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Continuous Trusses of Silicon Steel Feature New Allegheny River Bridge

Sharp Departure from Precedent in Bridge Design—Minimum Weight Desired for Economy and To Facilitate Cantilever Erection—Reactions Adjusted by Jacking—High-Strength Eyebars of New Kind

WHERE the Bessemer & Lake Erie R.R. crosses the Allegheny River a new double-track deck bridge proportioned for modern loading has just been built to replace a light single-track structure. The renewal presented a problem of unusual complication, in solving which the engineers were led to adopt a strikingly original solution.

The old bridge had simple spans. In the new bridge the six main spans, ranging from 272 to 520 ft., are continuous, arranged in two groups of three each. Through this sharp departure from prevailing practice the new bridge shares with its Ohio River contemporary at Sciotoville the distinction of reintroducing continuous construction into American practice in large-scale bridge work.

A special feature is a counterweight at the end of the southerly span, to counteract any uplift that might be caused by live-load on the adjoining span, 520 ft. long. The shore span was extended into the rock bluff, about 100 ft. back of the old abutment, to obtain sufficient anchor-span length, but a 350-ton concrete counterweight block was required in addition. The block is molded around the end sway-frame, and occupies the full width between trusses and the height from jacking girder to floorbeam.

The new bridge is made noteworthy also by its material. Silicon steel was used for the stiff members, while the eyebars—used for all tension members not subject to stress reversal—are of a new kind claimed to be equal to nickel-steel eyebars. The builders give no information as to the method of making these eyebars. Results of strength tests are quoted farther on.

Both the continuous construction and the choice of material were in large part due to the necessity for erecting the bridge by the cantilever method, which under the conditions governing the work required a design of minimum weight for the desired load capacity. Other difficulties of the case had contributory influence, however, and the fact that the adopted solution proved the cheapest was the controlling consideration.

Since the erection method was fixed, the relative costs of different types of bridge were practically expressed by their relative weights—after adding the extra section required for erection stresses—and therefore the weight economy of the continuous design meant corresponding gain.

The old bridge was built in 1896, for E40 loading, which then was heavier than existing railway loading. The Bessemer & Lake Erie transports large quantities of iron ore, in which class of traffic the greatest increase in train weights has occurred. Its present maximum train weights are in excess of E50. For some years the bridge had been overloaded, to the extent even that double-headed trains had to shift one engine to the rear before crossing. But the inadequate traffic capacity of the single track was equally hampering. In 1914, therefore, replacement was taken in hand. It was studied jointly by H. T. Porter, chief engineer of the railway, and C. G. E. Larsson, assistant chief engineer of the American Bridge Co.

Some of the governing conditions of the problem were

FIG. 1. NEW BRIDGE WILL BE ROLLED TO CENTRAL POSITION ON PIERS AFTER OLD BRIDGE IS DISMANTLED

Far half of pier is to be cut down after removal of old structure. Pier girders prevent concentration of load at edge of masonry. At the four main bearings the truss panel-point acts as upper shoe. Locking plates of rockers were removed when last span was being closed.

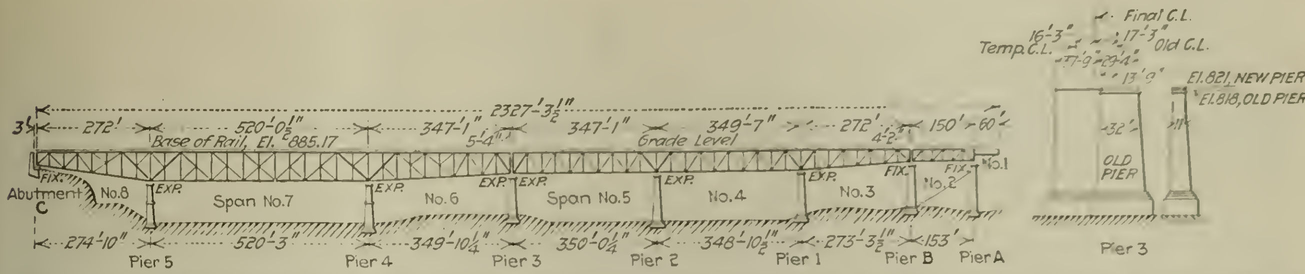


FIG. 2. DIAGRAM OF NEW CONTINUOUS-SPAN BRIDGE OVER ALLEGHENY RIVER BUILT FOR ORE-TRAIN TRAFFIC

the absolute necessity of maintaining traffic at all times; the importance of avoiding falsework in the river (as the Allegheny is considered dangerous to temporary timber work that must be maintained for a long period); the desirability of improving the profile of the road by raising the track 10 ft. at the north end of the bridge so as to make the grade level and eliminate a highway grade crossing; the existence at the north end of a 1200-ft. approach viaduct 130 ft. high; and the comparative weakness of the old piers in the direction of the track. It was thought at first that the bridge would have to be built on new alignment, but finally a practicable method of building on the existing alignment, with use of the old piers, was developed.

An essential feature of the solution was replacement of the north approach viaduct by a fill. This remarkable operation, in which two steel viaducts—the old one and a dumping viaduct built alongside—are buried, was described in *Engineering News* of Feb. 22, 1917, p. 322. The fill is not yet completed, delay having resulted from railway conditions during the last season and also through the occurrence of excessive lateral pressure of the fill on the viaduct.

For the river bridge, the replacement involved the following procedure: Widening the single-track piers on the old footings, which had been built of double-track width originally, and erecting the new bridge alongside the old one, since the piers as widened were long enough (with the help of pier girders) to accommodate both structures side by side; then transferring traffic to the new bridge and dismantling the old; and, finally, rolling the new bridge over into central position on the piers. The erection was begun at the two ends, by building the shore spans on falsework. All the other spans were cantilevered, and the work proceeded by continuous cantilever operation to closure at the middle of the 520-ft. span.

To avoid overstress in the pier masonry under the action of braking forces, the bearings on all the river piers were provided with expansion rollers, the two ends

FIG. 3. HIGH-STRENGTH STEEL AND CONTINUOUS CONSTRUCTION RESULT IN A STRUCTURE OF LIGHT APPEARANCE

Looking north from south bank. The span beyond the first pier is 520 ft. long. Designed for E75 loading, this double-track bridge is among the heaviest to be found in our railway system.

