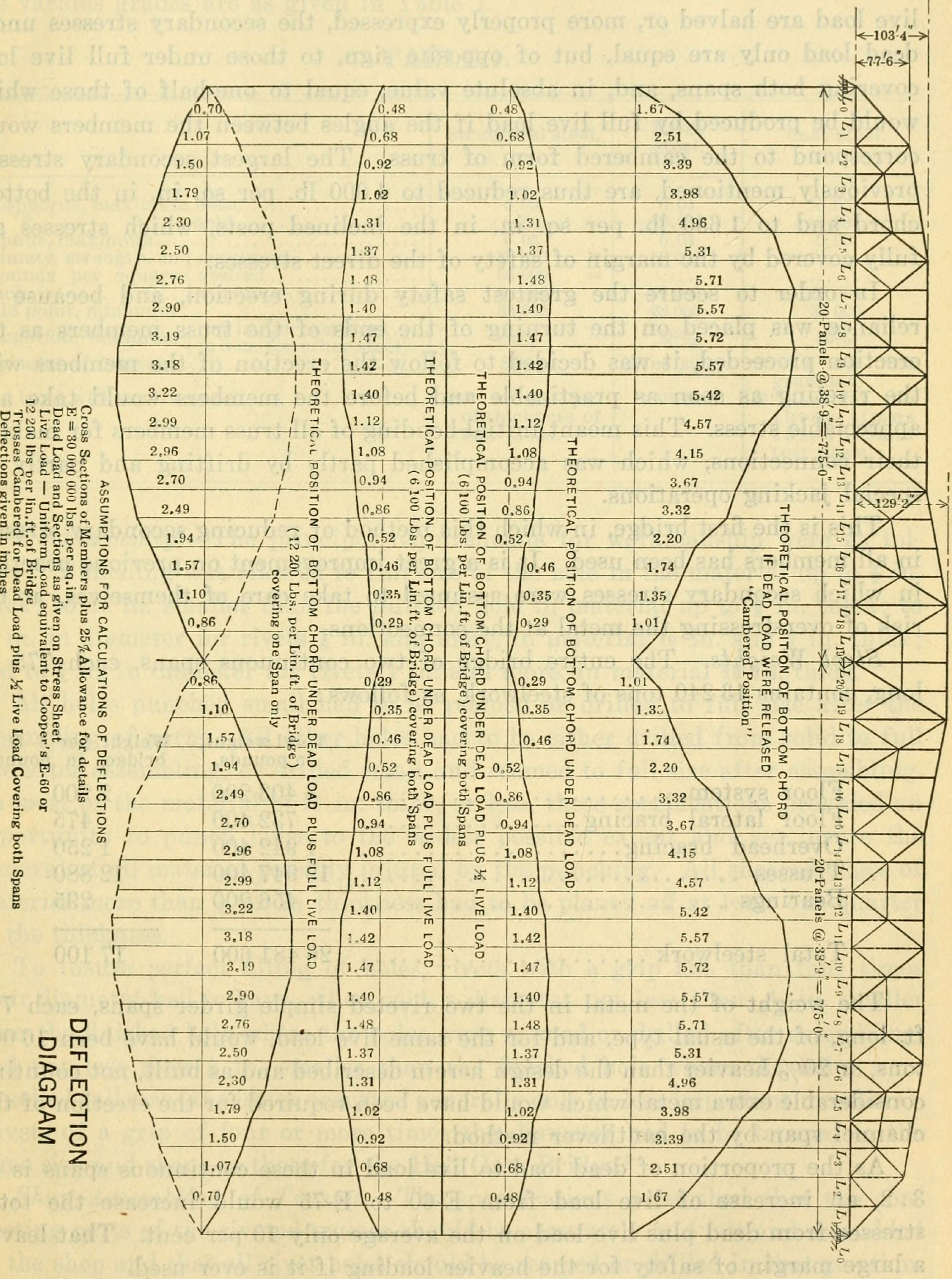

DEFLECTION DIAGRAM
ASSUMPTIONS FOR CALCULATIONS OF DEFLECTIONS
Cross Sections of Members plus 25% Allowance for details
E = 30 000 000 lbs. per sq.in.
Dead Load and Sections as given on Stress Sheet
Live Load — Uniform Load equivalent to Cooper's E-60 or
12 200 lbs per lin.ft.of Bridge
Trusses Cambered for Dead Load plus ½ Live Load Covering both Spans
Deflections given in inches
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD PLUS FULL LIVE LOAD
(12 200 Lbs. per Lin.ft. of Bridge) covering one Span only
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD PLUS FULL LIVE LOAD
(6 100 Lbs. per Lin.ft. of Bridge) covering both Spans
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD PLUS ½ LIVE LOAD
(6 100 Lbs. per Lin.ft. of Bridge) covering both Spans
THEORETICAL POSITION OF BOTTOM CHORD UNDER DEAD LOAD
THEORETICAL POSITION OF BOTTOM CHORD IF DEAD LOAD WERE RELEASED
"Cambered Position"
20 Panels @ 38'-9" = 775'-0"
103'-4"
77'-6"
129'-2"

FIG. 14.

the load $\left(d + \frac{1}{2} l\right)$, the trusses are calculated to assume their true geometric form and the members to become straight and free of secondary stresses.

The secondary stresses from dead load are thus eliminated and those from live load are halved or, more properly expressed, the secondary stresses under dead load only are equal, but of opposite sign, to those under full live load covering both spans, and, in absolute value, equal to one-half of those which would be produced by full live load if the angles between the members would correspond to the cambered form of truss. The largest secondary stresses, previously mentioned, are thus reduced to 4 000 lb. per sq. in. in the bottom chord and to 1 650 lb. per sq. in. in the inclined posts, which stresses are fully covered by the margin of safety of the direct stresses.

In order to secure the greatest safety during erection, and because no reliance was placed on the turning of the ends of the truss members as the erection proceeded, it was decided to follow the erection of the members with the riveting as soon as practicable and before the members would take any appreciable stress. This meant initial bending of all truss members for making their connections, which was accomplished partly by drifting and partly by special jacking operations.

This is the first bridge, in which this method of reducing secondary stresses in all members has been used. It is a great improvement on previous practice, in which secondary stresses were assumed to take care of themselves, at the risk of overstressing the metal at the connections.

Steel Weights.—The entire bridge of two continuous spans, each 775 ft. long, contains 13 240 tons of steelwork, as follows:

	Total weight, in pounds.	Weight per 1 ft. of bridge, in pounds.
Floor system	3 403 200	2 200
Floor lateral bracing	732 400	475
Overhead bracing	1 942 400	1 250
Trusses	19 947 400	12 880
Bearings	456 200	295
Total steelwork	26 481 600	17 100

The weight of the metal in the two riveted simple girder spans, each 775 ft. long, of the usual type, and for the same live load, would have been 16 000 tons, or 20% heavier than the design herein described and as built, not counting considerable extra metal which would have been required for the erection of the channel span by the cantilever method.

As the proportion of dead load to live load in these continuous spans is as 3:2, an increase of live load from E-60 to E-75 would increase the total stresses from dead plus live load on the average only 10 per cent. That leaves a large margin of safety for the heavier loading if it is ever used.

The truss span nearest in length is the 720-ft., double-track, simple girder span of the Ohio River Bridge at Metropolis. The metal weighs 8 023 500 lb., or 11 200 lb. per lin. ft. of bridge, the light weight being due to alloy steel, eye-bars instead of riveted tension members, and the use of higher unit stresses.

5.—Fabrication and Erection of Steelwork

Quality of Steel.—All parts, except the rivets and the cast-steel bearings, are of open-hearth structural steel. The chemical and physical requirements for the various grades are as given in Table 1.

TABLE 1.

	Structural steel.	Rivet steel.	Cast steel.
Phosphorus, maximum basic	0.04	0.04	0.05
" " acid	0.06	0.04	0.08
Sulphur, maximum	0.05	0.04	0.05
Ultimate strength, in pounds per square inch: maximum	70 000	58 000	......
Ultimate strength, in pounds per square inch: desired	66 000	......	......
Ultimate strength, in pounds per square inch: minimum	62 000	50 000	65 000
Yield point, minimum	35 000	28 000	33 000
Elongation, minimum { in 2 in. for cast steel; in 8 in. for other steel	22%	28%	20%
Character of fracture	Silky	Fine silky	Silky or fine granular.
Cold bend without fracture	180° around pin of thickness of test piece.	180° flat	90° around pin three times thickness of test piece.

Workmanship.—The principal requirements for workmanship were as follows: Punching was allowed to full size of the hole in the material up to ½ in. thick; to $\frac{3}{16}$ in. smaller than the finished hole in material up to ⅝ in. thick; to $\frac{11}{16}$ in. in diameter for rivets ⅞ in. and more, in material $\frac{11}{16}$ in. and ¾ in. thick; and to ¾ in. in diameter for rivets 1 in. and more, in material $\frac{13}{16}$ in. thick.

All holes punched small had to be reamed or drilled to full size after the assembling of parts. All other holes had to be either drilled from solid to full size after assembling, or drilled small and reamed to full size after assembling. As most of the material is $\frac{13}{16}$ in. thick, or less, these specifications provided an opportunity to punch holes to the widest possible extent and yet insure the removal of all material possibly injured by the punching. All sheared edges of material more than ½ in. in thickness had to be planed off at least one-quarter of the thickness.

To insure perfect filling of holes, rivets with a grip less than four times their diameter had to have, when cold, a diameter not less than $\frac{3}{64}$ in. smaller than the finished hole where the holes were reamed or drilled after assembling, and not less than ⅛ in. smaller than the finished hole where the holes were punched full size (the latter occurred generally only in less important members). Rivets of a grip of four or more times the diameter had to be tapered to the same size and shape as those for the Hell Gate Bridge.*

Shop Assembling of Trusses.—The specifications prescribed that the connecting parts of the riveted trusses should be accurately laid out and assembled at the shop and that all rivet holes should be reamed or drilled in that position.

The complete assembling of the trusses or, at least, of a series of complete panels, would have assured the greatest accuracy and least chance for errors. This, however, was impracticable because, as previously explained, the angles

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 920.

between the members had to conform to the "geometric" form of the truss, whereas the length of the members in their unstrained condition conformed to the "cambered" form of truss.

To assemble any panel or group of panels completely would have required forcible bending of the members and drilling of the holes in that condition, a very difficult and expensive operation. The trusses, therefore, were assembled in sections, as shown on Fig. 15, by connecting the web members to each chord separately. The members were carefully leveled and laid out with a transit to the correct angles, and the distances were carefully measured with a steel tape. The measurements were usually made in the early morning when the temperature was uniform in all parts.

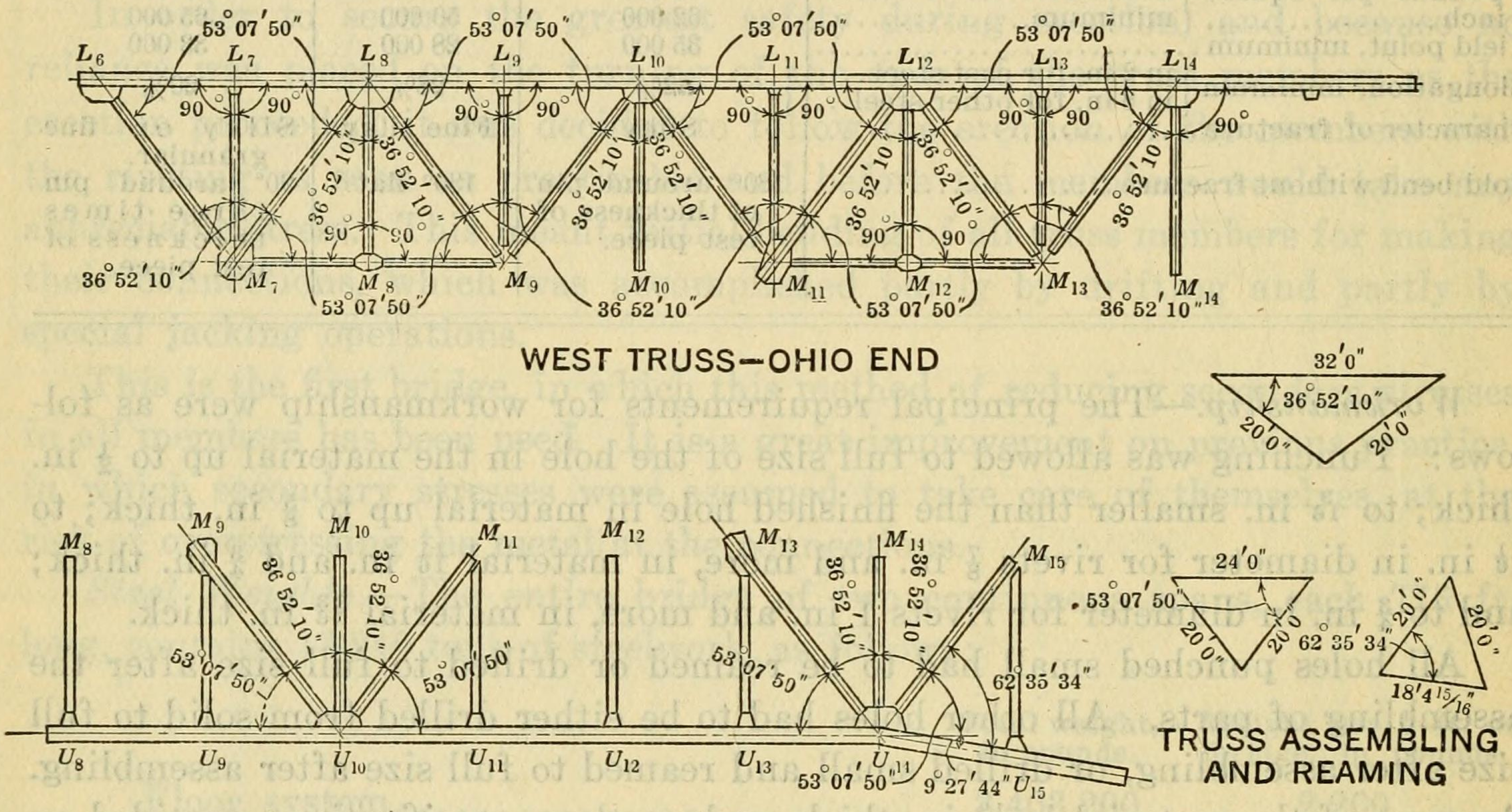


FIG. 15.

Connections were made with a sufficient number of $\frac{5}{8}$-in. tack-bolts to hold the members firmly together during the reaming and drilling of the holes. The holes for these bolts had been previously punched to $\frac{11}{16}$ in. in diameter, and all other holes were then drilled from the solid. For this purpose, all the holes in the top plate of each web, in the position in which the members were laid for assembling, had been previously punched to $\frac{11}{16}$ in., and this plate served as a template for drilling the holes. Finally, the holes which served for the tack-bolts, were reamed to full size.

The chords were laid first to an exact straight line, the joints being brought to perfect and forcible contact, and the reaming and drilling of the holes of the splices was started before the web members were assembled. The two halves of the main diagonals, having a splice at the *M*-points, were assembled to each other in the shop to a straight line, then taken apart and re-assembled to the chords in the assembling yard.

The sub-diagonals, hangers, and horizontal struts were first connected up and reamed at one end, then shifted slightly for connecting up and reaming of the other end. While one end of the sub-diagonals was being reamed, the other end was held in position by a temporary connection through a few extra holes provided for that purpose, which, later, were plugged. Connections of sub-posts, which are of secondary importance, were allowed to be reamed to an iron templet. All connections were match-marked before the assembled sections were dismantled.