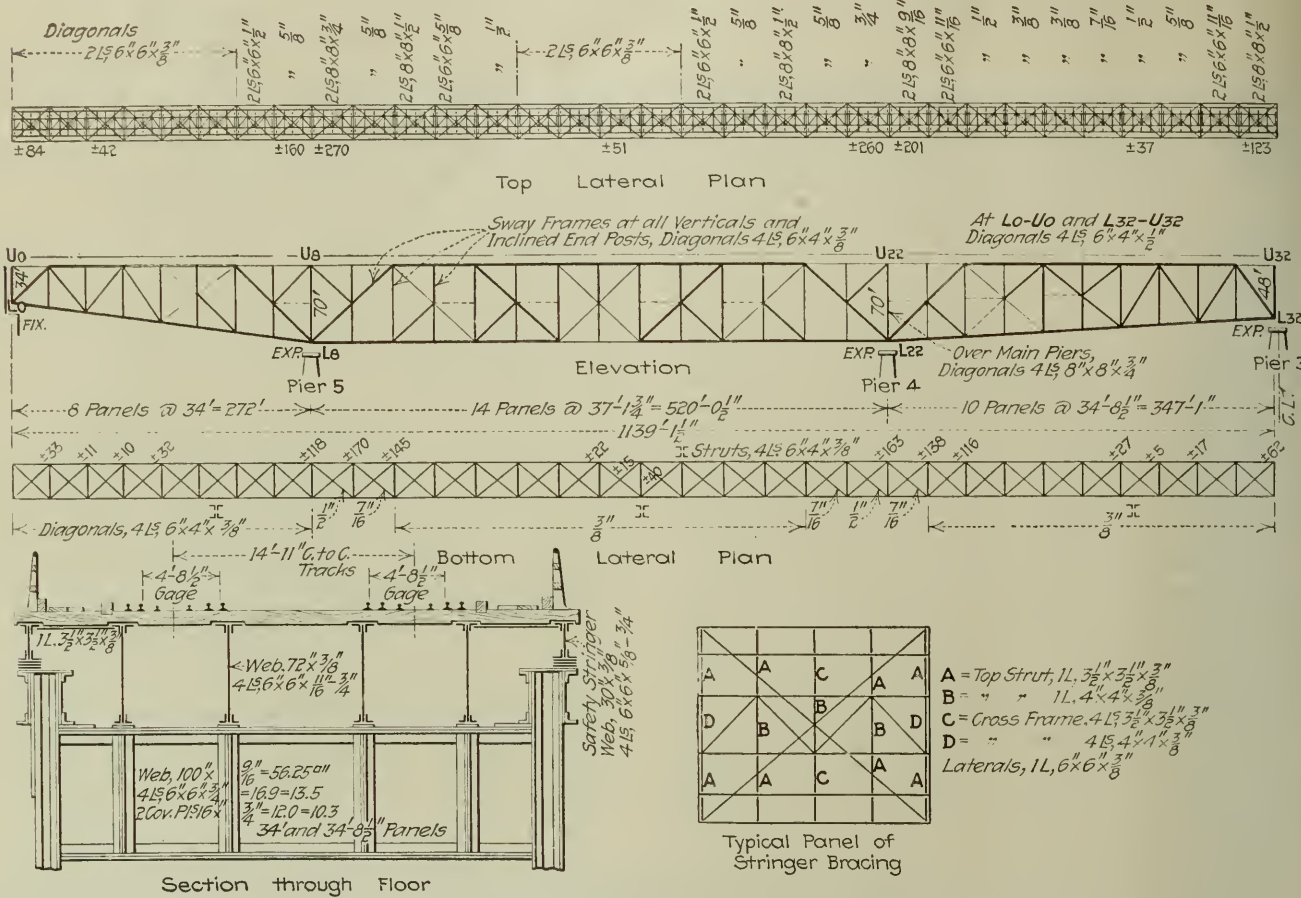


FIG. 4. CONDENSED STRESS-SHEET OF SOUTH-SPAN GROUP—ALL TENSION MEMBERS ARE EYEBARS

Number	Maximum Stresses: Dead	Live	Impact	Total	Erection	Material		Sectional Area, Gross	Net
Uo-U1	.....	.....	.....	.....	.....	2-15 in. channels, 33 lb.	Carbon steel	19.8	.....
U1-U3	—189	—1,175 674	—417 151	—2,099 954	—1,229 150	Cov. 36x5/8 2 Ls 6x4x3/4	2 Webs 36x7/8 2 Ls 6x6x3/4	114.1	.....
U-3-U6	52	—1,133 1,115	—402 250	—2,191 2,125	—1,359 650	Cov. 36x5/8 2 Ls 6x4x3/4	2 Webs 36x1 1/16 2 Ls 6x6x3/4	129.8	109.7
U6-U8	1,170	1,946	310	3,426	2,220	6 Bars 14x1 1/2	(U8-U10 same)	126.0	.....
U10-U12	—712	—1,489	—333	—2,534	—2,090 380	Cov. 40x5/8 2 Ls 6x6x3/4	2 Ls 6x6x5/8 2 Webs 40x1	136.1	.....
U14-U16	—1,181	—2,212	—495	—3,888	—2,600	Cov. 40x3/4 4 Ls 6x6x3/4	2 Webs 40x9/16 4 Webs 40x5/8	208.8	...
U22-U24	1,439	2,220	327	3,986	2,580	6 Bars 14x1 13/16	.....	152.3	....
U26-U28	—670	—1,875 796	—566 179	—3,263 457	—2,203 1,830	Cov. 36x3/4 2 Ls 6x6x3/4 2 Ls 6x4x3/4	2 Webs 36x9/16 4 Webs 36x1/2	170.3	....
L8-L10	—72	—426 370	—152 165	—881 694	—1,110 1,260	4 Ls 6x6x5/8	2 Webs 36x3/4	82.4	....
L12-L14	811	1,637	366	2,814	—270 2,190	4 Ls 6x6x3/4	2 Webs 36x1/2 4 Webs 36x9/16	150.8	125.8
L14-L16	1,062	2,064	462	3,588	2,530	6 Bars 14x1 5/8	.....	136.5	...
L22-L24	—369	—1,383 800	—310 241	—2,398 1,008	1,360 —1,070	4 Ls 6x6x3/4	2 Webs 36x5/8 2 Webs 36x11/16	128.3	...
L28-L30	606	1,651 —623	499 —140	2,834 —235	1,860 —2,650	4 Ls 6x6x3/4	6 Webs 36x9/16	155.3	129.5
Lo-U1	—232	—1,088 533	—387 119	—1,917 630	—1,152	Cov. 36x5/8 2 Ls 6x4x5/8	2 Webs 36x7/8 2 Ls 6x6x3/4	114.1	...
U1-L2	42	784 —480	311 —107	1,409 —817	697 —223	4 Ls 6x6x5/8	2 Webs 36x3/4	82.4	68.4
L4-M5	528	812	390	1,730	—170 1,000	4 Bars 12x1 9/16	(2 Bars laced)	75.0	71.7
M7-L8	—1,007	—1,505	—535	—3,047	—1.840	Cov. 36x5/8 2 Ls 6x6x3/4 2 Ls 6x4x3/4	2 Webs 36x9/16 4 Webs 36x1/2	170.3	.....
U10-M11	1,077	1,872	476	3,425	2,030	4 Bars 16x2 1/16	.....	132.0	.....
U12-M13	517	1,208	358	2,083	1,121	4 Ls 6x6x3/4	2 Webs 36x3/4 2 Webs 36x9/16	119.3	99.1
M25-L26	912	1,285	459	2,656	1,570 —120	4 Bars 14x1 13/16	(2 Bars laced)	101.5	97.9
U31-L32	—588	—1,410	—426	—2,424	—2,160	Cov. 36x3/4 2 Ls 6x6x3/4 2 Ls 6x4x3/4	2 Webs 36x5/8 2 Webs 36x9/16	143.3	.....
U4-L4	—104	—369	—254	—727	—288	1 Ls 4x4x11/16	2 Webs 22x11/16	50.3	....
U8-L8	—338	—658	—338	—1,334	—1,430 (Wind)	4 Ls 6x6x3/4	2 Webs 42x1 (U22-L22 same)	117.8	.....
U12-L12	—514	—1,059	—288	—1,861	—1,043	4 Ls 6x6x3/4	2 Webs 36x13/16 (U18-L18 same)	101.3	....
U26-L26	—568	—910	—325	—1,803	—1,060	4 Ls 6x6x3/4	2 Webs 36x7/8	96.8	.....
U28-L28	—197	—533	—315	—1,045	—463	4 Ls 6x6x5/8	2 Webs 28x11/16	66.9	.....
U8-M9 M21-U22	154	285	191	630	296	4 Ls 6x4x3/4	1 Web 22x3/8	36.0	28.4
L10-M11 etc.	—130	—285	—191	—606	—272	4 Ls 4x4x9/16	2 Webs 22x9/16 (M23-L24 same	41.4	.....
L24-M25	Temp. Erection		.....	.....	871	4 Pls. 18x3/4	Carbon steel	54.0	48.0
M25-U26	127	284	194	605	962	4 Ls 6x6x5/8	1 Web 22x5/8	47.5	39.3
U24-L24	150	187	128	465	—497 243	4 Ls 4x4x1/2	2 Webs 22x1/2 Carbon steel	37.0	30.2
ALL OTHER	HANGERS	.....	.....	.....	....	4 Ls 6x4x3/8	1 Web 22x3/8 Carbon Steel	22.6	..
SUBSTRUTS	and U1-L1, U3-L3, M5-L5, M7-L7, U29-L29, U31-L31				.....	2-15 in. channels at 33 lb.	Carbon Steel	19.8	.....

Column marked total includes 50 per cent. of smaller stress where there is reversal.

of the bridge being fixed, so that the end masonry served as anchorage against longitudinal motion. At the south end a new abutment was built back in the rock bluff, while at the north approach a new pier had to be erected and this could readily be proportioned to resist the traction forces.

Selection of the Continuous-Span Design

Essentially the adoption of continuous spans is the outcome of the designer's aim at producing the most economical structure. In the range of span length here in question, the differences in metal weight between simple, cantilever and continuous spans are in most cases equalized by differences in erection costs, when comparisons are made on the basis of normal prices of material and labor. But here the erection labor costs could not differ greatly, as the cantilever method was to be used in any event.

Simple spans would have required a considerable amount of extra material to take care of erection stresses. With cantilever or continuous trusses most of this extra material would be saved, adding to the inherent weight economy of these types. But a cantilever bridge, besides being undesirable for the short spans and heavy live-load, was not adapted to the requirement of erecting from one end of each span.

The choice as to how many spans should be connected up in a single continuous structure (from three to six; the short north approach span was never considered for inclusion in the continuous span system) was determined by the question of adjusting the reactions to their proper values. Grouping three spans in a continuous structure, only the two end reactions of the group had to be weighed off by jacks and gages to the predetermined amounts. With more spans grouped, for instance all six, as many as five pier reactions would have to be adjusted, and it was thought better to avoid this extra complication.

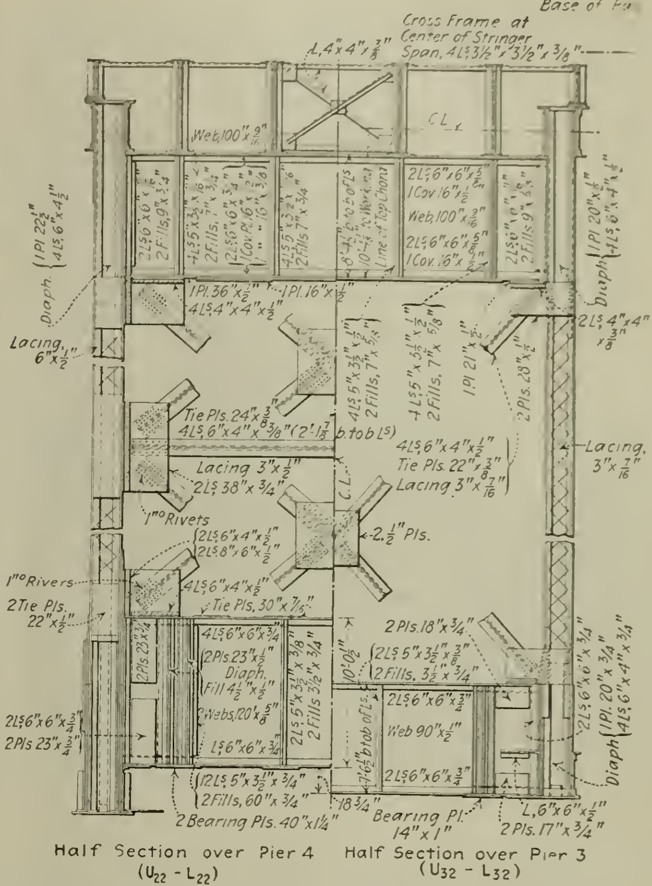


FIG. 6. JACKING GIRDERS AND MAIN SWAY-BRACING

FIG. 5. LONG CANTILEVERING FEATURED THE WORK
Three spans of 350 ft. each were consecutively cantilevered in this way.

As the erection procedure involved only 260-ft. cantilever projections in the 520-ft. span, the full-span projection of the three 350-ft. spans became the critical element of the erection. The first plan was to provide a falsework tower under each one of these spans, on which the cantilever overhang would be landed when erection had proceeded across one-half or two-thirds of the span length. However, when a design using high-strength steel was studied, it was found that the reduced weight made the full 350-ft. cantilever length practicable. A slight addition of metal in the end members of the spans for erection stresses was required, but the cost of the metal so added was less than the cost of a temporary bent, and represented a value permanently retained in the structure and adding materially to the rigidity of the completed bridge.

At the prices prevailing in 1914 the metal selected furnished also the cheapest bridge. Comparisons were based on American Railway Engineering Association unit-stresses for plain steel, 40% higher stresses for silicon steel, and 27,000 lb. per square inch for the eyebars.

Specially treated eyebars of high strength had been developed by the American Bridge Co. a short time before, and these were adopted for the Allegheny River bridge. The specifications under which they were furnished required them to show results in full-size tests equal to or exceeding the following figures: Elastic limit, 50,000 lb. per sq.in.; ultimate strength, 80,000 lb. per sq.in.; elongation, 8% in 18 ft.; reduction of area, 35%. Ten

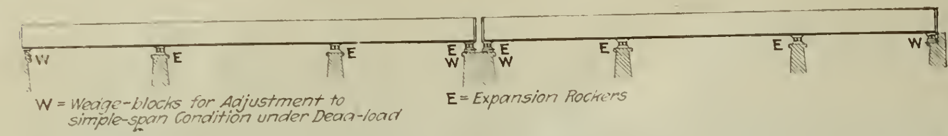


FIG. 7. EACH SPAN-GROUP ADJUSTED TO CORRECT STRESS CONDITIONS BY WEDGE BEARINGS AT END SUPPORTS

of the bars made for the bridge were tested, with the following extreme and average results:

FULL-SIZE TESTS OF SPECIAL EYEBARS

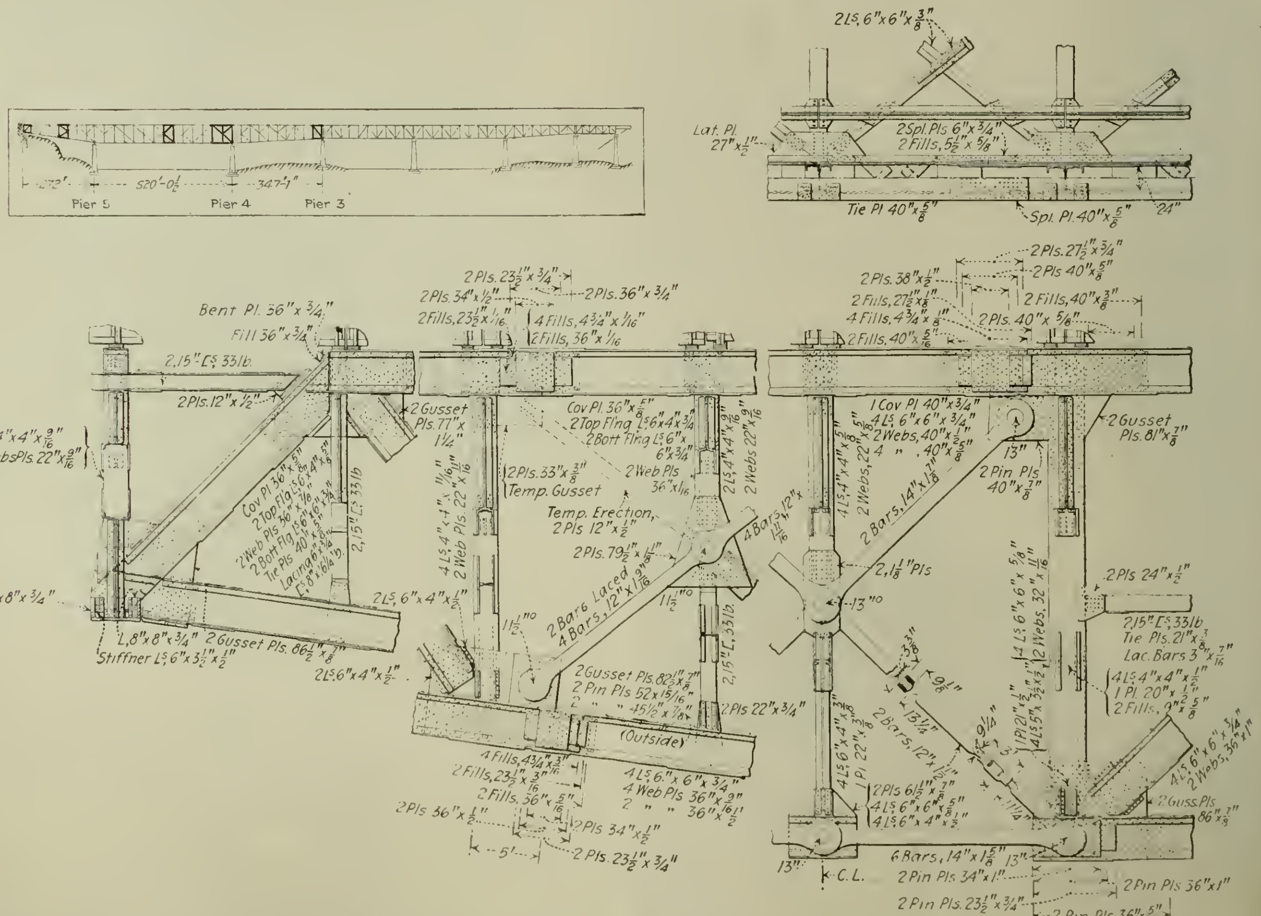
Elastic Limit Lb. per Sq.In.	Ultimate Strength Lb. per Sq.In.	Elongation In 18 Ft., per Cent.	Reduction of Area Per Cent.
53,000-63,000	81,000-90,000	7.8-13.2	46-59
av. 58,300	av. 86,300	av. 10.4	av. 52.5

Design Made for Cooper's E75 Loading

The design of the new bridge was based on Cooper's E75 loading. The engineers cited the following facts: Ore trains now running on the Bessemer & Lake Erie have a weight exceeding 5000 lb. per lin.ft., and there is no present indication that a limit has been reached. During the past 25 years or more the increase in railway loading has averaged about E1 per year, and if this rate is maintained an E75 loading will be reached 15 or 20 years hence. Another iron-ore road, the Duluth, Missabe & Northern, already carries trains exceeding 6000 lb. per ft. Solid trains of 100-ton ore cars may soon be operated, and they would amount practically to E70 loading. Engine loadings show at least an equal rate of growth. Passenger locomotives are now operating with 73,000 lb. on an axle; if the same axle loading should be adopted for freight locomotives the result would be not far from the Cooper E75 engine. The final adoption of E75 loading expressed the conclusion that it is a reasonable future contingency as to both engine and train weight.

Since railroad practice in general, apart from that on special ore-carrying roads, has shown a greater growth of engine weight than of train weight, some study was given to a departure from the Cooper type of loading making allowance for the greater probability of engine-weight increase. A loading consisting of two E75 engines followed by a train weighing 6500 lb. per lin.ft. was compared with a straight E75 loading. It was found that there would be little difference in stresses in this type of bridge, because most of the stress maxima occur under short load segments; little could be saved, therefore, by holding train loading to 6500 lb.

In designing the continuous spans a first approximation to the required distribution of metal was obtained by using ordinary continuous-girder formulas based on constant moment of inertia. The resulting design was then analyzed, corrected and re-analyzed by deflection calculations. Instead of taking the dead-load reactions at the amount corresponding to the condition of level supports, however, they were fixed arbitrarily at an early stage of the calculations, and the final adjustment



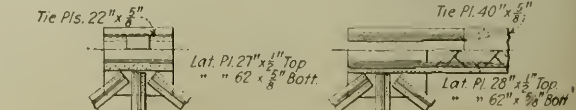


FIG. 9. TYPICAL DETAILS OF SOUTH SPAN-GROUP—DIFFERENCE IN LEVEL OF OLD AND

of the bridge after erection was accomplished by jacking up until the end reactions equaled those used in designing.

The dead load was assumed at 10,200 lb. per lin.ft. of bridge, consisting of 1900 lb. for the deck and 8300 lb. for the steel. For the live-load, impact was computed according to the A. R. E. A. specifications.

In continuous spans many members get their maximum stresses under broken load; i.e., two or more separated blocks of load. An unusual procedure was followed with respect to such loading. The stress due to broken load was not considered unless it increased the unit stress 25% beyond that allowed by the specifications, or unless it produced reversal of stress; in the former case, a 25% increase in unit stress was allowed in computing the section. The small probability of short lengths of train occupying just the required panels, with no load elsewhere on the structure, was at the bottom of this allowance.

In designing members subject to reversal of stress, it was assumed that the entire dead-load would be effective in counteracting live-load stress, which departs from the usual assumption intended to provide for reduced weight of deck, increase of live-load, and other effects. The member was designed on the basis of stresses found by increasing the maximum stress in each direction by 50% of the smaller. The connections in all cases were proportioned for the stress as thus increased, unless erection stresses required still more rivets.

Wind loads were assumed as follows: On the top chord, a distributed load of 30 lb. per sq.ft. of exposed surface of both trusses and the floor, and a moving load of 750 lb. per lin.ft.; on the bottom chord, a distributed load of 50 lb. per sq.ft. of exposed surface of both trusses. Wind during erection was taken at 30 lb. per sq.ft. of exposed surface. The unit stresses for combined wind and vertical load were increased 25% above

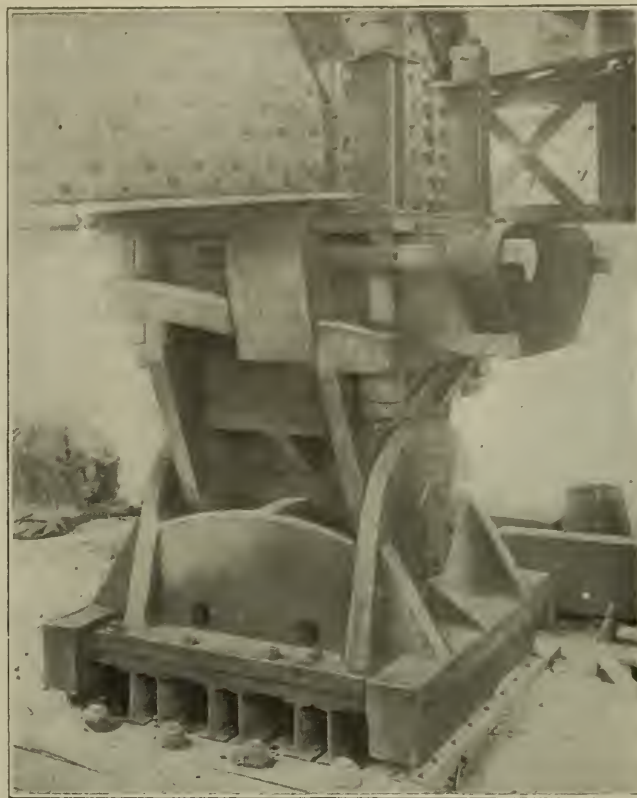
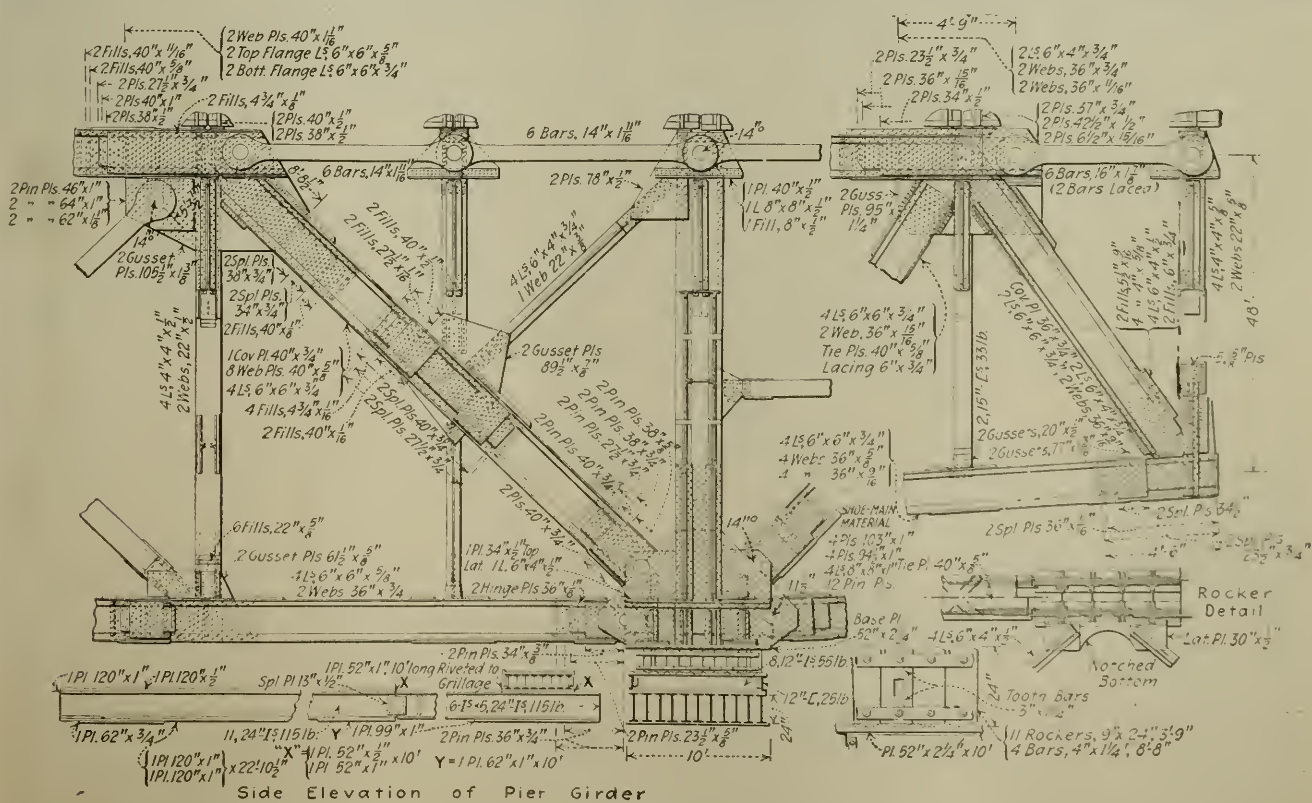


FIG. 8. FIXED-END SHOE WITH ADJUSTING WEDGES

When the three spans of a group were completely erected, hydraulic jacks were set under the jacking girders at the ends of the group; after lifting until the desired reaction is obtained, the wedges are drawn up.



COMPACT CONNECTIONS OBTAINED BY USE OF LARGE RIVETS
NEW MASONRY REQUIRED SPLICED PIER GIRDERS

FIG. 10. SUPPORT OF NORTH SPAN DURING ERECTION

those normally allowed by the specification, and for combined wind and load during erection conditions, 50% above normal. The single-track deck used in cantilever erection was computed as weighing 500 lb.; with one pair of stringers and bracing, it formed the floor load on the cantilever. The live-load consisted of a derrick car taken as weighing 212,000 lb. When the new bridge is completed, the old bridge will be supported from it by bracket connections, while being dismantled; stresses due to this weight, with E50 loading on the far track of the new bridge, were considered erection stresses. Carbon steel was used wherever rigidity or minimum section governed: In vertical hangers supporting the bottom chord, horizontal struts (secondary members in the truss system), safety stringers, shoes, grillages, and all lateral and sway bracing. Members over the abutments also were made of carbon steel. The rivets (as already noted) and all pins were of the grades ordinarily used. The bearing pressure on pins was taken at 30,000 lb. per sq.in., while for rivet stresses those of the A. R. E. A. specification were taken.

Stresses in the lateral bracing, which is continuous just like the main trusses, were calculated by the three-moment formula, assuming uniform moment of inertia and no sheer deflection. A stress sheet of the south half of the bridge is reproduced in condensed form in Fig. 4. This half contains the 520-ft. span and the counterweighted south span. The concrete counterweight already mentioned (350 tons weight) was not placed until after completion of the bridge. During erection of the south half of the 520-ft. span the abutment end of the shore span was loaded down with a temporary counterweight of steel rails weighing 120 tons.

As in other cases where high-strength steel has been used there was some perplexity concerning the riveting. Search and experiments did not bring to light any material affording with definite certainty a higher rivet strength, so as to bring rivets and body material into the same relation as in work using ordinary steel. The only way to avoid ungainly and expensive connections was to use large rivets, and therefore 1¼-in. rivets (of ordinary rivet steel) were adopted for the main connections.