Erection.—A detailed description of the erection of the steel superstructure which offered many difficult and unprecedented problems, has been given in a series of articles* by Clyde B. Pyle, M. Am. Soc. C. E., Field Engineer for the McClintic-Marshall Company, the contractors for the fabrication and erection of the steelwork. Therefore, only the general procedure and the conditions and considerations which governed the method of erection, will be mentioned here.

The War Department required the maintenance, during erection, of a minimum clear opening of 420 ft. over the river channel on the Kentucky side, for navigation. It was evident, therefore, that this part of the Kentucky span had to be erected by the cantilever method.

No opening for navigation was required under the Ohio span and this span lent itself better for falsework erection also on account of the shallow bottom. Even with that span, however, great risks had to be taken because of the danger of the falsework being carried away by a sudden flood, especially as no piles could be driven on account of the rock bottom. To minimize this danger the contractor used narrow falsework towers placed under the main panel points only, instead of closely spaced bents, thus leaving openings of about 60 ft. under the main portion of the span for the passage of drift and ice.

The erection of the entire Kentucky span as a cantilever, without intermediate support, would have required heavy additions to the chords and some web members near the middle pier. The plans, therefore, contemplated the erection of the four end panels near the Kentucky shore on falsework, with two panels cantilevering beyond, and the other fourteen panels by cantilevering from the center pier. This scheme would have required only a comparatively small addition to some of the members. The six panels on the Kentucky side would have been erected simultaneously with the other portion, which would have saved considerable time. It would have required, however, a separate erection plant on the Kentucky side, and, moreover, the connection of the two joining arms would have presented difficulties. For these reasons, the contractor preferred to erect the entire span as cantilever from the center pier, giving, however, intermediate support by steel bents at the eighth and fourth panel points from the end pier, as soon as these points were reached. (Plate XII.) Thus, the free cantilever portion was reduced to twelve panels, or 465 ft. Even so, it was the longest cantilever truss ever erected. The two falsework towers near the center pier on the Kentucky side were placed merely for convenience in erecting the extremely heavy panels near that pier by means of the gantry traveler, but did not assist in the cantilever erection of the remainder of the Kentucky span.

One of the principal features of the erection of this bridge was the initial bending of the members, which was necessary in order to reduce the secondary

* *Engineering News-Record*, January 10th and 31st and December 26th, 1918.

stresses, as explained previously. These operations, as well as the adjustments in height of the trusses at the end piers and the temporary intermediate supports, required elaborate preparations and special jacking devices (Fig. 16) as fully described in the article by Mr. Pyle. The contractors deserve full credit for the careful and elaborate manner in which the operations were prepared and successfully executed.

In the main, the erection procedure was as follows: By means of a gantry traveler, the falsework and on it the steel floor system and delivery tracks were laid from the pier on the Ohio shore to two panels beyond the center pier on the Kentucky side. (Fig. 17.) Then, working toward the pier on the Ohio shore, the traveler laid the bottom chords which were riveted at once while lying in a straight line and were then jacked to the desired camber. In this position, the Ohio end was 8¼ in. lower than its final position.

The traveler in the meantime having been raised to its full height and brought back to the center pier, the erection of the trusses was then proceeded with, working toward the Ohio end. (Fig. 18.) It had been intended originally to proceed simultaneously with the cantilever erection of the Kentucky span, but, on account of a shortage of labor and the approaching winter and high-water season, with its dangers to the falsework, all efforts were concentrated on the Ohio span in order to hasten its completion.

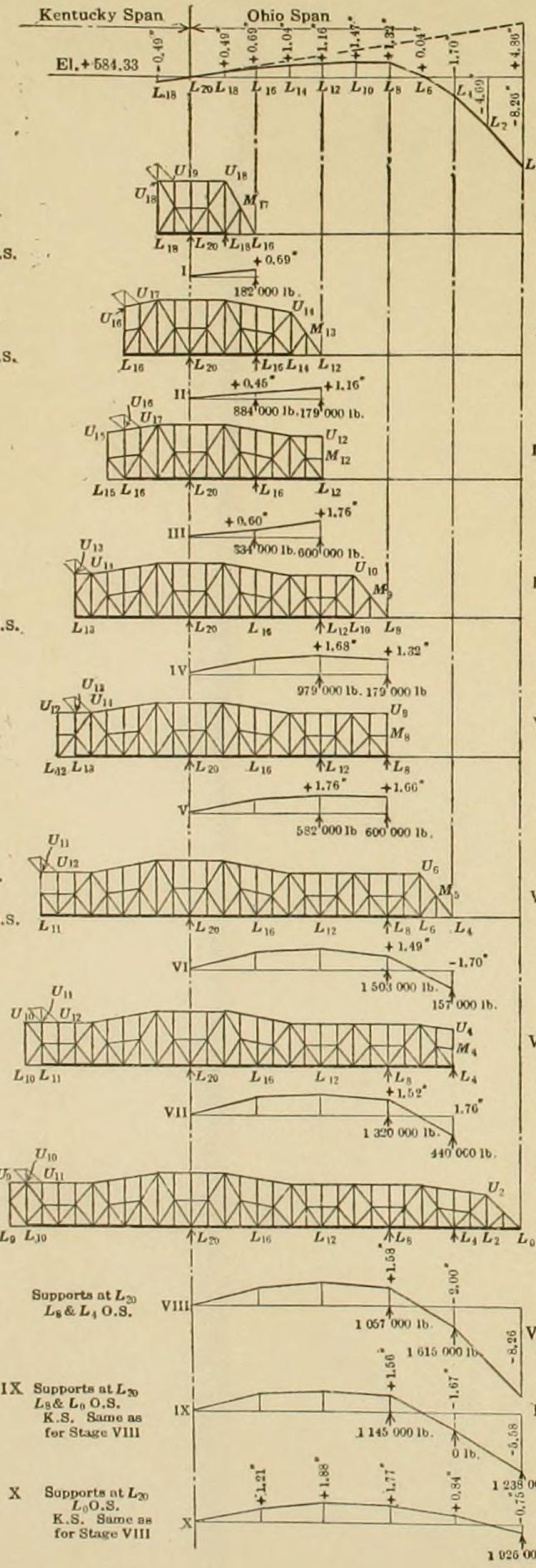
Consequently, the creeper traveler which, in the meantime, had been placed on top of the trusses over the center pier, had erected only one panel on the Kentucky side by the time the Ohio span was completely connected up (Fig. 19). The creeper traveler then proceeded with the cantilever erection of the Kentucky span and, at the same time, the timber falsework under the Ohio span was gradually removed, leaving only the steel columns under Panel Points 4, 8, 12, and 16 to support the trusses (Fig. 20). The releasing of these columns was finally accomplished by jacking the Ohio end of the span to its final position, when the Kentucky cantilever had reached about mid-span. The jacking was done by one 500-ton and four 200-ton hydraulic jacks under each truss (Fig. 21).

The Kentucky cantilever, having reached the eighth panel point from the end (Fig. 20), was jacked up 7⅝ in. from the steel bent erected at that point. This procedure was repeated when the truss reached the next bent at the fourth panel point from the end. The jacking height at that point was 1 in. (Fig. 22). When the truss reached the pier on the Kentucky shore (Fig. 23), it had a deflection of 16¼ in. It was then jacked up to its final position and placed on the rocker bearings, whereby the intermediate supports were released of their load. The final jacking force agreed so closely with calculated reactions that no further adjustment was necessary. Fig. 24 shows a pair of the top chord members being lifted simultaneously.

The operations described indicate sufficiently the sensitiveness of the structure from variations of deflections during erection and the necessity for their accurate analysis and computation in advance in order to insure an exact fit in the connections.

The erection of the steel work was started in June, 1916, and the bridge was completed in August, 1917. From the beginning of the work on the coffer-

Stage	Description
Initial	Elevations of lower chord for connection with diagonals
I	O.S. erected to $L_{18}M_{17}U_{18}$. Floor coupled to L_{18}. K.S. erected to $L_{18}U_{18}$. Floor coupled to L_{18}. Creeper in process of erection at $U_{18}U_{19}$ K.S. Elevation of lower chord for connection at L_{16} O.S. Supports at $L_{20}L_{18}$ O.S.
II	O.S. erected to $L_{12}M_{13}U_{14}$. Floor coupled to L_{14}. K.S. erected to $L_{16}U_{16}$. Floor coupled to L_{16}. Creeper at $U_{16}U_{17}$ K.S. Elevation of lower chord for connection at L_{12} O.S. Supports at $L_{20}L_{16}$ O.S.
III	O.S. erected to $L_{12}U_{12}$. Floor coupled to L_{12}. K.S. erected to $L_{15}U_{15}$. Floor coupled to L_{16}. Creeper at $U_{16}U_{17}$ K.S. Supports at $L_{20}L_{16}$ & L_{12} O.S.
IV	O.S. erected to $L_8M_9U_{10}$. Floor coupled to L_{10}. K.S. erected to $L_{13}U_{13}$. Floor coupled to L_{13}. Creeper at $U_{13}U_{14}$ K.S. Elevation of lower chord for connection at L_8 O.S. Supports at $L_{20}L_{12}$ O.S.
V	O.S. erected to L_8U_8. Floor coupled to L_8. K.S. erected to $L_{12}U_{12}$. Floor coupled to L_{12}. Creeper at $U_{13}U_{14}$ K.S. Supports at $L_{20}L_{12}$ & L_8 O.S.
VI	O.S. erected to $L_4M_5U_6$. Floor coupled to L_6. K.S. erected to $L_{11}U_{11}$. Floor coupled to L_{11}. Creeper at $U_{11}U_{12}$ K.S. Elevation of lower chord for connection at L_4 O.S. Supports at $L_{20}L_8$ O.S.
VII	O.S. erected to L_4U_4. Floor coupled to L_4. K.S. erected to $L_{10}U_{10}$. Floor coupled to L_{11}. Creeper at $U_{11}U_{12}$ K.S. Supports at $L_{20}L_8$ & L_4 O.S.
VIII	O.S. erected to L_0. Floor coupled to L_0. K.S. erected to L_9U_9. Floor coupled to L_{10}. Creeper at $U_{10}U_{11}$ K.S.
VIII	Supports at L_{20} L_8 & L_4 O.S.
IX	Supports at L_{20} L_8 & L_0 O.S. K.S. Same as for Stage VIII
X	Supports at L_{20} L_0 O.S. K.S. Same as for Stage VIII



Erection Notes

Initial position of lower chord after being fully riveted up and before Web members are connected

I Trusses erected to L_{18} O.S. No Jacking

II Trusses erected to L_{12} O.S. No Jacking

III Trusses erected to U_{12} O.S. Jack up with 600 000 lb. under L_{12}

IV Trusses erected to L_8 O.S. Column under L_{16} released No Jacking

V Trusses erected to U_8 O.S. Jack up with 600 000 lb. under L_8

VI Trusses erected to L_4 O.S. Column under L_{12} released No Jacking

VII Trusses erected to U_4 O.S. No Jacking

Trusses erected to L_0 O.S.

VIII Before jacking at L_0

IX Jack up under L_0 to release column under L_4

It is however not necessary to stop jacking at this intermediate stage

X Continue jacking under L_0 to release column under L_8 and remove that column to L_4 K.S. Insert rollers under L_0 O.S. and bring L_0 O.S. to final position

TEMPERATURE CORRECTIONS

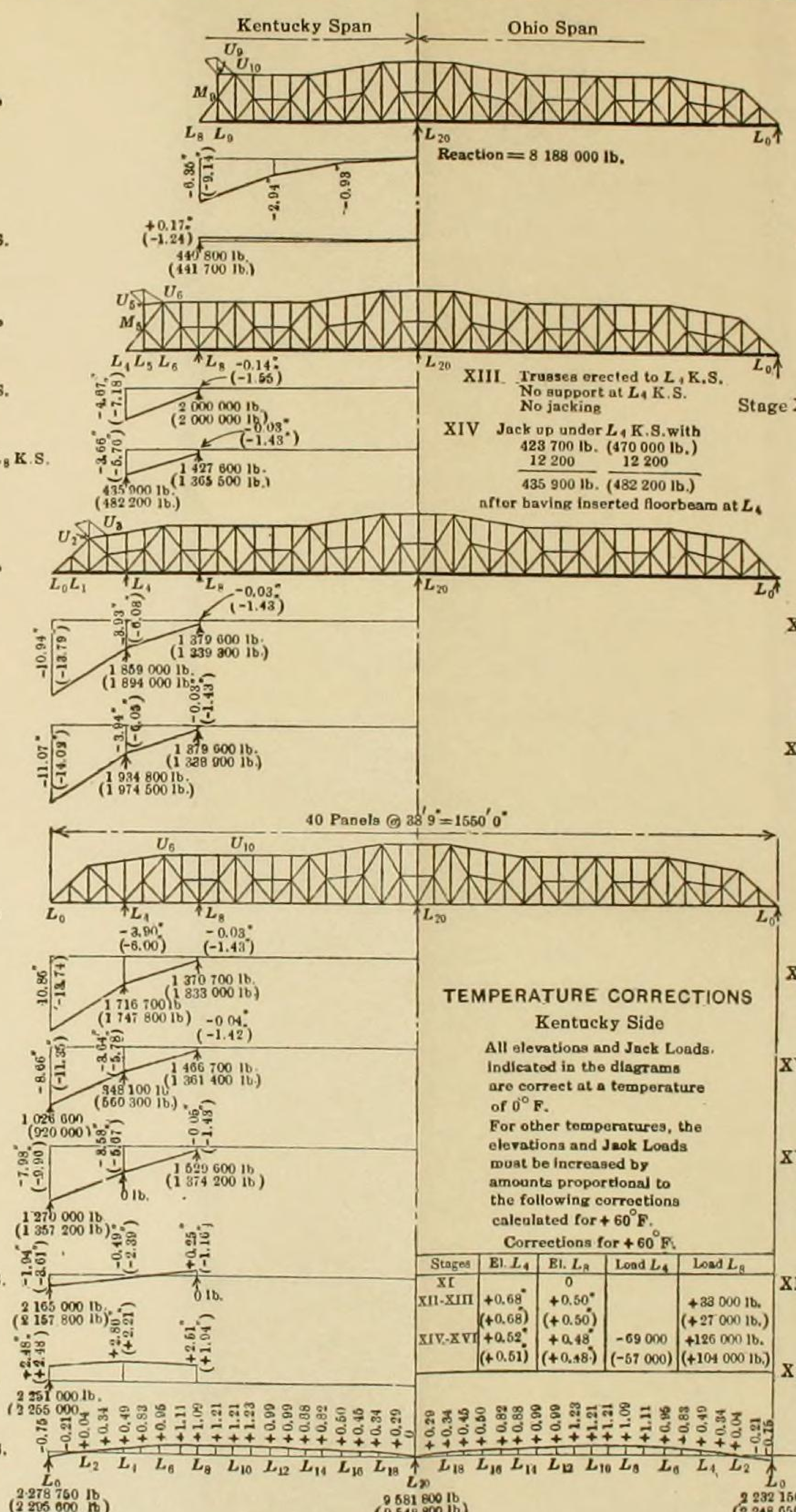
Ohio Side

Effect on Reactions of + 10° F.
Change in Temperature after Jacking

Stages	Increase in Reaction
I	L_{18}: No Change
II	L_{16}: No Change
III	L_{18}: + 62 000, L_{12}: − 31 000
IV	L_{12}: No Change
V	L_{12}: + 24 000, L_8: − 16 000
VI	L_8: No Change
VII	L_8: + 13 000, L_4: − 10 000
VIII	L_8: + 13 000, L_4: − 10 000
IX	L_0:
X	L_0: No Change

Each steel column (under L_4, L_8, L_{12} and L_{16}) expands ½ inch for a temperature rise of 60° F. The temperatures should therefor be noted and reported whenever levels are taken and whenever jacking is done.