An unusual rivet-hole specification was written, because of the use of silicon steel and extra large rivets. The intent of the specification is to limit the material punched to a thickness $\frac{1}{16}$ in. greater than the punch diameter, and at the same time have ⅛-in. of metal left for reaming (¼ in. on the diameter). The clause reads as follows:

> Material ½-in. thick or less shall be punched full size. Material over ½-in. thick, but not more than ¼ in. less than the nominal diameter of the rivet, may be punched to a diameter $\frac{3}{16}$ in. less than the nominal rivet diameter and reamed to $\frac{1}{16}$ larger than the nominal rivet diameter. Material thicker than above specified is to be drilled from the solid. No material over 1⅜ in. thick is to be used. The edges of all main gusset plates are to be planed.

Adjustment to Correct Stress Conditions

During erection, as soon as a span was landed it was jacked up to correct level—or rather to a level 8 in. above that shown by the drawings, so as to give room for inserting the track and rollers on which the new bridge is to be rolled along the piers into final position after the old structure shall have been removed. But adjustment to design elevation only approximated the desired condition of correct distribution of the reactions, as the assembly of the members is likely to make the shape of the structure depart slightly from the diagram shape. In the final adjustment, therefore, the jacking was governed by reaction rather than by amount of lift.

The reactions produced by the jacking were equal to those used in the design calculations. These were approximately but not exactly the same as the reactions corresponding to true continuity on supports exactly fitting the unstressed shape of the bridge. To allow readjustment in case of any subsequent pier settlement, provision is made for jacking and shimming. The lower members of the sway frames over the piers are heavy girders, under which jacks may be set and the bridge raised or lowered. Shimming to the changed elevation is provided for by pairs of wedges under the end shoes of the two groups, at Pier B, Pier 3 and the south abutment. The jacking and shimming, both at time of final adjustment and at future readjustment, are done at the two end supports of each group of spans, as sketched in Fig. 7. When the end reactions are brought to the proper amounts the middle reactions will also be right. However, jacking must be done at the intermediate supports in connection with the transverse rolling of the bridge, to free the blocking, lower the bridge down upon the roller track, later free the rollers and track and lower to final bearing. Jacking girders are therefore provided here as well as at the end supports (Fig. 6), but weighing the jack loads is to be done at the end bearings only. The operations of rolling the bridge to permanent position and making the final adjustment have not yet been performed.

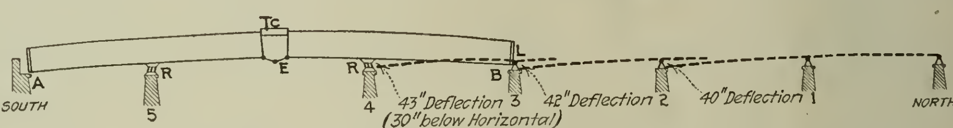


FIG. 11. DEFLECTION CONDITIONS AND FINAL CLOSURE OF CANTILEVERS

By making top-chord link L 1¼ in. short, the deflection at Pier 4 was limited to 30 in. below the horizontal, the total deflection from unstressed condition being 43 in. Link L was removed when span was landed and jacked up. To close south group, the cantilever halves were pushed forward and points A and B lowered. Eyebars E were slack when placed. After the closure section T was dropped in, jacks at A and B tipped the two arms until T came to bearing and then pulled bars E taut, rockers R coming back to vertical.

Typical details of the bridge are contained in the panel-point elevations Fig. 9 and the cross-sections Fig. 6.

The designers aimed to use the maximum economic panel-length. In the long span the length was partly controlled by the desirability of having an odd number of main panels, to get the simplest conditions for central closure. The width of the deck, 31 ft., was determined partly by consideration of convenience of providing for the lateral plates outside of the outer track stringers, and partly by the desire to make provision for future gantleting of tracks, so as to avoid switching on the approaches when four line tracks are put in. Ties long enough to extend over the entire width of deck are used, of 10 x 12-in. section.

A safety stringer was placed at the outer edge of the deck, directly over the top chord. There is only one set of stringer laterals, placed between the center stringers. In erection, the cantilever construction was carried out with only one pair of stringers (the middle pair) in place, in order to reduce the weight of the overhang; as these stringers required a lateral system in order to carry the derrick-car loads safely, the adopted arrangement avoids subsequent shifting of stringers or laterals. The cross-frames which brace the unsupported stringer flanges are carried out to the edges of the deck and here connect with a safety stringer along each edge.

The shoes at the four points of end bearing are of cast steel, in conformity with normal practice. At the intermediate bearings, however, no separate shoes are used, but the truss panel-point serves as upper shoe and grillage on the pier girders as lower shoe. The lower and upper roller plates which inclose the rockers are in direct contact with these two parts. The panel-point assemblage being pin-connected to four of the abutting members, it was considered a degree of adjustability practically equivalent to that of a pin-shoe bearing is obtained.

Expansion on the middle pier must take care of the entire 2200-ft. length of the river bridge. Using the customary allowance of 1 in. expansion per 100 ft. length of bridge, 22 in. of expansion clearance is provided, half at each of the two adjoining shoes. The rockers are unusually large, being 24 in. in diameter.

Fabrication of the bridge material involved no special features beyond those introduced by the rivet-hole drilling-and-reaming specification already noted. The truss members were built to full-load camber, each member being made of a length equal to its geometric length plus or minus the full-load compression or tension respectively, with a view to producing the true diagram outline under full load. The angles between the members at the connection points, however, were made as required for connecting up in the unstressed condition; that is, there was no attempt to eliminate secondary stresses. The trusses were in general not assembled in the shop yard for making the connections, but the gusset plates were drilled to iron templets.

Erection was handled by a 60-ton derrick car of the Mitchell type. The choice of equipment was partly controlled by the necessity of keeping the load on the overhanging spans in the cantilever erection down to a minimum in order to avoid excessive erection stresses.

The cantilever erection progressed from either end to closure at the middle of the 520-ft. span. This meant that the north span of the south group had to be erected from the north, or in other words had to be continuous with the north group for the time being. The continuity was brought about by eyebar links between the adjoining top chords, over Pier 3, and blocks between the bottom chords, which were removed as soon as the span was landed on Pier 4.

The links just mentioned were made short enough to hold the span 13 in. higher at Pier 4 than it otherwise would be (Fig. 11). The deflection of the span just before landing was 43 in., but the short links by tipping up the span held the end to 30 in. below the horizontal, giving more clearance between trusses and pier for placing the jacks. In jacking up here to release the links and blocking, however, the full 43-in. height had to be jacked, and the truss then lowered 13 in.

Large deflections occurred in the previously erected spans also. The second span from the north deflected 40 in. just before landing on Pier 2, and the next span deflected 42 in. at Pier 3.

In preparing for the cantilever work in the 520-ft. span, which was to close at midspan, the rocker bearings on Piers 4 and 5 were rendered fixed by bolting on interlocking side-plates. Just before this the two halves of the structure were pushed forward several inches, and the rear ends lowered to tip the forward ends up, so that when the meeting cantilevers came together they were too close at the bottom chord and too far apart at the top chord.

With this preparation, closing the span was an operation free from difficulty. The bottom-chord eyebars of the middle panel, which are jointed at the middle sub-panel point, were inserted and allowed to hang slack. The top-chord closing section was then dropped into place, and jacks were at once set to work raising the anchor ends of the arms. As soon as the top-chord joints came to bearing as a result of this jacking, the rocker joints on the main piers were released by taking off the fixing plates, so that the trusses might move backward at these points as the jacking continued. Further jacking drew the slack eyebars taut, and put them under tension corresponding to the desired simple-span condition.

There were some complications in the erection of the north anchor span on falsework due to local interferences. Two railway tracks pass under the span, and as traffic on them could not be interrupted a gap in the

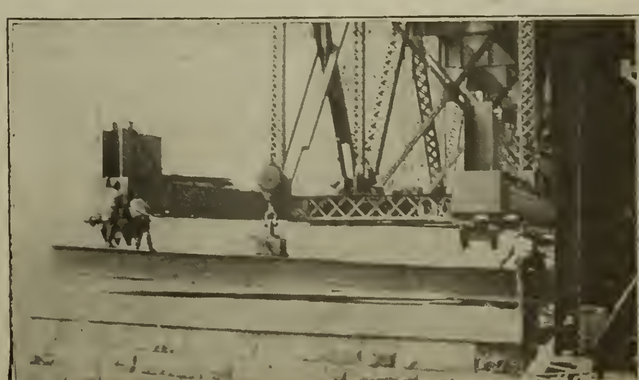


FIG. 12. DERRICK-CAR SETTING PIER GIRDERS; CANTILEVERED SPAN READY TO LAND

falsework resulted which had to be crossed by cantilever erection, requiring a steel fulcrum tower (Fig. 10) and a temporary tail counterweight.

Old Bridge Must Be Removed and New Structure Rolled to Final Position

The position in which the new bridge was erected, and its relation to the old bridge, may be seen in the cross-section sketch, Fig. 2. As the pier width is barely sufficient for both structures, pier girders (see Fig. 1) are used to prevent load concentration near the ends of the masonry and to tie the new and old masonry together. The same girders will carry the roller track required for shifting the new bridge tc final position.

Before this moving the old bridge will be taken apart, after each panel-point is supported by connection to the new bridge through beam brackets and diagonal-rod hangers. Following this operation the old half of each pier will be built up to final coping elevation (made necessary by the change of grade), and the remaining sections of grillage girders placed, before preparation can be made for moving.

Rolling the new bridge to place will follow closely the precedent established by recent bridge-rolling operations. As the entire length of the movement will be on masonry, the increased resistance and frequent delay encountered when part of the support is on timber falsework is expected to be absent. The two groups of three spans will be moved successively. The shift is almost exactly equal to the track spacing on the new bridge, and it will therefore be possible to resume operation, if desired, when only one group has been moved. On the other hand the moving of the north group will be complicated by the necessity of skewing the end spans of the filling viaduct at the north approach (the filling track, at the grade of the revised profile, being then used temporarily as running track) to make connection between this viaduct on the line of the erection position and the river bridge in the final position, until the approach fill can be built up to grade.

The steel weight of the new bridge is about 10,150 tons, or 8700 lb. per lin.ft.

Both fabrication and erection were carried out by the American Bridge Co. Erection began in April, 1917, and the long span was closed February, 1918. The new bridge has carried regular traffic since Mar. 1 in its temporary position.

Designs of Workmen's Dwellings Exhibited

Industrial dwellings of the "model city" at Morgan Park, Minn., are a feature of an architectural exhibit at the Art Institute, Chicago, one room being devoted to a display of plans, photographs and paintings of the town and its buildings. This town was established a few years ago by the Minnesota Steel Co. to provide accommodation for employees at its large steel plant near Duluth. The buildings are of concrete block construction and range from single residences to long structures forming three or four residences. Wide streets with a curved layout and ample provision for gardens, playgrounds and other open spaces are characteristics of the general plan. The designs were made by Dean & Dean, of Chicago, architects for the Morgan Park Company.

Wood Tieplates Found to Be Poor Substitute for Steel

ECONOMY in track construction was the goal of extensive experiments begun some years ago in the use of wood instead of steel for tieplates to protect ties from being cut and abraded by the rails. These plates were used mainly in connection with treated ties, but were also used to some extent on untreated ties. One special advantage suggested was their application on branch lines, thus protecting the ties at much less expense than the use of steel would involve. This feature of track work was dealt with in *Engineering News* of June 21, 1906, page 694. Recent inquiry indicates that on the whole the plates have not been satisfactory, and have not given the expected service.

About 65,000 creosoted wood tieplates were applied in 1906 by the Cleveland, Cincinnati, Chicago & St. Louis (Big Four) Ry., and in 1917 only about 20% of these remained in service. They were mainly of beech and elm, 5 x 8 in. and ⅜ in. thick, fastened to the ties by sixpenny nails. Their failure has been due almost exclusively to splitting after five or six years service. Some of the plates were also worn or abraded. While they protected the ties as long as they remained intact, they did not prove as efficient as was expected. In 1907, about 500 creosoted white-oak tieplates 6 x 7 x ¼ in., averaging ½ lb. of creosote each, were laid on a test track of the Northern Pacific Ry. in Montana. These were all removed in August, 1910. In service these plates soon split badly and worked out from under the rails, and they also cut into the softwood ties.

Creosoted Wood Tieplates Proved Unsatisfactory on Western Railroad

Creosoted wood tieplates were used extensively at one time on the Gulf, Colorado & Santa Fe Ry., but the results did not come up to expectations. The plates furnished were not all applied, as their replacement with steel plates was soon begun. That the results were somewhat unsatisfactory, however, is attributed to inadequate attention to the manner of carrying on the experiment. The main defect, and the one considered responsible for failure, was the omission of means of seating and fastening the plates properly. Unless the ties are adzed to give recessed and level bearings in which the plates may be inserted, the latter are likely to split and shift. At first they were secured only by the track spikes, but as they showed a tendency to creep in the direction of rail, each plate was afterward fastened to the tie by two small nails.

On this road the wood plates were regarded as a temporary experiment, and it was estimated that their life would be only a quarter to a third of that of steel tieplates. Furthermore, they were not advocated for main-line track carrying heavy traffic. But for branches and lines of medium traffic it was considered that if hardwood plates were inserted and fastened on ties properly adzed to receive them they would much more than pay for their cost by preventing the rails from cutting the ties. The cost of the plates in the track was about 1c. each, as compared with 0.6c. for those of the Cleveland, Cincinnati, Chicago & St. Louis Railway.

Continuous Trusses of Silicon Steel Feature New Allegheny River Bridge

Sharp Departure from Precedent in Bridge Design—Minimum Weight Desired for Economy and To Facilitate Cantilever Erection—Reactions Adjusted by Jacking—High-Strength Eyebars of New Kind

WHERE the Bessemer & Lake Erie R.R. crosses the Allegheny River a new double-track deck bridge proportioned for modern loading has just been built to replace a light single-track structure. The renewal presented a problem of unusual complication, in solving which the engineers were led to adopt a strikingly original solution.

The old bridge had simple spans. In the new bridge the six main spans, ranging from 272 to 520 ft., are continuous, arranged in two groups of three each. Through this sharp departure from prevailing practice the new bridge shares with its Ohio River contemporary at Sciotoville the distinction of reintroducing continuous construction into American practice in large-scale bridge work.

A special feature is a counterweight at the end of the southerly span, to counteract any uplift that might be caused by live-load on the adjoining span, 520 ft. long. The shore span was extended into the rock bluff, about 100 ft. back of the old abutment, to obtain sufficient anchor-span length, but a 350-ton concrete counterweight block was required in addition. The block is molded around the end sway-frame, and occupies the full width between trusses and the height from jacking girder to floorbeam.

The new bridge is made noteworthy also by its material. Silicon steel was used for the stiff members, while the eyebars—used for all tension members not subject to stress reversal—are of a new kind claimed to be equal to nickel-steel eyebars. The builders give no information as to the method of making these eyebars. Results of strength tests are quoted farther on.

Both the continuous construction and the choice of material were in large part due to the necessity for erecting the bridge by the cantilever method, which under the conditions governing the work required a design of minimum weight for the desired load capacity. Other difficulties of the case had contributory influence, however, and the fact that the adopted solution proved the cheapest was the controlling consideration.

Since the erection method was fixed, the relative costs of different types of bridge were practically expressed by their relative weights—after adding the extra section required for erection stresses—and therefore the weight economy of the continuous design meant corresponding gain.

The old bridge was built in 1896, for E40 loading, which then was heavier than existing railway loading. The Bessemer & Lake Erie transports large quantities of iron ore, in which class of traffic the greatest increase in train weights has occurred. Its present maximum train weights are in excess of E50. For some years the bridge had been overloaded, to the extent even that double-headed trains had to shift one engine to the rear before crossing. But the inadequate traffic capacity of the single track was equally hampering. In 1914, therefore, replacement was taken in hand. It was studied jointly by H. T. Porter, chief engineer of the railway, and C. G. E. Larsson, assistant chief engineer of the American Bridge Co.

Some of the governing conditions of the problem were

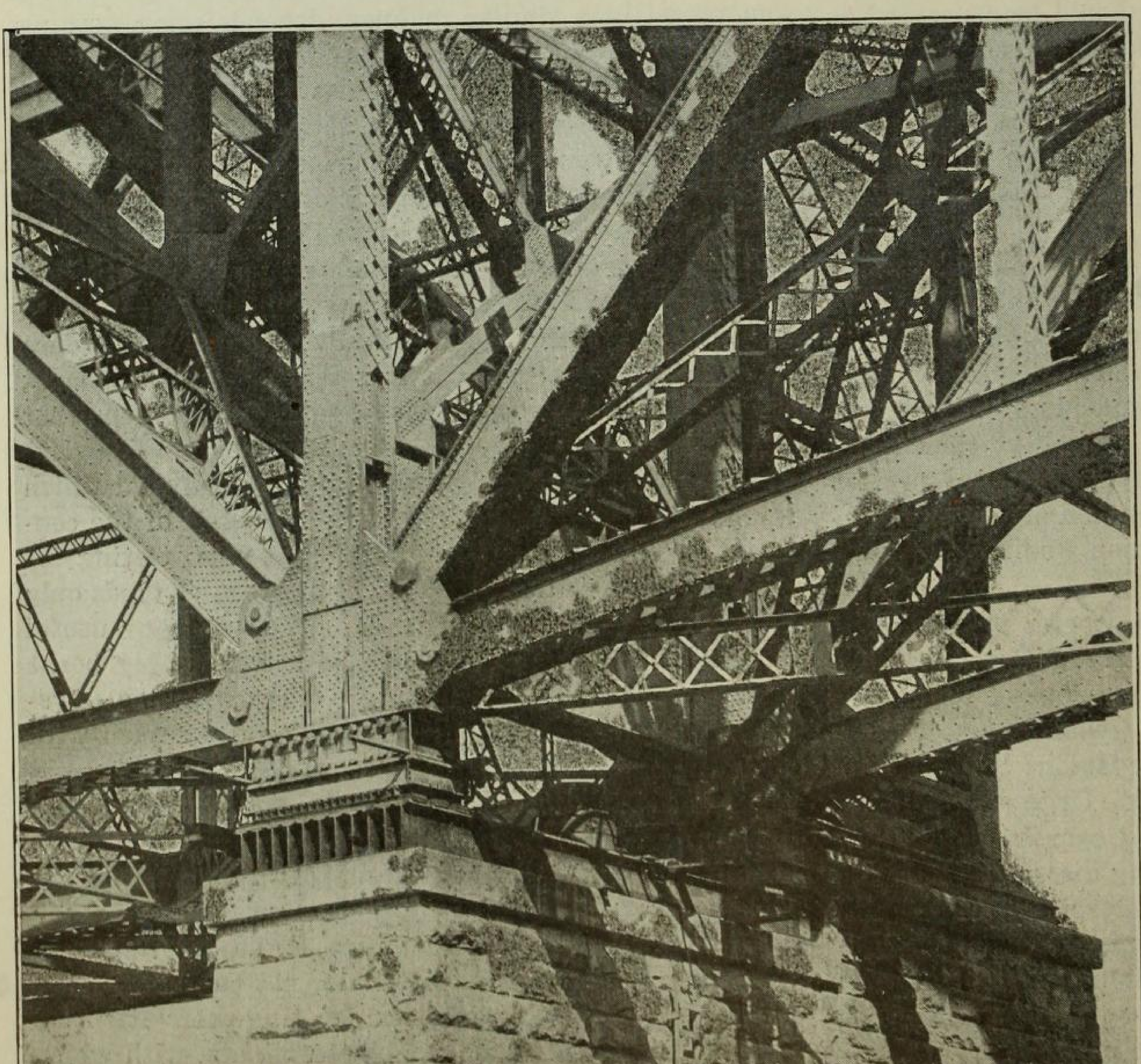


FIG. 1. NEW BRIDGE WILL BE ROLLED TO CENTRAL POSITION ON PIERS AFTER OLD BRIDGE IS DISMANTLED

Far half of pier is to be cut down after removal of old structure. Pier girders prevent concentration of load at edge of masonry. At the four main bearings the truss panel-point acts as upper shoe. Locking plates of rockers were removed when last span was being closed.

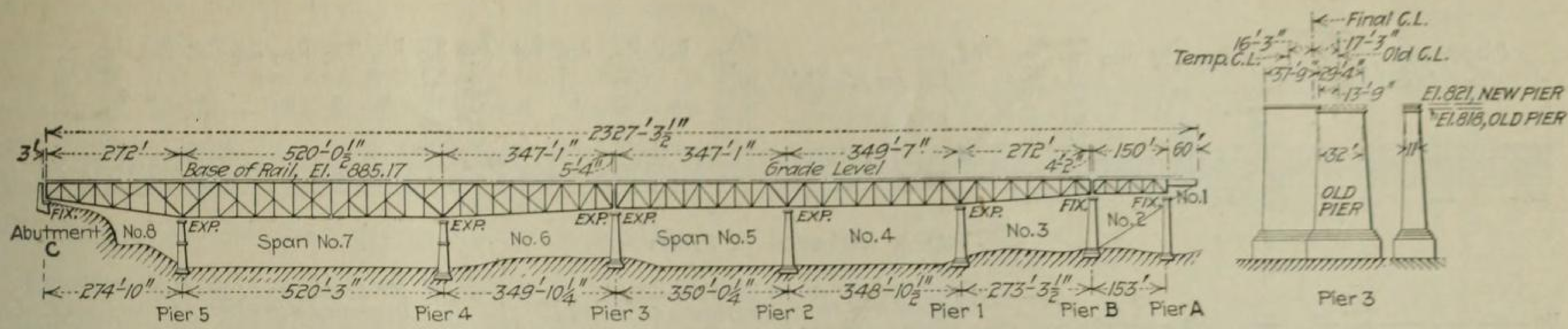


FIG. 2. DIAGRAM OF NEW CONTINUOUS-SPAN BRIDGE OVER ALLEGHENY RIVER BUILT FOR ORE-TRAIN TRAFFIC

the absolute necessity of maintaining traffic at all times; the importance of avoiding falsework in the river (as the Allegheny is considered dangerous to temporary timber work that must be maintained for a long period); the desirability of improving the profile of the road by raising the track 10 ft. at the north end of the bridge so as to make the grade level and eliminate a highway grade crossing; the existence at the north end of a 1200-ft. approach viaduct 130 ft. high; and the comparative weakness of the old piers in the direction of the track. It was thought at first that the bridge would have to be built on new alignment, but finally a practicable method of building on the existing alignment, with use of the old piers, was developed.

An essential feature of the solution was replacement of the north approach viaduct by a fill. This remarkable operation, in which two steel viaducts—the old one and a dumping viaduct built alongside—are buried, was described in *Engineering News* of Feb. 22, 1917, p. 322. The fill is not yet completed, delay having resulted from railway conditions during the last season and also through the occurrence of excessive lateral pressure of the fill on the viaduct.

For the river bridge, the replacement involved the following procedure: Widening the single-track piers on the old footings, which had been built of double-track width originally, and erecting the new bridge alongside the old one, since the piers as widened were long enough (with the help of pier girders) to accommodate both structures side by side; then transferring traffic to the new bridge and dismantling the old; and, finally, rolling the new bridge over into central position on the piers. The erection was begun at the two ends, by building the shore spans on falsework. All the other spans were cantilevered, and the work proceeded by continuous cantilever operation to closure at the middle of the 520-ft. span.