Stage	Description
XI	Trusses erected from L_0 O.S. to $L_8M_9U_9$ K.S. Floor coupled to L_9 K.S. Creeper at U_9U_{10}. Supports at L_0 O.S.: L_{20}
XII	Supports at L_0 O.S.; L_{20}: L_8 K.S.
XIII	Trusses erected from L_8 O.S. to $L_4M_5U_5$ K.S. Floor coupled to L_5 K.S. Creeper at U_5U_6 K.S. Supports at L_0 O.S.; L_{20}; L_8 K.S.
XIV	Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
XV*a*	Trusses erected from L_0 O.S. to L_0 K.S. Floor coupled to L_1 K.S. Creeper at U_2U_3 K.S. Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
XV*b*	Floor coupled to L_0 K.S. Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
XVI	Trusses erected from L_0 O.S. to L_0 K.S. Floor coupled to L_0 K.S. Creeper removed. Supports at L_0 O.S.; L_{20}; L_4 & L_8 K.S.
XVII	Supports at L_0 O.S.; L_{20}; L_0 L_4 & L_8 K.S.
XVIII	Supports at L_0 O.S.; L_{20}; L_0 & L_8 K.S.
XIX	Supports at L_0 O.S.; L_{20}; L_0 K.S.
XX	Reaction at L_0 K.S. from dead load & track of 300 lb. per lin. ft.
Final	Reactions at L_0 K.S., L_{20}; L_0 O.S. from dead load and finished flooring at 27 000 lb. per panel



Erection Notes

Creeper moved ahead to U_9U_{10} K.S.
Suspended jacking bridge removed
From this time on the flat car and locomotive crane should not be allowed out beyond L_{18} K.S.
Erect steel bent under L_8 K.S.

XI Trusses erected to L_8 K.S. No support at L_8 K.S.

XII Jack up under L_8 K.S with 428 600 lb. (429 500 lb.)
+ 12 200   12 200
440 800 lb. (441 700 lb.)
Insert floorbeam at L_8 before jacking

Steel bent under L_4 K.S. erected

XIII Trusses erected to L_4 K.S. No support at L_4 K.S. No jacking

XIV Jack up under L_4 K.S. with 423 700 lb. (470 000 lb.)
12 200   12 200
435 900 lb. (482 200 lb.)
after having inserted floorbeam at L_4

Stage XIV The actual jacking force to be applied should be established by the following table:
425 000 lb. lift of L_4 = 1.02
434 000 lb. lift of L_4 = 1.10
443 000 lb. lift of L_4 = 1.18
452 000 lb. lift of L_4 = 1.26
461 000 lb. lift of L_4 = 1.34
470 000 lb. lift of L_4 = 1.42
Whenever the force applied and the actual lift coincide with the figures in the table, we have the ideal condition aimed for

XV*a* Trusses erected to L_0 K.S. No support at L_0 K.S. No jacking

XV*b* End floorbeam coupled to trusses No jacking

XVI No support at L_0 K.S. Creeper removed

XVII Jack up under L_0 K.S. with 1 026 600 lb. (920 000 lb.) when stress in U_9-U_{10} should be zero

XVIII Jack up under L_0 K.S. with 1 270 000 lb. (1 357 200 lb.) when thrust on column under L_8 should be zero

XIX Jack up under L_0 K.S. with 2 165 000 lb. (2 157 800 lb.) when thrust on column under L_8 should be zero

XX Jack up under L_0 K.S. with 2 251 000 lb. (2 255 000 lb.)

TEMPERATURE CORRECTIONS

Kentucky Side

All elevations and Jack Loads indicated in the diagrams are correct at a temperature of 0° F. For other temperatures, the elevations and Jack Loads must be increased by amounts proportional to the following corrections calculated for + 60° F.

Corrections for + 60° F.

Stages	El. L_4	El. L_8	Load L_4	Load L_8
XI		0		
XII-XIII	+0.68" (+0.68)	+0.50" (+0.50)		+33 000 lb. (+27 000 lb.)
XIV-XVI	+0.52" (+0.51)	+0.48" (+0.48)	−69 000 (−57 000)	+126 000 lb. (+104 000 lb.)

DEFLECTION CONSTANTS (Ky. Side)

	Deflection at L_0	Deflection at L_4	Deflection at L_8
Bridge erected to L_8, Traveler at U_9 U_{10}			−13.03 (−15.82)
Bridge erected to L_4, Traveler at U_5 U_6		−51.99 (−63.10)	−37.22 (−45.08)
Bridge erected to L_0, Traveler at U_2 U_3	−121.86 (−148.06)	−94.93 (−115.12)	−66.27 (−80.20)
100 000 lb. at L_0	5.17 (6.27)	3.90 (4.72)	2.63 (3.19)
100 000 lb. at L_4	3.90 (4.72)	3.05 (3.69)	2.08 (2.51)
100 000 lb. at L_8	2.63 (3.19)	2.08 (2.51)	1.52 (1.84)
Column Compression for a load of 100 000 lb.	0	0.0193	0.0193

DIAGRAMS SHOWING PRINCIPAL ERECTION STAGES. DEFLECTIONS AND JACKING OPERATIONS

NOTES:- All Reactions and Jacking Loads are given in Pounds for one Truss.
All Deflections are given in inches.
Figures in parentheses are computed on the basis of no allowance made for details as affecting the cross-sections of the members.
All other figures are computed on the basis of an addition to the cross-sections equivalent to 75% of the weight of the details of the members.
Figures of elevations denote the height above or below a horizontal line through the L_{20} point. (El. Base of Rail + 590.08.)
O. S. denotes Ohio Span. K. S. denotes Kentucky Span
Compare Mc Clintic Marshall Co.'s Drawings: D_1, D_2, D_3 & D_4

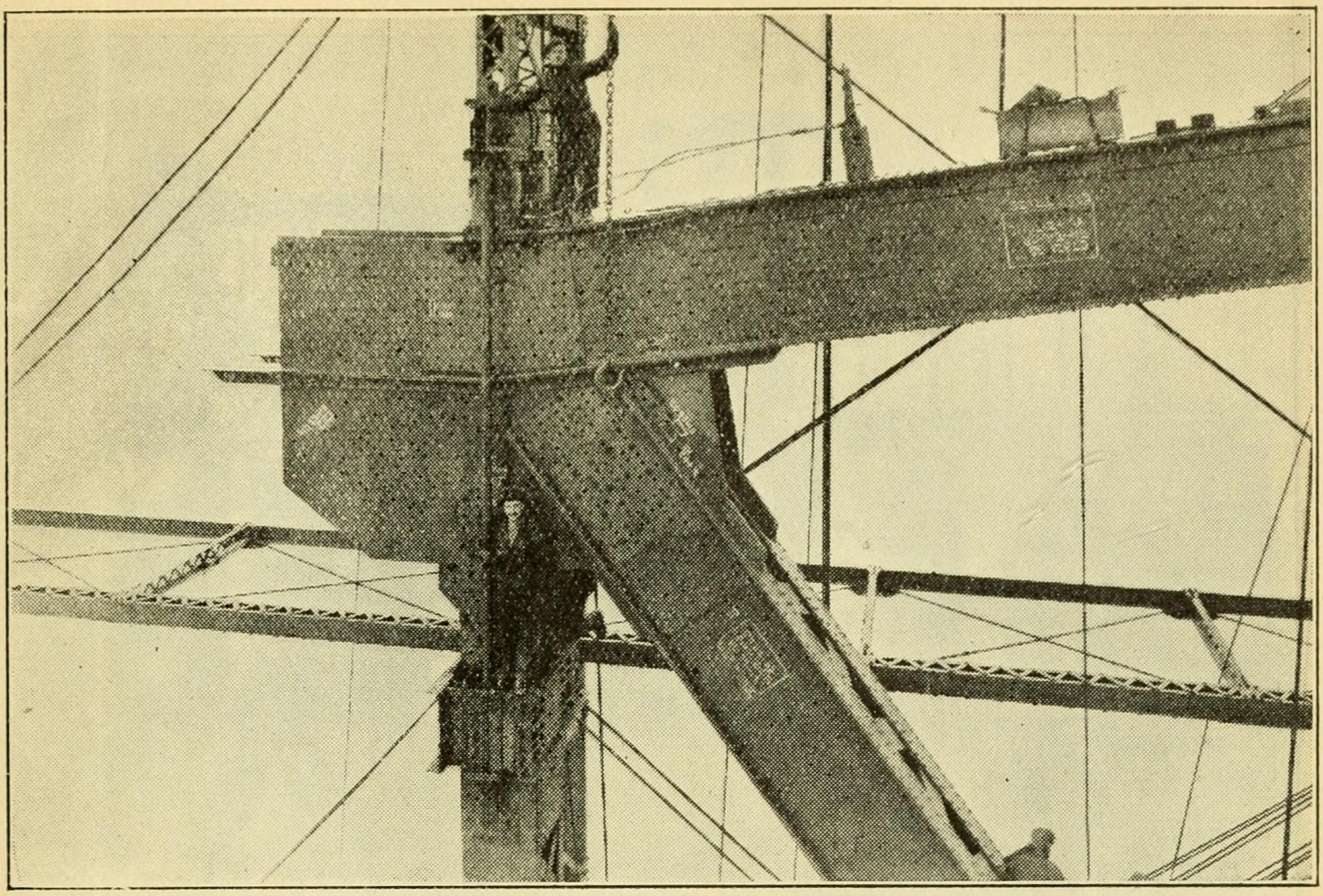


Fig. 16.—Special Jacking Apparatus, Sciotoville Bridge.

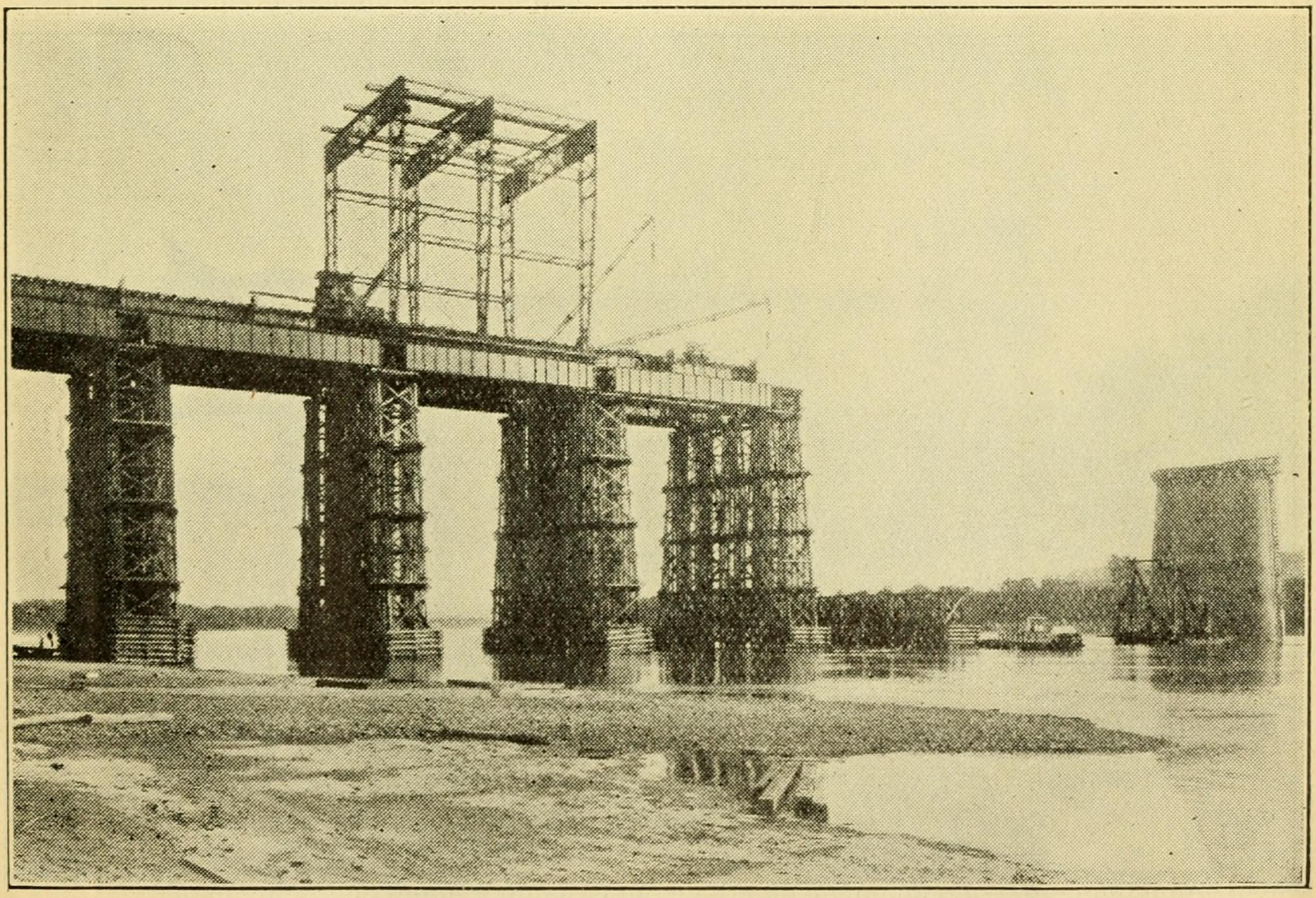


Fig. 17.—Erection of Falsework and Laying of Steel Floor System and Tracks by Gantry Traveler, Sciotoville Bridge.