To avoid overstress in the pier masonry under the action of braking forces, the bearings on all the river piers were provided with expansion rollers, the two ends

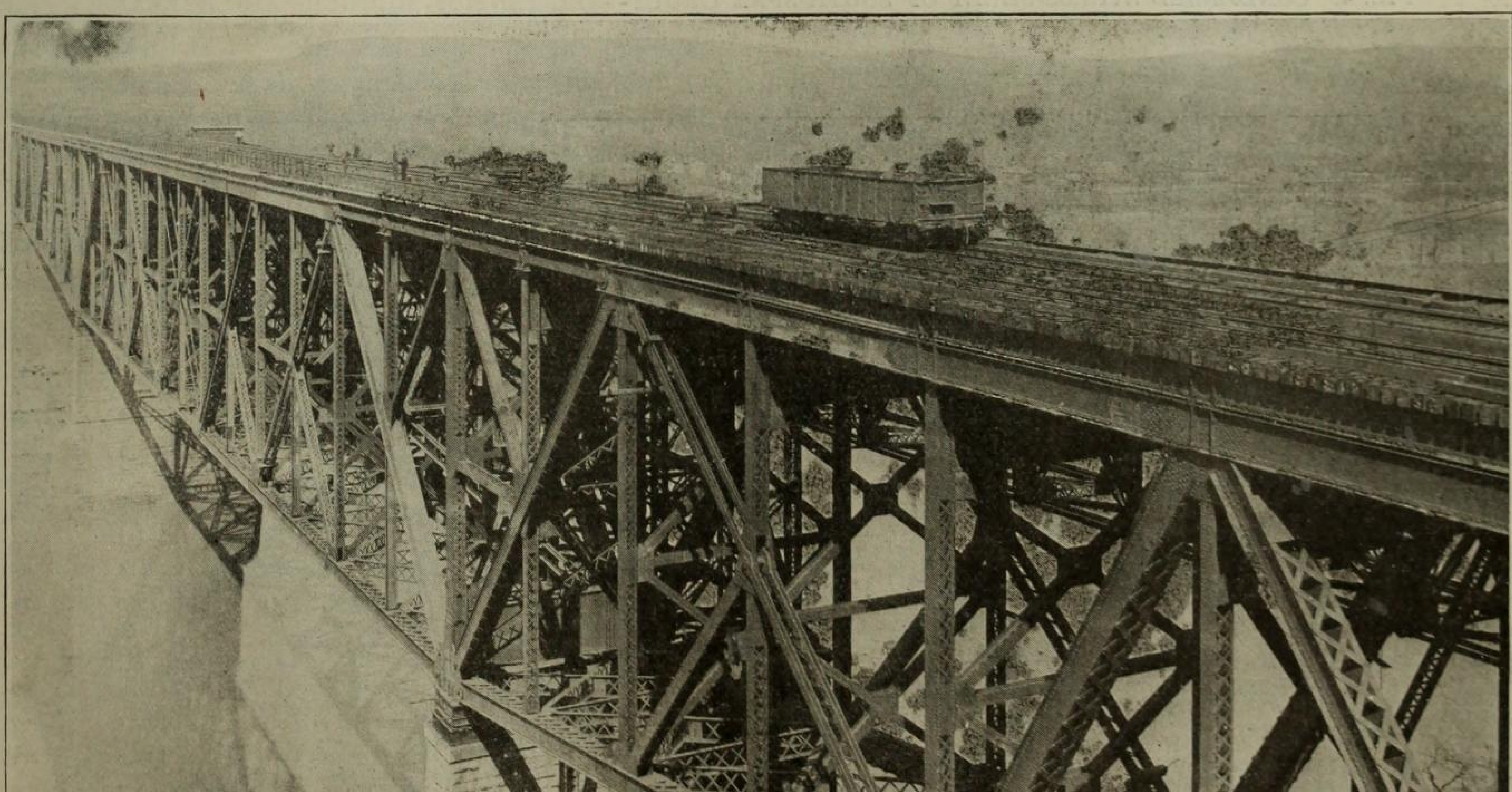


FIG. 3. HIGH-STRENGTH STEEL AND CONTINUOUS CONSTRUCTION RESULT IN A STRUCTURE OF LIGHT APPEARANCE

Looking north from south bank. The span beyond the first pier is 520 ft. long. Designed for E75 loading, this double-track bridge is among the heaviest to be found in our railway system.

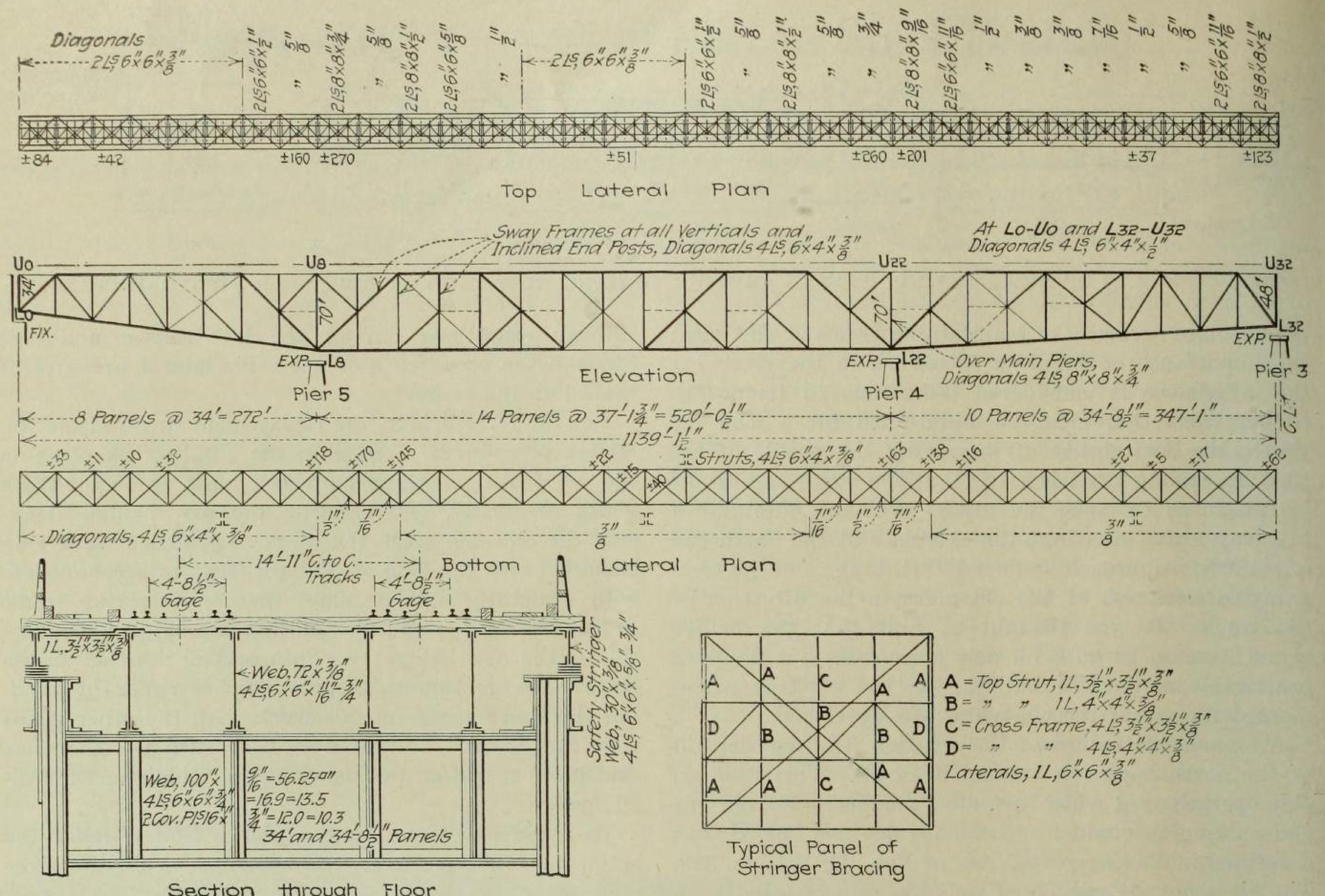


FIG. 4. CONDENSED STRESS-SHEET OF SOUTH-SPAN GROUP—ALL TENSION MEMBERS ARE EYEBARS

Number	Maximum Stresses: Dead	Live	Impact	Total	Erection	Material		Sectional Area, Gross	Net
Uo-U1	......	......	.....	....	....	2-15 in. channels, 33 lb.	Carbon steel	19.8	.....
U1-U3	—189	—1,175 674	—417 151	—2,099 954	—1,229 150	Cov. 36x$\frac{5}{8}$ 2 Ls 6x4x$\frac{3}{4}$	2 Webs 36x$\frac{3}{8}$ 2 Ls 6x6x$\frac{3}{4}$	114.1	.....
U-3-U6	52	—1,133 1,115	—402 250	—2,191 2,125	—1,359 650	Cov. 36x$\frac{5}{8}$ 2 Ls 6x4x$\frac{3}{4}$	2 Webs 36x1$\frac{1}{16}$ 2 Ls 6x6x$\frac{3}{4}$	129.8	109.7
U6-U8	1,170	1,946	310	3,426	2,220	6 Bars 14x1$\frac{1}{2}$	(U8-U10 same)	126.0	.....
U10-U12	—712	—1,489	—333	—2,534	—2,090 380	Cov. 40x$\frac{5}{8}$ 2 Ls 6x6x$\frac{3}{4}$	2 Ls 6x6x$\frac{5}{8}$ 2 Webs 40x1	136.1	.....
U14-U16	—1,181	—2,212	—495	—3,888	—2,600	Cov. 40x$\frac{3}{4}$ 4 Ls 6x6x$\frac{3}{4}$	2 Webs 40x$\frac{9}{16}$ 4 Webs 40x$\frac{5}{8}$	208.8	...
U22-U24	1,439	2,220	327	3,986	2,580	6 Bars 14x1$\frac{13}{16}$	....	152.3	.....
U26-U28	—670	—1,875 796	—566 179	—3,263 457	—2,203 1,830	Cov. 36x$\frac{3}{4}$ 2 Ls 6x6x$\frac{3}{4}$ 2 Ls 6x4x$\frac{3}{4}$	2 Webs 36x$\frac{9}{16}$ 4 Webs 36x$\frac{1}{2}$	170.3	.....
L8-L10	—72	—426 370	—152 165	—881 694	—1,110 1,260	4 Ls 6x6x$\frac{5}{8}$	2 Webs 36x$\frac{3}{4}$	82.4	.....
L12-L14	811	1,637	366	2,814	—270 2,190	4 Ls 6x6x$\frac{3}{4}$	2 Webs 36x$\frac{1}{2}$ 4 Webs 36x$\frac{9}{16}$	150.8	125.8
L14-L16	1,062	2,064	462	3,588	2,530	6 Bars 14x1$\frac{5}{8}$	....	136.5	.....
L22-L24	—369	—1,383 800	—310 241	—2,398 1,008	1,360 —1,070	4 Ls 6x6x$\frac{3}{4}$	2 Webs 36x$\frac{5}{8}$ 2 Webs 36x$\frac{11}{16}$	128.3	.....
L28-L30	606	1,651 —623	499 —140	2,834 —235	1,860 —2,650	4 Ls 6x6x$\frac{3}{4}$	6 Webs 36x$\frac{9}{16}$	155.3	129.5
Lo-U1	—232	—1,088 533	—387 119	—1,917 630	—1,152	Cov. 36x$\frac{5}{8}$ 2 Ls 6x4x$\frac{5}{8}$	2 Webs 36x$\frac{3}{4}$ 2 Ls 6x6x$\frac{3}{4}$	114.1	.....
U1-L2	42	784 —480	311 —107	1,409 —817	697 —223	4 Ls 6x6x$\frac{5}{8}$	2 Webs 36x$\frac{3}{4}$	82.4	68.4
L4-M5	528	812	390	1,730	—170 1,000	4 Bars 12x1$\frac{9}{16}$	(2 Bars laced)	75.0	71.7
M7-L8	—1,007	—1,505	—535	—3,047	—1,840	Cov. 36x$\frac{5}{8}$ 2 Ls 6x6x$\frac{3}{4}$ 2 Ls 6x4x$\frac{3}{4}$	2 Webs 36x$\frac{9}{16}$ 4 Webs 36x$\frac{1}{2}$	170.3	.....
U10-M11	1,077	1,872	476	3,425	2,030	4 Bars 16x2$\frac{1}{16}$	....	132.0	.....
U12-M13	517	1,208	358	2,083	1,121	4 Ls 6x6x$\frac{3}{4}$	2 Webs 36x$\frac{5}{8}$ 2 Webs 36x$\frac{9}{16}$	119.3	99.1
M25-L26	912	1,285	459	2,656	1,570 —120	4 Bars 14x1$\frac{11}{16}$	(2 Bars laced)	101.5	97.9
U31-L32	—588	—1,410	—426	—2,424	—2,160	Cov. 36x$\frac{3}{4}$ 2 Ls 6x6x$\frac{3}{4}$ 2 Ls 6x4x$\frac{3}{4}$	2 Webs 36x$\frac{5}{8}$ 2 Webs 36x$\frac{9}{16}$	143.3	.....
U4-L4	—104	—369	—254	—727	—288	4 Ls 4x4x$\frac{11}{16}$	2 Webs 22x$\frac{11}{16}$	50.3	.....
U8-L8	—338	—658	—338	—1,334	—1,430 (Wind)	4 Ls 6x6x$\frac{3}{4}$	2 Webs 42x1 (U22-L22 same)	117.8	.....
U12-L12	—514	—1,059	—288	—1,861	—1,043	4 Ls 6x6x$\frac{3}{4}$	2 Webs 36x$\frac{13}{16}$ (U18-L18 same)	101.3	.....
U26-L26	—568	—910	—325	—1,803	—1,060	4 Ls 6x6x$\frac{3}{4}$	2 Webs 36x$\frac{5}{8}$	96.8	.....
U28-L28	—197	—533	—315	—1,045	—463	4 Ls 6x6x$\frac{5}{8}$	2 Webs 28x$\frac{11}{16}$	66.9	.....
U8-M9 M21-U22	154	285	191	630	296	4 Ls 6x4x$\frac{3}{4}$	1 Web 22x$\frac{3}{8}$	36.0	28.4
L10-M11 etc.	—130	—285	—191	—606	—272	4 Ls 4x4x$\frac{9}{16}$	2 Webs 22x$\frac{9}{16}$ (M23-L24 same	41.4	.....
L24-M25	Temp. Erection		.....	....	871	4 Pls. 18x$\frac{3}{4}$	Carbon steel	54.0	48.0
M25-U26	127	284	194	605	962	4 Ls 6x6x$\frac{3}{4}$	1 Web 22x$\frac{3}{8}$	47.5	39.3
U24-L24	150	187	128	465	—497 243	4 Ls 4x4x$\frac{1}{2}$	2 Webs 22x$\frac{1}{2}$ Carbon steel	37.0	30.2
ALL OTHER	HANGERS	......	......	....	....	4 Ls 6x4x$\frac{3}{8}$ 1 Web 22x$\frac{3}{8}$	Carbon Steel	22.6	....
SUBSTRUTS	and U1-L1, U3-L3, M5-L5, M7-L7, U29-L29, U31-L31				....	2-15 in. channels at 33 lb.	Carbon Steel	19.8	.....

Column marked total includes 50 per cent. of smaller stress where there is reversal.

of the bridge being fixed, so that the end masonry served as anchorage against longitudinal motion. At the south end a new abutment was built back in the rock bluff, while at the north approach a new pier had to be erected and this could readily be proportioned to resist the traction forces.

Selection of the Continuous-Span Design

Essentially the adoption of continuous spans is the outcome of the designer's aim at producing the most economical structure. In the range of span length here in question, the differences in metal weight between simple, cantilever and continuous spans are in most cases equalized by differences in erection costs, when comparisons are made on the basis of normal prices of material and labor. But here the erection labor costs could not differ greatly, as the cantilever method was to be used in any event.

Simple spans would have required a considerable amount of extra material to take care of erection stresses. With cantilever or continuous trusses most of this extra material would be saved, adding to the inherent weight economy of these types. But a cantilever bridge, besides being undesirable for the short spans and heavy live-load, was not adapted to the requirement of erecting from one end of each span.

The choice as to how many spans should be connected up in a single continuous structure (from three to six; the short north approach span was never considered for inclusion in the continuous span system) was determined by the question of adjusting the reactions to their proper values. Grouping three spans in a continuous structure, only the two end reactions of the group had to be weighed off by jacks and gages to the predetermined amounts. With more spans grouped, for instance all six, as many as five pier reactions would have to be adjusted, and it was thought better to avoid this extra complication.

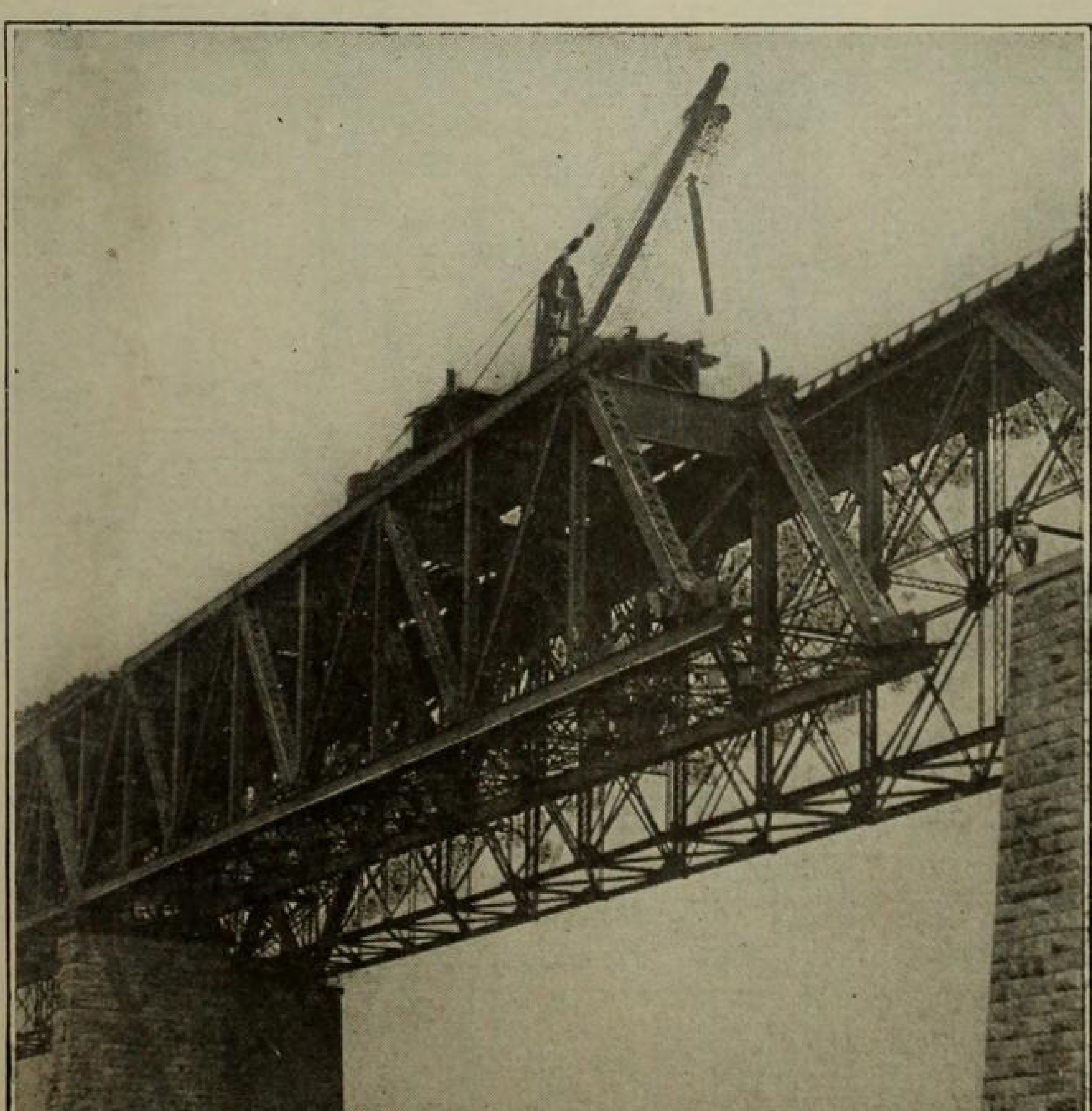


FIG. 5. LONG CANTILEVERING FEATURED THE WORK
Three spans of 350 ft. each were consecutively cantilevered in this way.

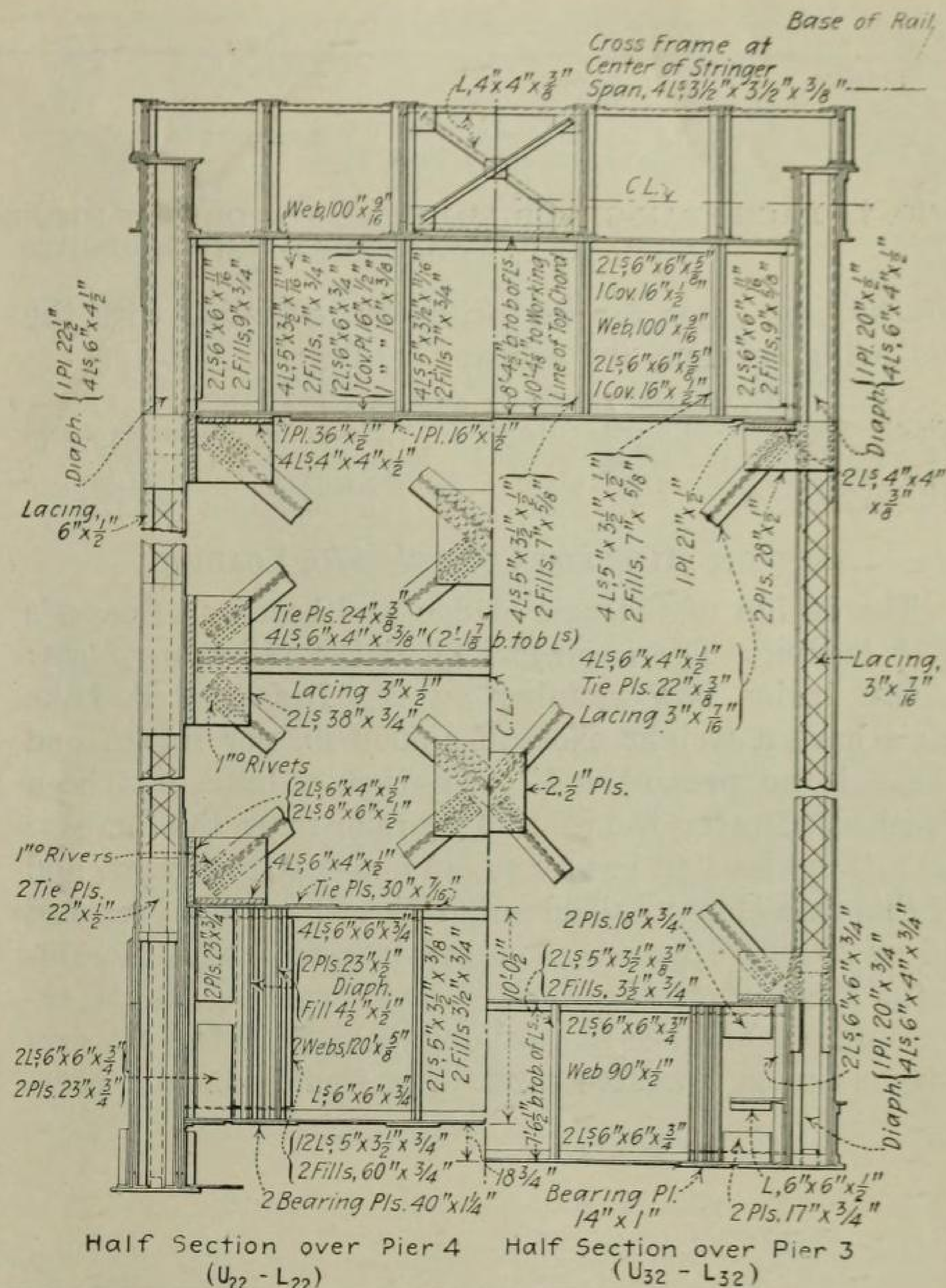


FIG. 6. JACKING GIRDERS AND MAIN SWAY-BRACING

As the erection procedure involved only 260-ft. cantilever projections in the 520-ft. span, the full-span projection of the three 350-ft. spans became the critical element of the erection. The first plan was to provide a falsework tower under each one of these spans, on which the cantilever overhang would be landed when erection had proceeded across one-half or two-thirds of the span length. However, when a design using high-strength steel was studied, it was found that the reduced weight made the full 350-ft. cantilever length practicable. A slight addition of metal in the end members of the spans for erection stresses was required, but the cost of the metal so added was less than the cost of a temporary bent, and represented a value permanently retained in the structure and adding materially to the rigidity of the completed bridge.