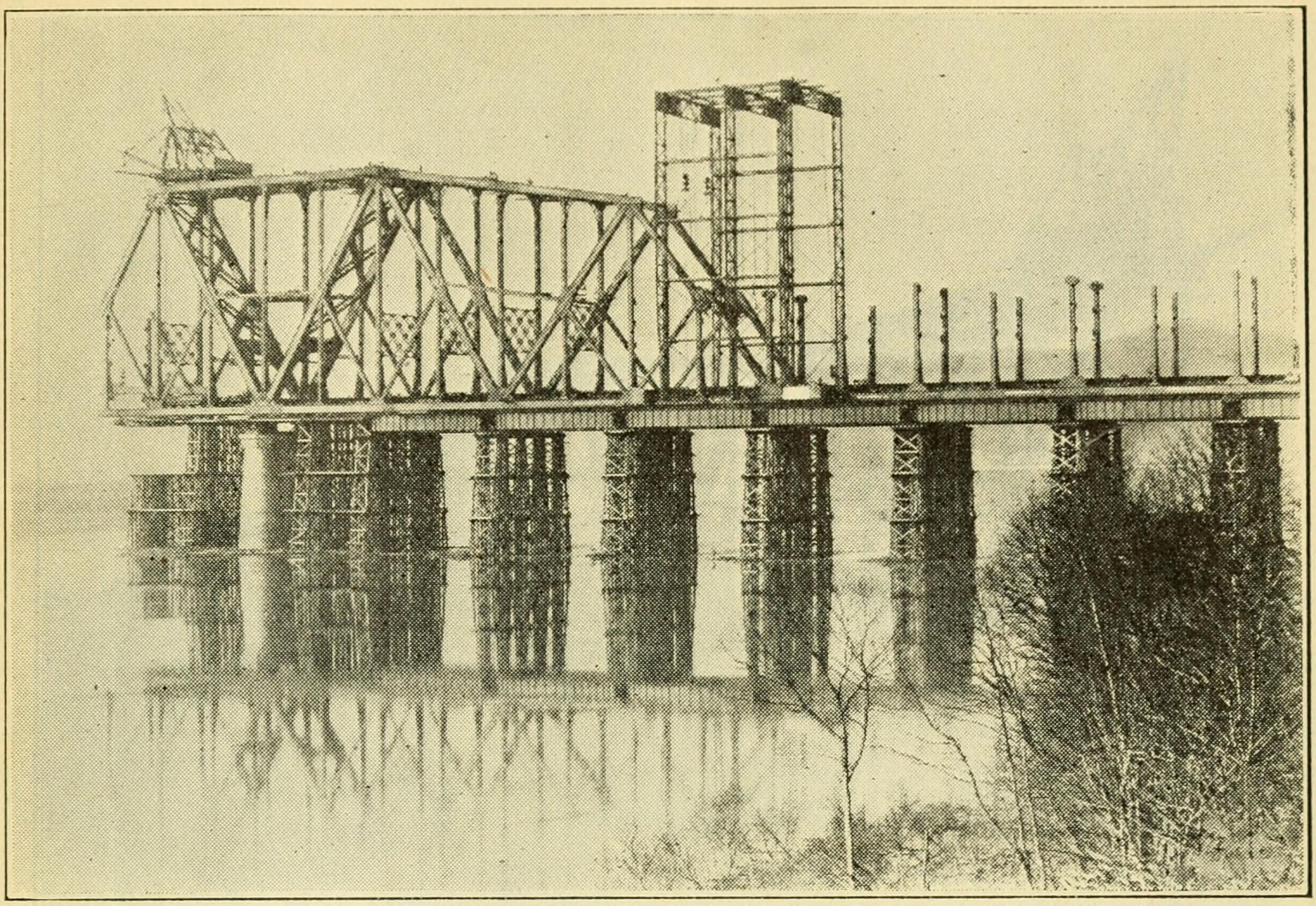


FIG. 18.—ERECTION OF TRUSSES, OHIO SPAN, SCIOTOVILLE BRIDGE.

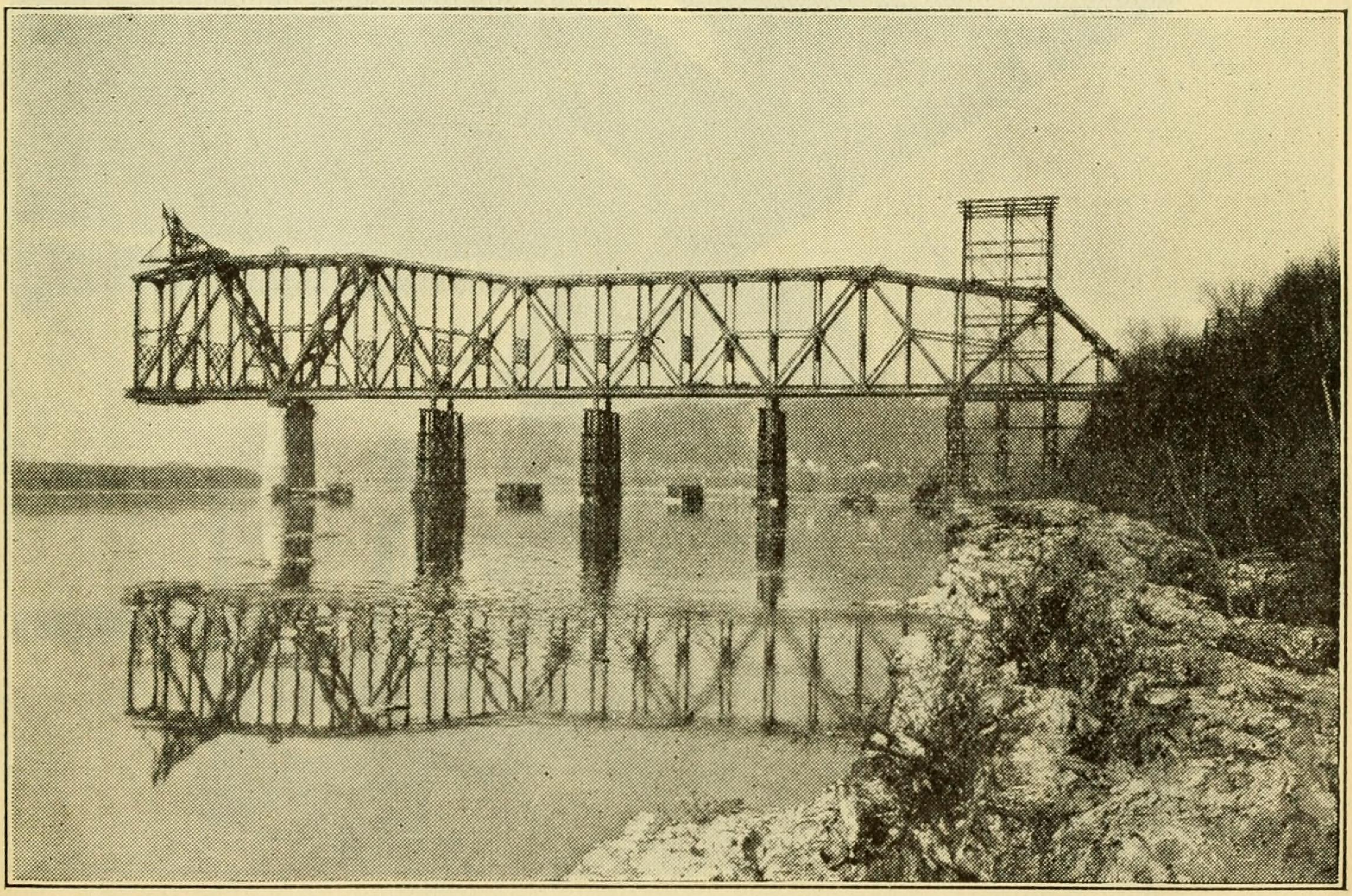


FIG. 19.—ERECTION OF OHIO SPAN COMPLETED, SCIOTOVILLE BRIDGE.

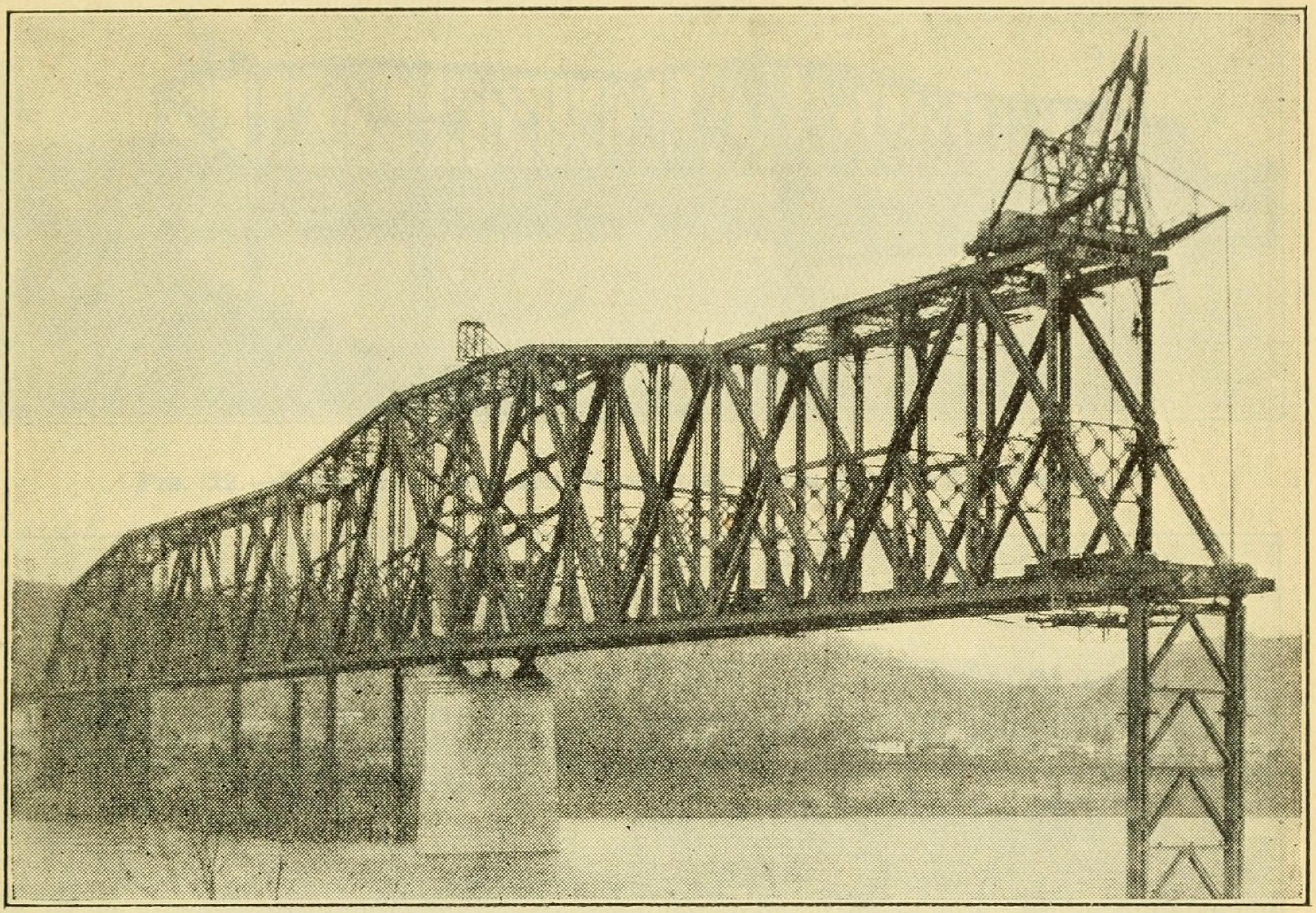


Fig. 20.—Cantilever Erection, Kentucky Span, and Falsework Under Ohio Span Removed, Sciotoville Bridge.

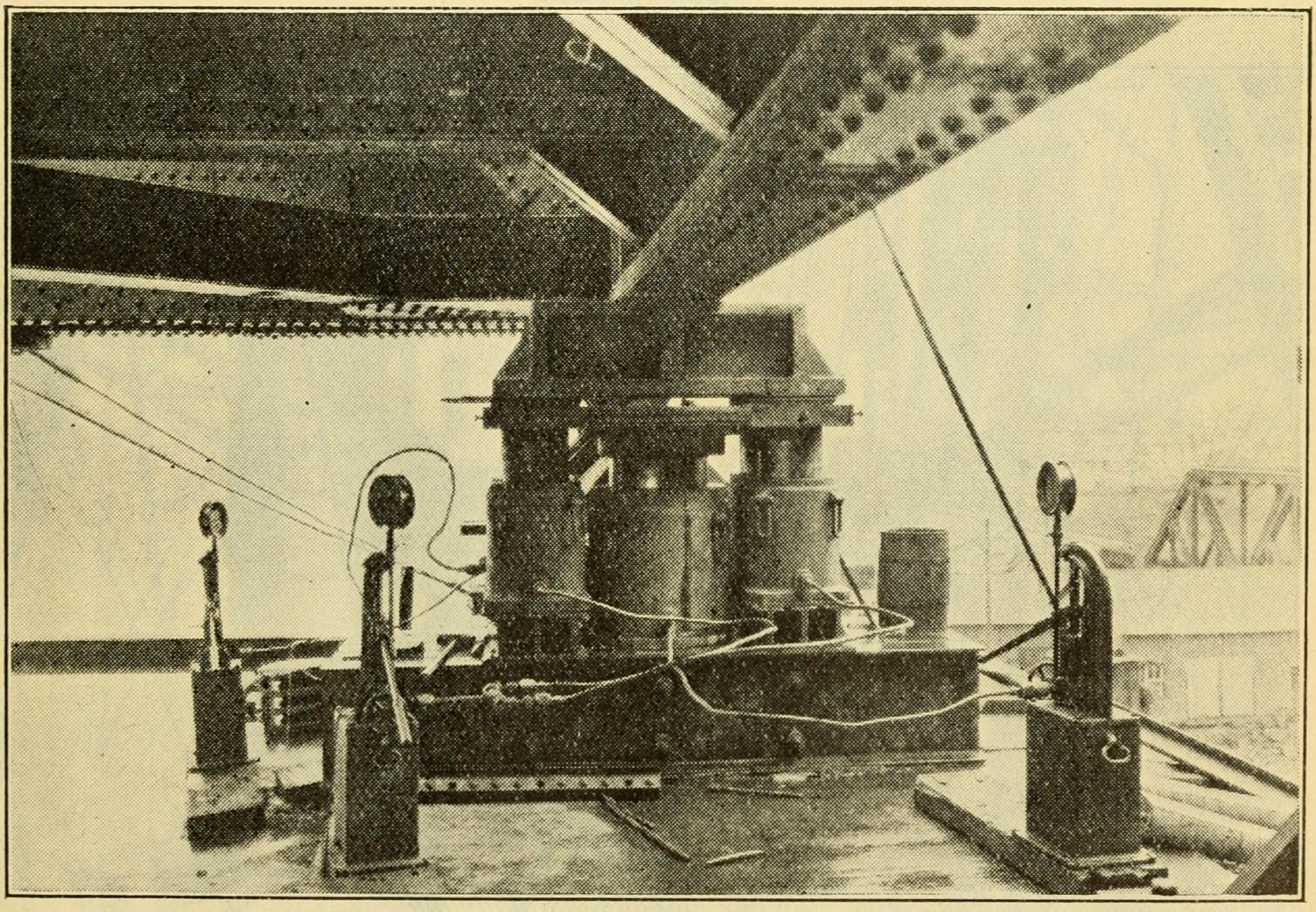


Fig. 21.—Adjustment of Kentucky Span, Sciotoville Bridge, by Hydraulic Jacks.

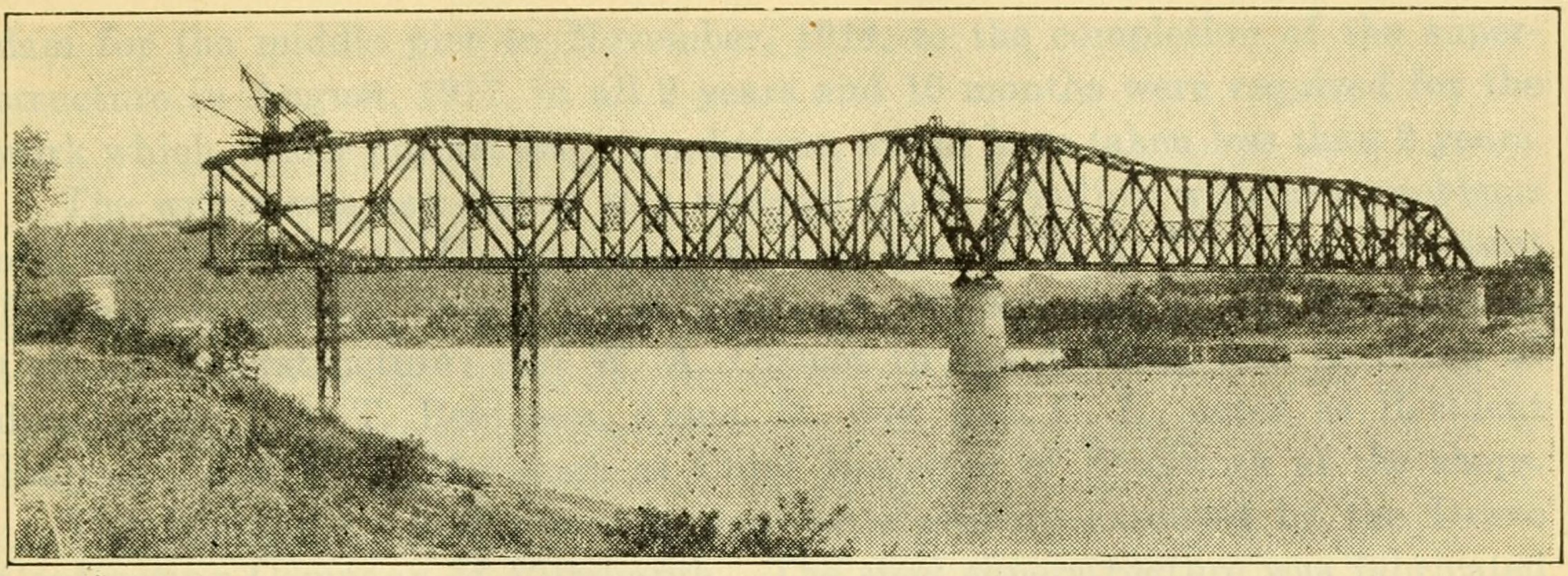


Fig. 22.—Cantilever Section, Kentucky Span, Sciotoville Bridge.

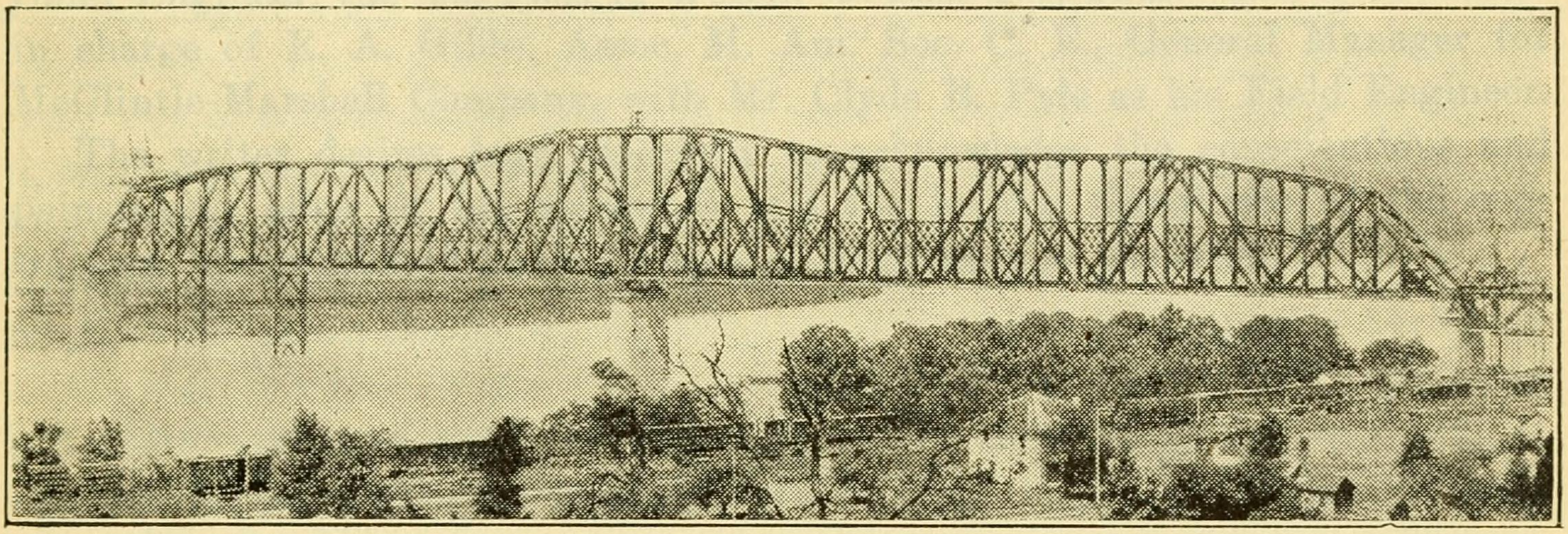


Fig. 23.—Kentucky Span, Sciotoville Bridge: Temporary Supports Ready for Dismantling.

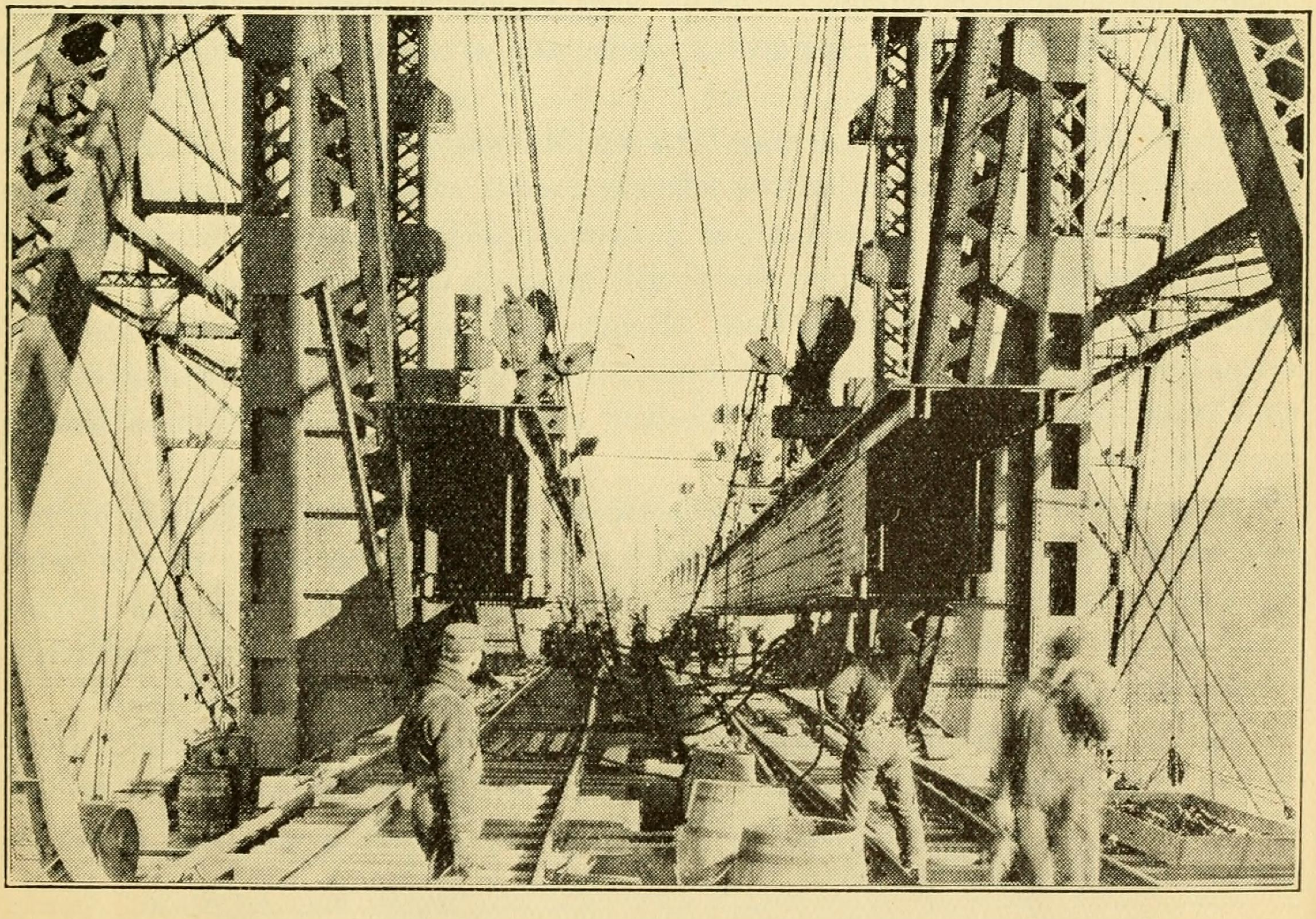


Fig. 24.—Pair of Top Chord Members, Sciotoville Bridge, Being Lifted Simultaneously.

dam for the middle pier in November, 1914, to the completion of the superstructure in August, 1917, in all 2 years and 10 months were required for the work which, under normal labor conditions, would have taken less than 2 years.

The writer was assisted in this unusual work, bristling with new problems and difficulties, by O. H. Ammann, M. Am. Soc. C. E., as Principal Assistant Engineer in general charge; D. B. Steinman, M. Am. Soc. C. E., on computations of superstructure; and W. A. Cuenot in the drafting and checking of detail plans. R. T. Robinson, Assoc. M. Am. Soc. C. E., acted as Resident Engineer, and R. E. McGough as Chief Inspector of Steelwork at the shops.

The foundations and masonry were satisfactorily executed by the Dravo Contracting Company of Pittsburgh. The steel superstructure was fabricated and erected by the McClintic-Marshall Company of Pittsburgh, under its Chief Engineer, the late Paul L. Wolfel, M. Am. Soc. C. E. The erection was in charge of E. A. Gibbs, Assoc. M. Am. Soc. C. E., General Manager for McClintic-Marshall Company, with Mr. Clyde B. Pyle as his Field Engineer.

The writer desires here especially to acknowledge the conscientious and painstaking labor of his assistants and the helpful experience of the contractors, who combined in the successful completion of the work.

DISCUSSION

C. A. P. Turner,* M. Am. Soc. C. E. (by letter).—This paper on the Sciotoville Bridge, coming as it does from an engineer who has earned his place in the front rank of the Profession by the design and execution of one of the finest examples of long-span bridge construction—the Hell Gate Arch Bridge—will be received and read with unusual interest. The structure described is pleasing in appearance, satisfactory from the standpoint of rigidity and safety, and was erected in a creditable manner without mishap.

The span is somewhat in excess of those common in simple bridge truss design; but the fact that it is a double-track structure, and is wider and of correspondingly greater weight than a single-track railway bridge or the ordinary highway bridge, should extend the economic span length for the simple truss up to 850 ft. at least.

As an advocate of the continuous bridge, Mr. Lindenthal does not limit its application to long spans in place of the cantilever, but contends that it is economical for spans customarily regarded as solely within the province of the simple truss span. In assuming this position, he has raised fundamental questions regarding the underlying considerations affecting the economy of truss design.

The disadvantages of the continuous structure have been reasonably and fairly treated in the paper. They should not prevent the adoption of the type, providing economy results. However, if the economy claimed by the advocates of the continuous truss is unsubstantiated by the application of mechanical laws governing economic proportions and form, then the field of the type is limited to long spans which, designed with economic depth for vertical loads as simple spans, would be top-heavy under the overturning moment of wind pressure.

In the now classic controversy between Mansfield Merriman, M. Am. Soc. C. E., and Charles Bender, on the relative merits of continuous *versus* simple span bridge structures, Merriman as a strong point predicated the economy of the continuous truss as against the simple truss on the known economy of continuous beams of uniform section. Bender claimed that he could design a series of simple trusses, which would meet the requirements of a given specification with less metal than his opponent could design a continuous bridge covering several spans.

It would seem that the analogy between the economical relations of the continuous beam to the simply supported beam is inapplicable to the relations of the simple *versus* the continuous truss frame, because the web is of constant section in the beam, whereas in the truss it is proportioned to meet the requirements only of the variation of shear along the length of the span.

Again, although the claimed ability of Bender to design simple trusses of less weight than the continuous frame proposed by Merriman, might decide the question of ability of the contestants, it would not necessarily settle or explain the principle at issue.

---

* Cons. Engr., Minneapolis, Minn.

As in the case with the beam, the truss frame is called on to support the moment tending to elongate the bottom chord and shorten the top chord horizontally, and the shear or cross-breaking vertical force. The total weight of the frame is the sum of the weights of the members designed to support these different forces added to the weight of the floor system and lateral bracing.

Simple Bridges vs. Continuous Bridges—Parallel Chords.—With parallel chords, the cross-section and the weight of the chord decrease as the depth of the frame increases, and, conversely, the weight of the web increases as the depth increases. Therefore, in such a frame, the economic weight approaches a minimum when the weight of the web equals the weight of the chords, and this relation holds true whether the frame is continuous or simple.

Economic Depth Greater for Simple Span.—Continuity renders the live load shears resisted by the webbing somewhat greater than those in the simple span. Therefore, the web of the continuous span would be heavier than that for the simple span, but the maximum moments are smaller for the continuous span. If it is a through span with inclined end posts, the chord length is shorter in the simple span than in the continuous span and if, for this reason, the chords were assumed to be equal in weight with trusses of the same depth, then, because of the difference in the webbing, the economic depth of the simple span would of necessity be greater. Parallel chord bridges, however, are not economical for even moderately long spans and, for these trusses of variable depth, there is a more radical difference in the relative economic depth than for the case of parallel chord bridges of the two respective types.

Economic Depth—Non-Parallel Chords.—Simple Spans.—The reverse law of summation of shears and moments is most favorable to the simple span. Thus, shear increases from mid-span to the support, and applied moment increases from the support to mid-span. Accordingly, the simple span truss may be made deep at the center and of small depth at the end. With the greater depth at the center where the shear is the least, the web members, although long, are light. With decreased depth toward the support, the heavy web members are short, so that by this arrangement the weight of the web becomes greatly reduced over the case of parallel chords. Moreover, because of the inclination of the chord, the inclined chord functions in a dual capacity, resisting both shear and moment, still further reducing the weight of the web which, with the parallel chords, was one-half the total truss weight.

Contrast this reverse law of summation, so helpful to the economy of the simple span, with the conditions which are unfavorable in the continuous span. The moment is greatest numerically over the support where the shear is greatest and to obtain depth to resist this moment, heavy web members become long in place of the correspondingly short members of the simple span resisting the same shear. The economic depth for the simple span thus becomes from 20 to 30% greater than for the continuous truss for more moderate spans with reduction, in view of overturning moment of wind for the longer spans.

As, with economic height, the web of the simple span would reduce to about one-third the weight of the continuous type, and the chord lengths would be less, the conclusion appears inevitable that instead of the economy claimed for the continuous frame, it is generally lacking in economy to the extent of 30 to 35%, as compared with the simple truss frame for moderate spans. The question of the economic frame or truss is only a part of the problem of economic bridge design. The lowest total cost is the object that the progressive engineer strives to attain, but rarely achieves. As he approaches it from a consideration of the minimum truss weight, other factors appear, which are antagonistic to the triangulation he has fixed on, for truss economy. It may be said that although the economic theory of design may appear to be very simple in theoretical works on bridges, in practice, it is otherwise.

The truss triangulation which gives apparently the least weight for the truss, commonly gives difficulty in obtaining desirable panel lengths for the floor system; the stringers are either too long or too short; there is always something that does not fit, and the resulting solution is a series of compromises to approximate the desired end. Again, in the longer span bridge, if the depth is economic for vertical loading, it will be ill proportioned to resist the overturning moment of severe storms, unless the floor is wide and the work heavy.

Viewing the Sciotoville Bridge as a continuous truss span, the writer believes Mr. Lindenthal has approached closely to the proper economic depth. His view that the steel weight is 20% less than that of a properly designed simple span is mistaken, and the writer would substantiate his view by the submission of Fig. 25, which shows the proportions of the Sciotoville Bridge, together with a suggested form of simple truss span which, approximate computation indicates, would be lighter than the continuous design by the margin noted. The moment over the support and the moment at the center of the continuous and simple span design would be equal. As the simple span is about 10% deeper, the maximum cross-section of the chord will be nine-tenths as great in the simple span. As the simple span design is such that advantage is taken of the reverse summation of shear and moment to an extent impossible in the continuous span and, because of the greater inclination of the chord members in the latter, the web proper would be about one-third of the weight of the web of the continuous span and, again, the total weight of the chords would be less, because of the shorter length, and it would appear that, as far as the truss frame is concerned, there should be a difference of at least 35% in favor of the simple span as against the continuous frame.

Consider the floor system: As the panel length is shorter, the stringer moments will be approximately six-tenths as great for the simple span, and the dead load moment would be one-half as great. The floor-beams would be increased in number, but reduced in cross-section. The number of main joints in the truss would be few, with longer members to handle, thereby reducing details. Wide variation in the make-up of the top chord and end post of the continuous type would be avoided, and simplification of details would result from greater uniformity of sections. The breaking truss to stiffen the floor-beams laterally would be eliminated automatically and the lateral system reduced in weight by connecting the laterals at the center of alternate floor-beams. Lateral stiffness would be secured by full triangulated

28 @ 27' 7½" = 773' 6"
20 @ 38' 9" = 775'
1550' C. to C. of End Bearings
129' 2"
103' 4"
20 @ 38' 9" = 775'
28 @ 27' 7½" = 773' 6"
141' 0"
131' 6"
103' 0"
55' 6"
77' 6"

FIG. 25

bracing on a smaller number of planes than the portal bracing used, with a corresponding reduction in weight. A batter of the trusses of 5 ft. from the vertical in the total height would add to the lateral rigidity and permit some reduction of section otherwise required for top lateral and chord bracing. The number of members which perform no work as truss members, other than supporting the dead weight of the chords, would be greatly reduced in the suggested simple span.

The simple truss design suggested might come somewhat outside the pale of precedent which Mr. Lindenthal commends in the work of the early builder, but, because it is practical, it should not be considered a criticism of the serviceable and excellent structure described in this interesting paper. Approach to the goal of ultimate economy, for which all engineers are working and may never expect to reach, must create precedent.

Erection.—In order not to exceed the cost of erection of the continuous bridge, the method of erection of the simple span would likewise need to follow somewhat unprecedented lines. The fact that the river bottom is of sound rock would make the problem much simpler than if dealing with the Missouri River where there is from 50 to 60 ft. of shifting silt. Leaving a wide channel under one span, as was done in the erection of the Sciotoville Bridge, would necessitate the cantilever erection of three main panels of one span, which would not appear to present any especial difficulty.

Although opinion may differ on the relative cost of erection of the two types, of the quantity of steel required for the same specified working stresses, any difference of opinion is readily solved by working out the designs and calculating the weights, if the approximate solution by the application of mechanical principles is considered open to question.

Many American engineers have made as conscientious an effort as Mr. Lindenthal to design continuous truss bridges economically, impelled thereto by the mistaken assumption of some proportionate economy comparable to the divergence of the moment areas of the two respective types, only to realize, more or less, the heavy handicap imposed by the reverse law of summation of shear and moment favorable to the simple span and unfavorable to the continuous truss. In the comparison of the cantilever and continuous type, the moment areas approach identity for comparable depth and section over and between supports, and, therefore, it would appear that the discredit Mr. Lindenthal would cast on the cantilever truss should be focused rather on a degenerate section typified by all web and no flange, toward which the make-up of the compression members in many designs has carelessly approached and, in the unfortunate instance of the cantilever, the ultimate danger line had been passed.

In the foregoing discussion, the consideration of the web as that part of the truss between the external chords is a departure from the comparison of the equality of the web and chords in the parallel truss bridge. It would seem, therefore, that the inclined chord or end post might be apportioned as web or chord as the sine and cosine, respectively, of its angle of inclination. On this basis, a fair mathematical apportionment might be realized, an equalization from which reasonably satisfactory conclusions could be drawn as to the economic depth in the types under consideration. Because there is very little inclination in the chords of the continuous bridge, the apportionment suggested immediately indicates greater economy, as the hypothenuse of the triangle is less than the length of the other two sides, the sine and cosine of the angle of inclination, another viewpoint is had, from which comparisons and conclusions may logically be drawn.

The theory of economic truss proportions is too complicated to be embodied in simple mathematical form. The suggestion of Mr. Bender, that the best method of comparison of any two designs of similar types, without going into details too completely, was by multiplying the length of the members by the maximum stress thereon, and the sum total of the products should be a minimum for the economic frame, appears in the light of a good approximation, providing the uniformity of sections and such lack of wide variation as occur in the change of sign and intensity of the chord stress presented by the continuous truss, in comparison with the constant kind of stress and relative uniformity presented by the arched chord simple span. The wide variation in the stress of the chords of the Sciotoville Bridge undoubtedly accounts for the uneconomical thickness of the webs of the sections, their lack of breadth in proportion to the height of the truss, and that apparently unavoidable compromise in the make-up from which the designer would be free in treating the simple truss span.

If engineers are to consider the Sciotoville example as a riveted truss of unprecedented dimensions, they should not forget the old Forth Bridge with clear spans of 1 700 ft., and its riveted construction throughout. Riveted construction in simple span design has the inherent advantage that rolled shapes which make up the members can be secured in very long lengths and so incorporated in the structure, reducing the number of joints necessitated by pin-connected chords, because of the facilities for fabrication and the necessity of making the joints at panel points for the proper connection of the laterals, which is immaterial when one is dealing with riveted sections.

The suggestion of a slight inclination of the truss might be objected to on the ground of difficult lateral connections, but this objection is more imaginary than real, as was demonstrated in the St. Croix River Arch.* Like Mr. Lindenthal, the writer appreciates the added advantage from the standpoint of stiffness which some indeterminate structures present, but this may be secured with the indeterminateness of the frame largely reduced for temperature and settlement, as was done in the structure noted. The adoption of the riveted sections throughout in the simple span permits partial erection as a cantilever, adapting the design to conditions under which the pin-connected frame is at a disadvantage.

It is generally assumed that the continuous bridge has some inherent advantages from the standpoint of stiffness, as compared with the simple span. If, however, the economic forms, and maximum deflections are compared, this will not be found to be true, as maximum deflection with the continuous bridge occurs when some of the spans are loaded and others un-

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 1.

loaded, and with its smaller depth at mid-span, with partial restraint only at the support, and with its smaller chord section, the maximum deflection will be found to be greater than with the economically designed simple span. The difference, although in favor of the simple span, is insufficient to offset material economy did that, in fact, pertain to the continuous bridge.

Diverse Laws of Economic Chord Inclination in Simple and Continuous Spans.—As the apportionment of the efficiency of inclination of the inclined chord in the simple span is as the cosine of its angle of inclination in resisting horizontal deformation of moment, and the sine of its angle of inclination in resisting the vertical deformation of shear, its efficiency in resisting shear and moment is as the secant of its angle of inclination is to unity, compared with horizontal chords on a unit weight basis.

Because of the reverse sign of the bending moment in the cantilever parts of the continuous span, there is a reversal in this law of economic efficiency. Take, for example, the member of U_{18} to U_{14}, in the author's continuous bridge design. For negative moment, its efficiency in resisting horizontal moment deformation is as the cosine of its angle of inclination. Its efficiency in resisting vertical shear is likewise as the sine of its inclination, but it carries that shear vertically a longer distance upward from the point of reaction, L_{20}, thereby increasing the material necessary to support it, instead of decreasing the material necessary to support the shear, as is the case with the inclined chord where the moment is positive. Accordingly, the cross-section of U_{18}–U_{14} presents a total efficiency in resisting combined shear and moment of an amount equal to the cosine of the angle of inclination instead of the secant of the angle of inclination, as is the case with the inclined chord of the arched simple truss span.

It is true that this inclination reduces the section required in the member, U_{18}–U_{18}, so that the economies of inclination are the algebraic sum of the plus and minus savings instead of the consecutive positive summation, as with inclined chords in the simple truss span.

The Latitude of Compromise of the Divergent Economic Requirements of Floor and Truss Panels Unfavorable to the Continuous Span.—In the foregoing discussion, it has been noted that truss triangulation frequently conflicts with economic floor paneling. Thus, comparing from the standpoint of the simple span suggested, the bending moment of the four lines of stringers is 70% greater in Mr. Lindenthal's continuous truss than in the simple truss diagram suggested by the writer. The dead load bending moment is about 100% greater; whereas, conversely, beam shear and bending moment of all the beams is approximately 10% greater for the simple span than for the continuous span partly offset by the greater dimensions laterally of the chords which would naturally be used. Accordingly, the range for compromise adjustment is the difference between one clear dimension for the simple span against two shorter dimensions, that is, the cantilever arm and the suspended span in the continuous truss of two spans, or three, if the span is an intermediate one, with an economic location for the point of inflection inharmonious with favorable panels for either cantilever, suspended span, or floor.

This is a practical disadvantage of no small importance, which should not be overlooked in a theoretical survey of the two types of construction. General experience with continuous draw-spans gives the practical engineer a better basis for a comparison of the stiffness of two-span continuous structures with single span structures of the dimension of the length of the arm, than the relations of bending and moment forces presented in works on bridge design, and from this experience conclusions regarding stiffness are diametrically opposed to those customarily drawn from incomplete theory of resistance based on moment and neglecting shear.

Among engineers, experienced in the art of bridge construction, few are egotistical enough to assume ability to design the most economical frame possible to devise, yet it would seem that a thorough discussion of theoretical principles should lead to less divergence of opinion. With theoretical treatises on continuous bridge construction estimating from moment distribution an economy of 25 to 40% in the material required for the continuous truss bridge, and practical experience and conscientious design apparently indicating a lack of economy of 35% for a two-span continuous structure to 25% for a four-span continuous structure, from consideration of essential provision for combined shear and moment and the relations indicated by the constant of the moment magnitude equations, as well as numerical values of the three-moment equations, there appears indeed ample room for harmonization of book theory and practical experience as it is viewed by those who take the negative side of the economic question and ample room for explanation on the part of those who assume the affirmative side. Undoubtedly, the author's closing discussion in the affirmative will be interesting and instructive.

In closing this discussion, the writer thanks Mr. Lindenthal for the presentation of a most interesting and valuable paper, which skillfully treats of many practical questions and discloses meritorious details in advance of current practice. Preference for the continuous bridge of moderate span differs from the majority opinion of American bridge engineers, because of lack of demonstrated economy on a scientific mathematical or design basis.

T. Kennard Thomson,* M. Am. Soc. C. E.—Reference has been made to the danger of placing falsework in the Ohio River, which reference is fully appreciated by the speaker, as he was Engineer of Bridges for the Ohio River Division of the Norfolk and Western Railroad when, in 1890-91, the Kenova Bridge was built a few miles above the site of the Sciotoville Bridge. At Kenova, the records showed a low-water depth of 6 ft. and a high-water depth of 106 ft.

The author has called attention to the desirability of rock foundations for continuous girders. Any departure from this fundamental should only be adopted after the most careful consideration. For example, a sufficient number of piles might be driven to carry safely several times the designed load; but unexpected contingencies may divert a river in such a manner that these foundations would be seriously imperiled by undermining. Again, the teredo, which had never been known in the locality before, might obtain access to the piles.

* Cons. Engr., New York City.

Some years ago, it would have been considered safe to cut piles off 30 or 40 ft. under water, as the teredo was only supposed to find access to the piles between high and low water. About twelve years ago, however, the speaker was retained to report on a highway bridge at Fall River, Mass., where one end of a pier had dropped 2 ft. in one night. A pile head was cut off and brought up with live teredo and also limnoria, both actively boring.

The piles had been cut off 30 or 35 ft. below the surface of the water and 4 ft. of timber grillage, carrying masonry pier, had been sunk on top of these piles. The bridge had been completed just two years prior to the collapse of the pier. As the piles were cut off at such great depth, and as there was a sewer discharging within 150 ft. of the pier, it would have been considered safe against the ravages of the teredo.

The speaker, however, has never been able to satisfy himself as to whether the teredo and the limnoria obtained access to the piles after they were driven, or while they were floating on the surface of the water before being driven. In any event, great care should be exercised against allowing piles to float in teredo-infested water before driving.

As the Fall River Bridge did not have continuous trusses, no serious damage was done. A coffer-dam was built around the pier and concrete was forced under the grillage around the old pile-caps and brought up outside the pier, above the timber grillage line, to form a new support for the bridge and to prevent any further damage by the teredo. The steelwork was then jacked back to its original position on a new bridge seat.

Charles Evan Fowler,* M. Am. Soc. C. E. (by letter).—The present era of bridge building may well be termed the long-span bridge era, as it is now possible to span wide rivers, estuaries, or canyons more scientifically and economically than at any past, or, perhaps, any relatively near future, period. The Sciotoville Bridge is a striking example as to what may be accomplished by the use of continuous bridges. It is the longest of that type ever constructed and now gives to America the proud distinction of having the longest spans for every type of bridge construction, namely, the Sciotoville Continuous Bridge, the Hell Gate Arch, the Quebec Cantilever, the Williamsburgh Suspension, the Metropolis Simple Truss Span, and the Willamette River Draw-Bridge.

The piers of the Sciotoville Bridge are illustrative as to what should be done in the reinforcing of monolithic concrete piers, not only for the prevention of cracks, but as providing a strong protective shell for resisting the impact from ice, logs, scows, or collisions from steamboats. For this purpose, the writer's practice has been to use wire cloth, placed 3 or 4 in. inside the surface of the pier, and it is economical and effective. The care exercised in obtaining a proper foundation for these piers is to be commended, as it is a fundamental and vital matter in the building of long-span bridges.

The appearance of the bridge is peculiarly pleasing when one considers that it is a two-span structure, with outlines formed by straight lines. This satisfying feature is due to proper depths of trusses and a strictly symmetrical struc-

* Cons. Engr., New York City.

ture. The system of trussing adopted is, also, without question the best that could have been selected and one well adapted to meet the problems that arose in the method of erection. These results were evidently achieved as a result of a trained judgment and careful investigation. It is no surprise, therefore, to know the final outcome as to the economy and efficiency of the structure.

The unit stresses used in making the design should receive careful study by the Special Committee on Specifications for Bridge Design and Construction, and all engineers having to do with the design of steel bridges of any magnitude, as engineers can no longer be so wasteful of the resources of Nature as they have been in the past. The loading adopted undoubtedly will prove heavy enough for the future, as the uniform train load was the governing one for the main truss members, and with the margin allowed inside the elastic limit of the material, the floor and primary truss members will probably never be wanting in strength. The use of riveted construction throughout is to be commended, as the structure in consequence thereof is much stiffer and more lasting, thus nullifying the slight decrease in cost that the use of eye-bars would have effected.

The salient points of this structure, as well as of this type of bridge, have been so well covered in the paper that the writer will not comment on them, but engineers should study them carefully and endeavor to absorb those larger problems of design which have been solved so well.

J. E. Greiner,* M. Am. Soc. C. E. (by letter).—The author mentions those early engineers who had the "genius that originates as distinguished from routine which merely imitates." It may be said also that in every structure of importance designed by Mr. Lindenthal there is evidence of this genius which originates. Each of his structures is practically a new creation as compared with the routine and stereotyped bridges throughout the United States.

The Sciotoville Bridge is no exception. It is a daring and handsome structure, decidedly "Lindenthalic" in all its features, and designed to carry Cooper's E-60 loading. No one will question its strength or its adequacy for any load that will ever pass over it, but one may well wonder whether this bridge designed for E-60 loading can be rated as an E-60 structure. The customary rating of railroad bridges is based on the ratio of the working stress to the ultimate strength, which ratio according to the American Railway Engineering Association specifications is 16:60, or 0.267. The ratio used by Mr. Lindenthal is 20:66, or 0.303, which is 13.5 higher than the standard and places this structure in the E-53 and not in the E-60 class.

The author's compression as well as his impact formulas are quite at variance with general practice. He may be right, and all other engineers may be wrong. It is admitted that working stresses and column and impact formulas always have invited juggling, and probably always will be subjects for the engineer's dicta when he has such a prerogative. There are, of course, standards for these stresses and formulas, but no one would expect Mr. Lindenthal to give much consideration to such standards. He establishes his own standards.

* Cons. Engr., Baltimore, Md.

If he is correct in rating his Sciotoville Bridge as an E-60 structure, then all the railroad bridges built for such a loading in accordance with the American Railway Engineering Association specifications are E-68 bridges, and if the Association's E-60 bridges are really such, then the Sciotoville Bridge is only an E-53 bridge.

D. B. STEINMAN,* M. AM. SOC. C. E. (by letter).—The continuous truss is an excellent bridge type, offering decided advantages (under suitable conditions) over all other forms of construction. Its general adoption for fixed spans has long been retarded by prejudices based on erroneous notions; but the successful execution of several notable examples in the last few years (Sciotoville, 1917; Allegheny River, 1918; Nelson River, 1918; Cincinnati, 1922) has served to dispel these prejudices, and the continuous truss has become established as an important type in American bridge practice.

In comparison with simple spans, the continuous bridge offers the same advantages as the cantilever, namely:

1.—Economy of material.

2.—Suitability for erection of one or more spans without falsework.

In addition, the continuous bridge is superior to the cantilever in

3.—Rigidity under traffic.

4.—Less abrupt stress changes under traffic.

5.—Elimination of expensive and troublesome hinge details.

6.—Less extra material or hazard in erection.

7.—Safety of the completed structure.

The results of economic comparisons between continuous and simple spans will be materially affected by

1.—The length of spans.

2.—The system of web bracing used.

3.—The specification provision for reversal of stresses.

The relative economy of the continuous type increases with the length of span or, in general, with the ratio of dead load to live load. It is wrong to draw a general conclusion against the economy of continuous bridges based on a comparison for small spans (as Bender did in his book "Continuous Bridges" in 1876).

Furthermore, for a correct economic comparison, the most suitable system of web bracing should be assumed for each respective type; a Pratt or Petit system may be most economical for the simple spans, but a Warren system will generally yield the best results for the continuous truss.

Finally, the results of the economic comparison may be upset by the unscientific practice of imposing a stringent reversal clause; these reversal clauses as applied to the proportioning of main sections are relics of exploded fatigue theories, and should find no place in modern specifications for long span bridges.

A proper comparison with corresponding simple spans will generally show a substantial saving of material in favor of the continuous structure. It is

* Cons. Engr., New York City.

difficult or impossible to arrive at reliable conclusions in this question from abstract argument or theoretical considerations.

For purposes of comparison, the writer has made careful estimates of the weight of simple spans of 775 ft., based on detailed calculations of all stresses and sections. The live load was E-60 (on two tracks), and designs were made both for Lindenthal's specifications and for the American Railway Engineering Association specifications. The results were as follows:

Weight of two trusses, in pounds per linear foot:

1 550-ft. Sciotoville Bridge	12 880 = 100%
775-ft. Simple span (Lindenthal's specifications)	15 300 = 119%
775-ft. Simple span (A. R. E. A. specifications)	17 650 = 137%

Total steelwork, in pounds per linear foot:

1 550-ft. Sciotoville Bridge	17 100 = 100%
775-ft. Simple span (Lindenthal's specifications)	20 300 = 119%
775-ft. Simple span (A. R. E. A. specifications)	23 000 = 134%

The foregoing weights of simple spans do not include the extra metal which would have been required for the erection of the channel span by the cantilever method.

These figures bear out Mr. Lindenthal's claim of a saving of 20% in the weight of the steel by the adoption of the continuous type for the Sciotoville Bridge.

According to comparative studies made by Winkler, the saving for continuous bridges of two, three, and four spans is 16, 19, and 21%, respectively, when the span length is about 300 ft.; and 20, 24, and 28%, respectively, when the span length is about 500 ft.

Generally, however, the economy of material is a secondary consideration in the adoption of the continuous type, the deciding advantages being the convenience of cantilever erection and the increased stiffness of the structure.

The following conditions are particularly favorable to the economy and efficiency of the continuous bridge in comparison with other types:

1.—Long spans.
2.—High ratio of dead to live load.
3.—Good foundations.
4.—Piers of moderate height.
5.—Moderate truss depth.
6.—Spans approximately equal.
7.—Cantilever erection.

When the spans are long, the other requirements assume minor importance.

Both the economy and rigidity of the continuous type increase with the number of spans, but the gain beyond three or four spans is insignificant. Moreover, a larger number of spans would create difficulty in providing for expansion on account of the great length between expansion joints. Another objection is the greater number of supports at which jacking operations would be required during erection for the adjustment of the reactions. (In a two-span bridge, only one support out of three requires jacking adjustment; in a five-span bridge, four supports out of six would require jacking, and the operation would be more complicated.) For these considerations, the number of spans in a continuous group is generally limited to three or four.

In a two-span bridge, the requirements of economy as well as of appearance are best satisfied by making the two spans equal in length. In bridges of three or more spans, a symmetrical layout is also desirable for appearance and for shop economy. In a three-span bridge, the economic ratio of spans is approximately 7 : 8 : 7; but considerable variations from these proportions will not materially affect the economy. In a four-span bridge, the economic ratio of spans is approximately 3 : 4 : 4 : 3; but these proportions may also be varied considerably without materially affecting the economy.

In many cases, the span arrangement is determined by natural conditions of the crossing (or by the desire to utilize existing piers) rather than by economic or esthetic considerations.

The effect of possible pier settlement on the stresses in continuous bridges has been grossly over-estimated by former writers on the subject. In the case of the Sciotoville Bridge, according to the writer's computations, an excess settlement of 1 in. at the end support would produce a relief in the end reaction amounting to 15 000 lb. This is only 0.6% of the dead load reaction, or 0.3% of the total $(D + L + I)$ reaction. The simultaneous increase in the middle reaction would be 30 000 lb., which is only 0.3% of the dead load reaction, or 0.2% of the total $(D + L + I)$ reaction at the middle support. An excess settlement of 1 in. at the middle support would produce effects equal to double those just given, but opposite in sign. The middle reaction would be relieved 60 000 lb., or less than 0.4% of the total $(D + L + I)$ reaction at the middle support. It is evident from these figures that any ordinary settlement of the piers would affect the stresses in the structure to so small an extent as to be negligible.

A former objection to the continuous bridge was its static indeterminateness. With modern methods of design and construction, however, it is possible to know the exact stresses in a continuous structure for any given conditions; the uncertainties can be made as small as in simple spans and the extra labor of the computations is trifling in itself, as well as in comparison with the advantages to be derived.

The method of calculation developed for the Sciotoville Bridge was described by the writer in an article entitled "The Elastic Curve Applied to the Design of the Sciotoville Bridge".* The entire work of calculating the elastic curve by this method requires only 2 or 3 hours at the most. After the elastic curve is determined, the remainder of the design is essentially the same as for a simple structure.

In the case of the Sciotoville Bridge, three successive designs were made:

1.—Preliminary approximate design, treating the truss as a beam with constant moment of inertia.

2.—More exact design, allowing for the variation in moment of inertia but with the web members neglected.

3.—Final exact design, allowing for the variation in moment of inertia with the contributions of all the members included.

* *Engineering Record,* August 28th, 1915.

The sections obtained in the first approximation were used as a basis for the succeeding designs. The elastic ordinates for the three assumptions, also for the assumption of triangular variation of I, are compared in Table 2.

TABLE 2.—COMPARISON OF ELASTIC CURVES, SCIOTOVILLE BRIDGE.

Panel point (1)	$\frac{x}{l}$ (2)	Assumption I = constant (3)	Web members neglected (4)	All members included (5)	Assumption, triangular variation of I (6)
A 0	0	1.000	1.000	1.000	1.000
2	0.1	0.875	0.871	0.876	0.855
4	0.2	0.752	0.744	0.754	0.720
6	0.3	0.632	0.623	0.632	0.595
8	0.4	0.516	0.505	0.509	0.480
10	0.5	0.406	0.393	0.392	0.375
12	0.6	0.304	0.287	0.287	0.280
14	0.7	0.211	0.193	0.192	0.195
16	0.8	0.128	0.114	0.115	0.120
18	0.9	0.057	0.053	0.055	0.055
B 20	1.0	0	0	0	0
Area AB		4.381	4.283	4.312	4.175
Area BC		0.619	0.717	0.688	0.825
Sum of areas		5.000	5.000	5.000	5.000

Table 2 is useful inasmuch as its values can be adopted for the preliminary designs of other structures, thereby saving time and labor. For a structure similar in general outline to the Sciotoville Bridge, the values in Column 5 should be used. For girders and trusses with parallel chords, the values in Column 3 would probably be a closer approximation.

The assumption of triangular variation of moment of inertia (assuming I to vary as the ordinates of a triangle from zero at the ends to a maximum at the intermediate support) may also be recommended as a basis for preliminary or approximate design. This assumption will generally represent the actual variation of I in a two-span continuous truss about as well as the assumption of constant, I; moreover, it yields results of striking simplicity. The elastic curves are parabolas, and other convenient relations obtain. Instead of the more laborious "Theorem of Three Moments", one is able to use the following simple formulas for the moment over the middle support. For concentrated loads:

$$M_2 = -\frac{1}{2} \Sigma M$$

in which M is the simple-beam bending moment produced by each concentrated load at its own point of application. For continuous or concentrated loads:

$$M_2 = -\frac{1}{l} (A_1 + A_2)$$

in which A_1 is the area of the simple-beam moment diagram in one span, and A_2 is the area in the other span.

In making preliminary assumptions as to distribution of dead load, variations of sections of truss members, and variations of moment of inertia, the corresponding distribution graphs for the Sciotoville Bridge may be of assistance. With this object in view, the writer has prepared Fig. 26 which shows, by graphs, the distribution of material in the Sciotoville spans.

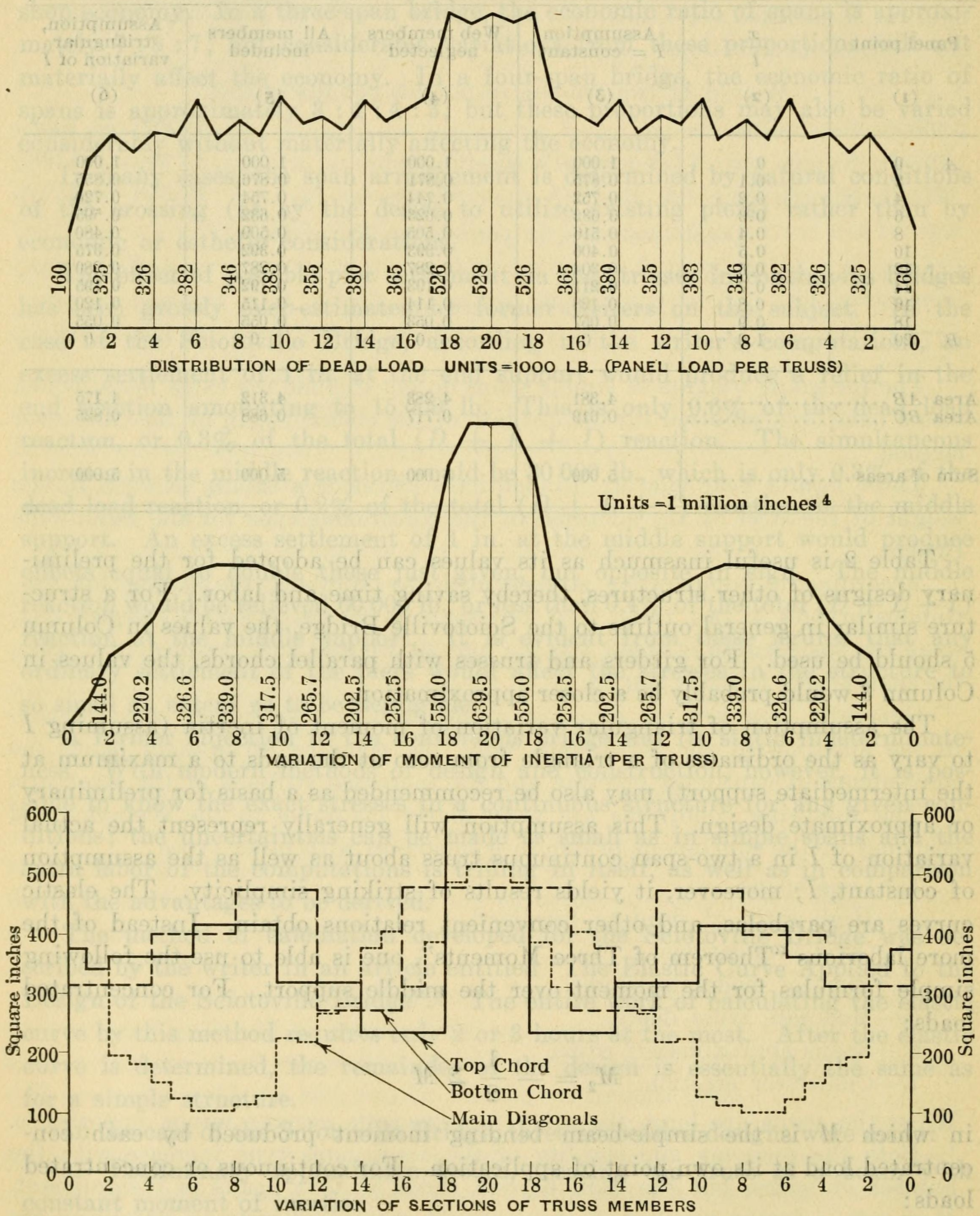


FIG. 26.

Certain improvements in distribution of material, with possible saving in total weight, may be secured by intentionally building the middle support lower than the end supports by a predetermined amount. The excess of middle reaction over end reactions can thus be somewhat relieved, and the greatest negative and positive moments can be equalized. In a two-span girder of equal spans and constant, I, it will be advantageous to lower the middle support an amount defined by:

$$D = (1.9\,w + p)\,\frac{l^4}{145\,E\,I}$$

in which w = the dead load, and p = the live load per linear foot. This will reduce the negative moment over the middle support from about 16 to 31% (depending on the ratio of $w : p$) and will increase the maximum positive moment within the span to an equal value. For the Sciotoville Bridge, the necessary camber or lowering corresponding to this formula would be about 14 in. This would have reduced the maximum required section (over the middle support) from 596 to 520 sq. in., a reduction of 76 sq. in.; and it would have increased the mid-span section from 474 to 520 sq. in., an increase of 46 sq. in. Such equalization of maximum sections, with a reduction of 13% in the extreme heaviest sections, is an advantage worth considering in future designs.

In common with other indeterminate structures, the continuous bridge offers the possibility of varying the distribution of stress by adjustment. In the case of the Niagara Railway Arch Bridge,* it will be recalled how Charles Evan Fowler, M. Am. Soc. C. E., secured a favorable readjustment of dead load stresses by changing the thickness of the shims at mid-span. Similarly, in a continuous bridge, the dead load stresses may be redistributed by the predetermined raising or lowering of one or more supports. This may be done during erection, in order to reduce the maximum required sections, or in order to secure a more favorable distribution of material to resist erection stresses; or the jacking operation may be resorted to after the bridge has been standing for some time in order to relieve members which, for any reason, may be found to be overstressed.

Three notable innovations in the design of the Sciotoville Bridge were as follows:

1.—The adoption of U-shaped floor-beams, designed as inverted arches.

2.—The adoption of lattice sway-frames, instead of the usual rigid cross-frames.

3.—The initial bending of the members to neutralize the secondary stresses.

The adoption of the U-type of floor-beam is probably without precedent in practical bridge design, although the combination of conditions warranting its use may easily occur in any long span structure. The application of the U-shaped floor-beam is indicated in any structure in which there obtains limited floor depth with excessive width between trusses. This combination of conditions renders the straight type of floor-beam uneconomical; and the second condition, from considerations of clearance, renders the U-type practicable. Both these conditions obtained in the Sciotoville Bridge. In this structure, in order to secure adequate transverse stiffness against wind and lateral forces, the length of floor-beam, or distance, center to center of trusses, was made 38 ft. 9 in., or one-twentieth of the span. This is nearly 10 ft. in excess of the width required for train clearance, this excess of clearance being a

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1919.

consequence of the unusual length of span for a bridge of only two tracks. The excessive distance between the truss and stringers results in high bending moments, so that a floor-beam depth of 8.5 or 9.0 ft. would be desirable for economy. Actually, a depth of only 7.5 ft. was available. Although an additional 12 or 18 in. of height would have given lighter floor-beams and stringers, it could not be secured except at the expense of raising and lengthening a 40-ft. fill nearly 2 miles in length at the Kentucky approach. To overcome the disadvantage caused by this limitation of depth, the U-shaped floor-beam was devised. This design yields the following advantages:

1.—A material saving in the total weight of floor-beam.

2.—A considerable reduction in the bending moments, thereby avoiding some of the difficulties of providing extra heavy flange sections.

3.—Increased transverse stiffness of the bridge, the U-frame becoming an addition to the sway-bracing.

4.—Elimination of the secondary stresses in posts or hangers caused by the deflection of straight floor-beams.

5.—Elimination of the secondary stresses in straight floor-beams due to their partial constraint at the ends.

6.—Increased resistance of the structure to vibration.

In the Sciotoville Bridge, the adoption of the U-shaped design, according to the original comparative estimate, reduced the maximum live load moments by 39% and effected a saving of 9 000 lb. of steel in each of the thirty-nine intermediate floor-beams. This saving in weight may be offset somewhat by increased unit cost of the shopwork, but the other advantages alone would justify the adoption of the U-shaped floor-beam wherever adequate lateral clearance is available.

In analyzing the stresses in the U-frame, it is important to make proper allowance for the shortening of the bracing strut which takes up the horizontal thrust of the inverted arch.

The use of ordinary sway-bracing of the rigid type was avoided, in the design of the Sciotoville Bridge, on account of the high stresses which such bracing would suffer under one-sided loading of the bridge. Under the extreme case of one track of one span and the opposite track of the other span being fully loaded, there will be a difference of 2 in. in the mid-span deflections of the two trusses. To permit this unequal deflection without rupture, the usual sway-frames of rigid type were replaced by lattice-frames combining strength with elastic stiffness.

The scheme for neutralizing the high secondary stresses in the structure is another feature adding distinction to the Sciotoville Bridge. The necessary erection operations were calculated with unprecedented care and thoroughness, and the measurements of jacking forces and deflections in the various erection stages afforded an excellent check on the calculations.

The Sciotoville Bridge is a striking example of scientific design. It represents an unusually intensive application of engineering theory and resourcefulness to the determination of the most efficient disposition of the material of a structure, and to the rigorous advance planning of every step in the operations of fabrication and erection.

HENRY H. QUIMBY,* M. AM. SOC. C. E.—The speaker recalls that forty years ago the computers of a certain bridge company made simple spans of the two plate-girder spans of a railroad bridge because of want of confidence in their figures and fear of some indefinite consequences if the girders were made continuous over three supports.

Continuity of structure is an economical and advantageous principle of design and, if intelligently used, is practical and safe. In bridges of several arched spans, it has been applied with satisfactory results—in one case of five spans, the steelwork was both the reinforcement of the concrete arch ribs and provision for the erection of both the steel and the concrete without falsework. In another bridge that had a large number of rod-reinforced concrete arches, reinforcing rods were embedded over the piers as the tension members to constitute the first voussoirs cantilevers for carrying the forms for successive voussoirs. The rods were left exposed for a few inches, and after erection was complete they were cut by sawing. The tension was so great that each rod snapped apart when partly cut. In later bridges, both with structural shapes and screw rods as tension members over the piers, the ties were permitted to remain, as being not only harmless, but possibly contributory to stiffness as well as to stability.

In the Sciotoville Bridge, the effect on the stresses in the members, which the paper states would be produced by deformations that would attend certain settlements of piers, is surprisingly small in view of the great height of the trusses. As so much settlement can hardly be imagined, the continuity of truss is certainly justified.

The method of erection and the means adopted to minimize secondary stresses are interesting. Both must have been expensive, and some information regarding the cost of the work would be instructive. The plan of erection was dictated in a measure by considerations of safety under the attendant conditions of the site and was justified by its success. The method adopted for reducing secondary stresses was bold and scientific, but must have been very costly, and it is understood to have involved a great deal of heavy drifting and reaming to get the holes to admit the rivets. Can the author give any facts regarding these points, and any comparison of the extra cost of the work as done with the cost of providing additional section at the critical points of members affected by secondary stresses?

GUSTAV LINDENTHAL,† M. AM. SOC. C. E. (by letter).—The contention made by Mr. Turner, that simple girders may be designed with no more or, perhaps, with less steel than continuous girders, is based on the same incomplete grounds often asserted before. Short panels to reduce floor weights, great height between chords to reduce chord sections, and curved chords to reduce web stresses, are all long known devices for attaining economy. They can be used in both the simple and the continuous types, and also in the cantilever type. The question of economy can hardly be settled by a comparison of strain lengths in the several types. The experience of bridge designers during the

* Chf. Engr., Dept. of City Transit, Philadelphia, Pa.
† Cons. Engr., New York City.

last 30 or 40 years has fairly well established the best proportions for each type.

The theory of strain lengths originated by Charles Bender (who was personally known to the writer 45 years ago), seemed at the time a plausible basis for estimating and comparing the economy of bridge and roof trusses, fortified as it was by an elegant deduction from Clapeyron's theorem of deflections. With simple elements of loads and strains, it can be useful; for the correct dimensioning of bridge members, however, a greater number and variety of stresses have now to be considered than were thought of in Bender's day. To the stresses from dead and live load must be added the various combinations with effects of impact, traction stresses, wind pressure, secondary stresses, etc. The theory then ceases to be useful, especially so in bridge frames having heavy members with reversible stresses (tension and compression), as in continuous and cantilever trusses.

No fact on the strength of steel is more firmly established from practical tests and after much theoretical travail than that steel and wrought iron will safely resist without deterioration a nearly infinite number of variations and reversions of stresses, provided the unit stress remains within, say, two-thirds of the elastic limit. To take advantage of this fact in economic bridge designing, the stresses must be accurately known. This is not always feasible with live load stresses, some of which are dynamic, but are, nevertheless, assumed to behave statically like stresses from dead or other quiescent loads. In some members, impact stresses may exceed the statical stresses from live load of the same or opposite sign. The theory of strain length under all these conditions becomes too uncertain for the *a priori* comparison of economy in designs. The only reliable comparison in any given case is on the basis of fully worked out stress sheets, complete erection schedules, and estimates of cost.

If two spans of simple girders with a height of $\frac{1}{5.5}$ of the span, as proposed by Mr. Turner, should show greater economy, which is doubtful, even with alloy steel, then there is no reason why greater height could not also be used for the same purpose in continuous girders; but would it be good designing? Engineers are expected to build good bridges economically. That does not mean freak bridges under the pretense of scientific economy which too often may be false economy.

In this double-track bridge, the great simple-girder height would be nearly four times the width between girders. A train passing over the bridge would deflect one girder more than the other by several inches. That would throw out the top of the girder nearly four times as many inches. It would be a very wabbly structure, possibly a so-called economic design, but inferior as far as behavior under use is concerned. The inclining of the trusses, as suggested by Mr. Turner, would not improve it.

In the continuous girders, the greatest height (also the most weight) is over the middle pier, where the girders cannot deflect horizontally or vertically. The wind bracing between the chords is also continuous. The four continuous girders form a long rigid box of great stability, open at the ends, exposed to the least amount of vertical and horizontal oscillation. In other words, the continuous girders are in every way stiffer than the simple girders.

A floor with 28 panels per span, as proposed by Mr. Turner, in place of 20 panels, for the sake of saving an insignificant quantity of steel in the stringers, would require 40% more handling in the shops, with many thousand more field rivets, and 40% more field operations during the erection. The greater cost of field labor would cancel the inconsiderable saving in steel for stringers of shorter panels. In addition, the greater compactness of long panels is discarded. Owing to the lighter floor, the center of gravity of the single girder structure is higher and contributes another element to the wabbliness of the bridge. The top chord panels would be 50% longer than in the continuous girder and the cross-sections, therefore, 30% larger, thus effacing any economy from greater height of trusses.

The simple girders would require considerable extra material to be erected by the cantilever method over the large river opening needed during erection. It would also have required more time for shifting the falsework and another material yard on the Kentucky side, which was not available. Compact, short, well braced, compression members keep the chords rigidly in line, an advantage largely absent in the simple girder. The bridge is situated in a region subject to tornadoes which have wrecked one or more bridges over the Ohio River. The bracing of the continuous girders can be made better and stronger, and they are thus capable of offering greater resistance to wind pressure.

The gravamen of Mr. Greiner's discussion is that the capacity of the bridge is not really for an E-60 loading, but only for an E-53 or, say, 12% lighter loads, when based on the specification of the American Railway Engineering Association. The A. R. E. A. specifications are intended only for ordinary spans, not exceeding, say, 350 ft.; for longer spans they are wasteful in the trusses. This fact is confirmed by a comparison of a few sections as shown in Table 3, in which is given the sectional area of a number of typical members of the Sciotoville Bridge, as designed according to the writer's specifications in comparison with those required by the latest A. R. E. A. specifications and E-60 loading. Table 3 also gives a comparison of the strength of these members for the two specifications, based on the fact that the writer's specifications require steel at least 13% stronger than that called for by the A. R. E. A. specifications (minimum yield point, 35 000 against 30 000; ultimate strength, 62 000 against 55 000).

The statement by Mr. Greiner that the bridge has a live load rating of only E-53 is shown by Table 3 to be unjustified and is made without examination of the requirements of the writer's specifications other than the basic unit stress (20 000), which may mislead superficial readers.

In the floor system and floor hangers (as in short spans generally), the writer's specifications give sectional areas up to 25% in excess, and in strength of members up to 41% in excess of those required by the A. R. E. A. specifications. That makes the live load rating for the floor construction in this bridge really E-70 to E-84 on that specification. This is due principally to the greater impact allowance for short spans, resulting from the writer's formula for impact, which, as far as is known is the only impact formula deduced from well-known facts on the durability of steel under impact conditions. It is simple of application and gives rational results for the shortest and the longest spans alike. This advantage adheres also to the writer's specifications, or "Rules of Design", used in the Hell Gate, Sciotoville, and other bridges. These specifications give well-balanced working stresses, insofar as uniform durability of structure is concerned, for the shortest and the longest spans, which is not the case with the A. R. E. A. specifications.

TABLE 3.

Member.	Existing area, in square inches, Lindenthal design.	Area, in square inches, required by A. R. E. A. specification.	Percentage of excess area, Lindenthal specification over A. R. E. A. specification.	Percentage of excess strength, Lindenthal specification over A. R. E. A. specification.
Stringer:				
Tension flange, net	26.66	24.0	+11	+25
Compression flange, gross	30.58	29.4	+ 4	+17
Floor-beam:				
Tension flange, net	42.63	34.1	+25	+41
" " , gross	46.32	41.8	+11	+25
Trusses:				
Top chord:				
U 6-*U*10, gross	435	518	−16	− 5
*U*10-*U*14, gross	317	374	−15	− 4
*U*14-*U*18, net	198	187	+ 5	+18
*U*18-*U*20, net	501	579	−13	0
Bottom chord:				
L 4-*L* 8, net	403	404.1	− 0	0
*L*12-*L*16, net	230.1	255.1	−10	+ 2
Diagonals:				
M 3-*L* 4, net	149.0	154.6	− 3.7	9
L 4-*M* 5, gross	150	152.6	− 1.6	11
U 6-*M* 7, gross	104.5	140	−34	−26
L 8-*M* 9, net	93	124.0	−33	−25
*U*10-*M*11, gross	224	258.0	−13	0
*U*18-*M*19, gross	486.7	655.0	−25	−15
Main hanger, net	90	85.3	+ 6	+19
Sub-hanger, net	49	44.2	+11	+25

The sectional areas of the bottom chord (loaded chord) of the Sciotoville Bridge are nearly the same, because in these members the impact allowance is more nearly the same, for both specifications, but the reduction in areas which might result from the higher unit stresses of the writer's specifications, is offset by the justified greater allowance for lateral, wind, and brake forces, which may co-exist in these chords with the dead load, live load, and impact stresses.

The A. R. E. A. specifications would require some sections of the top chord and main web members, to be 34% larger, and in strength to be 26% greater. This would be plainly a waste of steel due to the low unit stresses allowed by the A. R. E. A. specifications (maximum 16 000 lb. per sq. in. in tension and 12 500 lb. per sq. in. in compression), which low limits are not justified in a bridge of this size. The waste is also in part due to the absurd requirement of the A. R. E. A. specifications that members subject to reversal of stress shall be proportioned for the greater stress plus 50% of the smaller stress.

The A. R.-E. A. specifications do not require sufficient metal for the floor structure and for short spans where it is necessary, because these structures are stressed to the violent maximum from nearly every train, and they require too much metal for the truss members of long spans, in which members, the maximum stresses are far from violent and rarely, if ever, occur.

The belief of Mr. Quimby that the method of reducing secondary stresses was costly and involved a great deal of heavy drifting and reaming in order to get the holes to admit rivets is not based on fact. There was no trouble or extra work on that account in the shop.

Although it is true that the bending of the members in the field, in order to make the rivet holes fit, required some special operations, the cost of such work was comparatively small and was fully justified by the advantage gained. In fact, the fabrication and erection cost of the bridge per pound of steel was remarkably low, and less than for pin-connected work, as proven by the bids received for both types of structures.

The discussion by Mr. Steinman is a welcome contribution on some details and features already mentioned, in the main, by the writer in his paper, omitting elaborations.

The conclusions by Professor Winkler, the German nestor of bridge mathematicians 45 years ago, mentioned by Mr. Steinman, on the economy of continuous girders, are fully confirmed in the Sciotoville Bridge. French and German engineers early showed much less hesitation in the use of that system than American engineers. Almost all those early European bridges are still in good condition, although railroad loads on them have increased, but not as much as in America.

Mr. Steinman's idea of reducing the height of trusses over the middle pier and increasing the height at mid-span, would not be, on the whole, advantageous or economical, as the questionable saving of metal in the chords would be outbalanced by increased sections and metal in the web members toward the ends of trusses. There would also be, in this case, the objection of greater erection stresses and too much deflection during erection, with greater secondary stresses in cantilevering.