At the prices prevailing in 1914 the metal selected furnished also the cheapest bridge. Comparisons were based on American Railway Engineering Association unit-stresses for plain steel, 40% higher stresses for silicon steel, and 27,000 lb. per square inch for the eyebars.

Specially treated eyebars of high strength had been developed by the American Bridge Co. a short time before, and these were adopted for the Allegheny River bridge. The specifications under which they were furnished required them to show results in full-size tests equal to or exceeding the following figures: Elastic limit, 50,000 lb. per sq.in.; ultimate strength, 80,000 lb. per sq.in.; elongation, 8% in 18 ft.; reduction of area, 35%. Ten

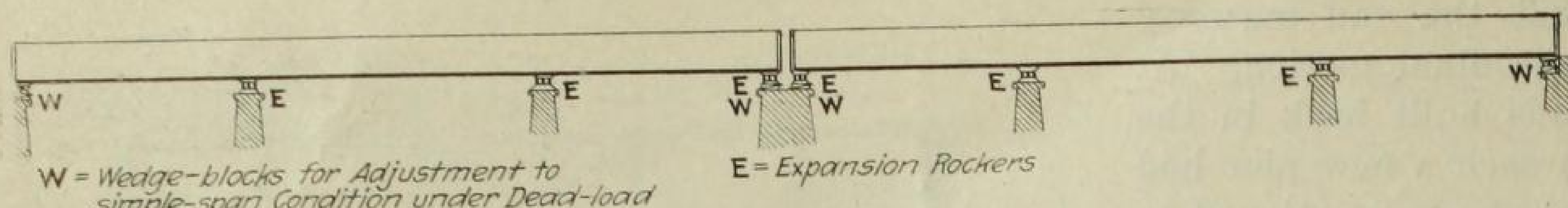


FIG. 7. EACH SPAN-GROUP ADJUSTED TO CORRECT STRESS CONDITIONS BY WEDGE BEARINGS AT END SUPPORTS

of the bars made for the bridge were tested, with the following extreme and average results:

FULL-SIZE TESTS OF SPECIAL EYEBARS

Elastic Limit Lb. per Sq.In.	Ultimate Strength Lb. per Sq.In.	Elongation In 18 Ft., per Cent.	Reduction of Area Per Cent.
53,000-63,000	81,000-90,000	7.8-13.2	46-59
av. 58,300	av. 86,300	av. 10.4	av. 52.5

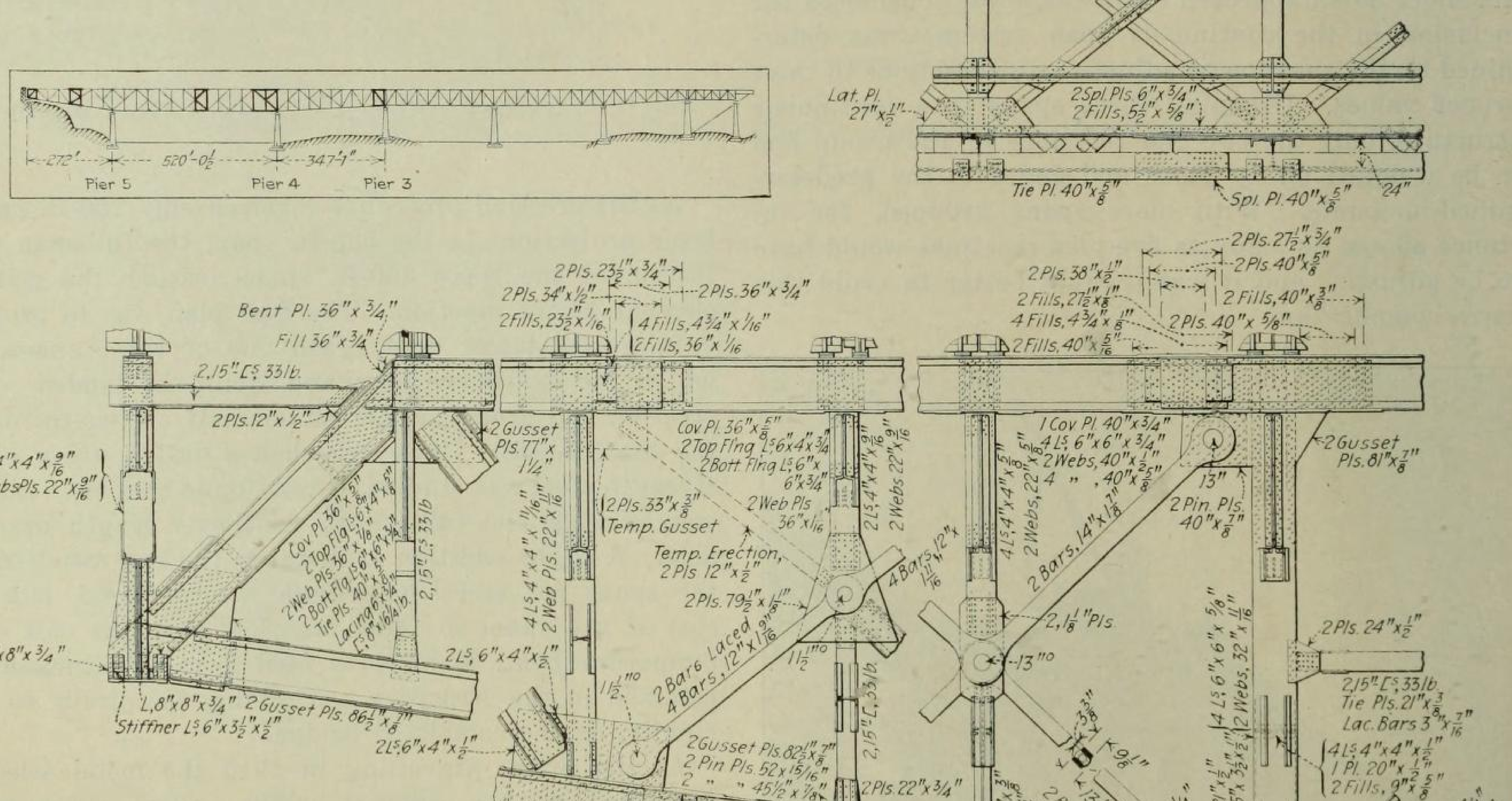
Design Made for Cooper's E75 Loading

The design of the new bridge was based on Cooper's E75 loading. The engineers cited the following facts:

Ore trains now running on the Bessemer & Lake Erie have a weight exceeding 5000 lb. per lin.ft., and there is no present indication that a limit has been reached. During the past 25 years or more the increase in railway loading has averaged about E1 per year, and if this rate is maintained an E75 loading will be reached 15 or 20 years hence. Another iron-ore road, the Duluth, Missabe & Northern, already carries trains exceeding 6000 lb. per ft. Solid trains of 100-ton ore cars may soon be operated, and they would amount practically to E70 loading. Engine loadings show at least an equal rate of growth. Passenger locomotives are now operating with 73,000 lb. on an axle; if the same axle loading should be adopted for freight locomotives the result would be not far from the Cooper E75 engine. The final adoption of E75 loading expressed the conclusion that it is a reasonable future contingency as to both engine and train weight.

Since railroad practice in general, apart from that on special ore-carrying roads, has shown a greater growth of engine weight than of train weight, some study was given to a departure from the Cooper type of loading making allowance for the greater probability of engine-weight increase. A loading consisting of two E75 engines followed by a train weighing 6500 lb. per lin.ft. was compared with a straight E75 loading. It was found that there would be little difference in stresses in this type of bridge, because most of the stress maxima occur under short load segments; little could be saved, therefore, by holding train loading to 6500 lb.

In designing the continuous spans a first approximation to the required distribution of metal was obtained by using ordinary continuous-girder formulas based on constant moment of inertia. The resulting design was then analyzed, corrected and re-analyzed by deflection calculations. Instead of taking the dead-load reactions at the amount corresponding to the condition of level supports, however, they were fixed arbitrarily at an early stage of the calculations, and the final adjustment



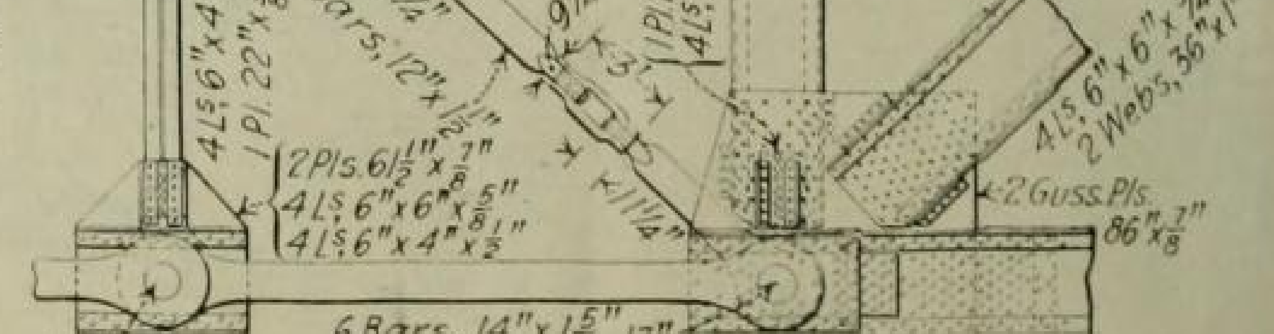


FIG. 9. TYPICAL DETAILS OF SOUTH SPAN-GROUP—DIFFERENCE IN LEVEL OF OLD AND

of the bridge after erection was accomplished by jacking up until the end reactions equaled those used in designing.

The dead load was assumed at 10,200 lb. per lin.ft. of bridge, consisting of 1900 lb. for the deck and 8300 lb. for the steel. For the live-load, impact was computed according to the A. R. E. A. specifications.

In continuous spans many members get their maximum stresses under broken load; i.e., two or more separated blocks of load. An unusual procedure was followed with respect to such loading. The stress due to broken load was not considered unless it increased the unit stress 25% beyond that allowed by the specifications, or unless it produced reversal of stress; in the former case, a 25% increase in unit stress was allowed in computing the section. The small probability of short lengths of train occupying just the required panels, with no load elsewhere on the structure, was at the bottom of this allowance.

In designing members subject to reversal of stress, it was assumed that the entire dead-load would be effective in counteracting live-load stress, which departs from the usual assumption intended to provide for reduced weight of deck, increase of live-load, and other effects. The member was designed on the basis of stresses found by increasing the maximum stress in each direction by 50% of the smaller. The connections in all cases were proportioned for the stress as thus increased, unless erection stresses required still more rivets.

Wind loads were assumed as follows: On the top chord, a distributed load of 30 lb. per sq.ft. of exposed surface of both trusses and the floor, and a moving load of 750 lb. per lin.ft.; on the bottom chord, a distributed load of 50 lb. per sq.ft. of exposed surface of both trusses. Wind during erection was taken at 30 lb. per sq.ft. of exposed surface. The unit stresses for combined wind and vertical load were increased 25% above

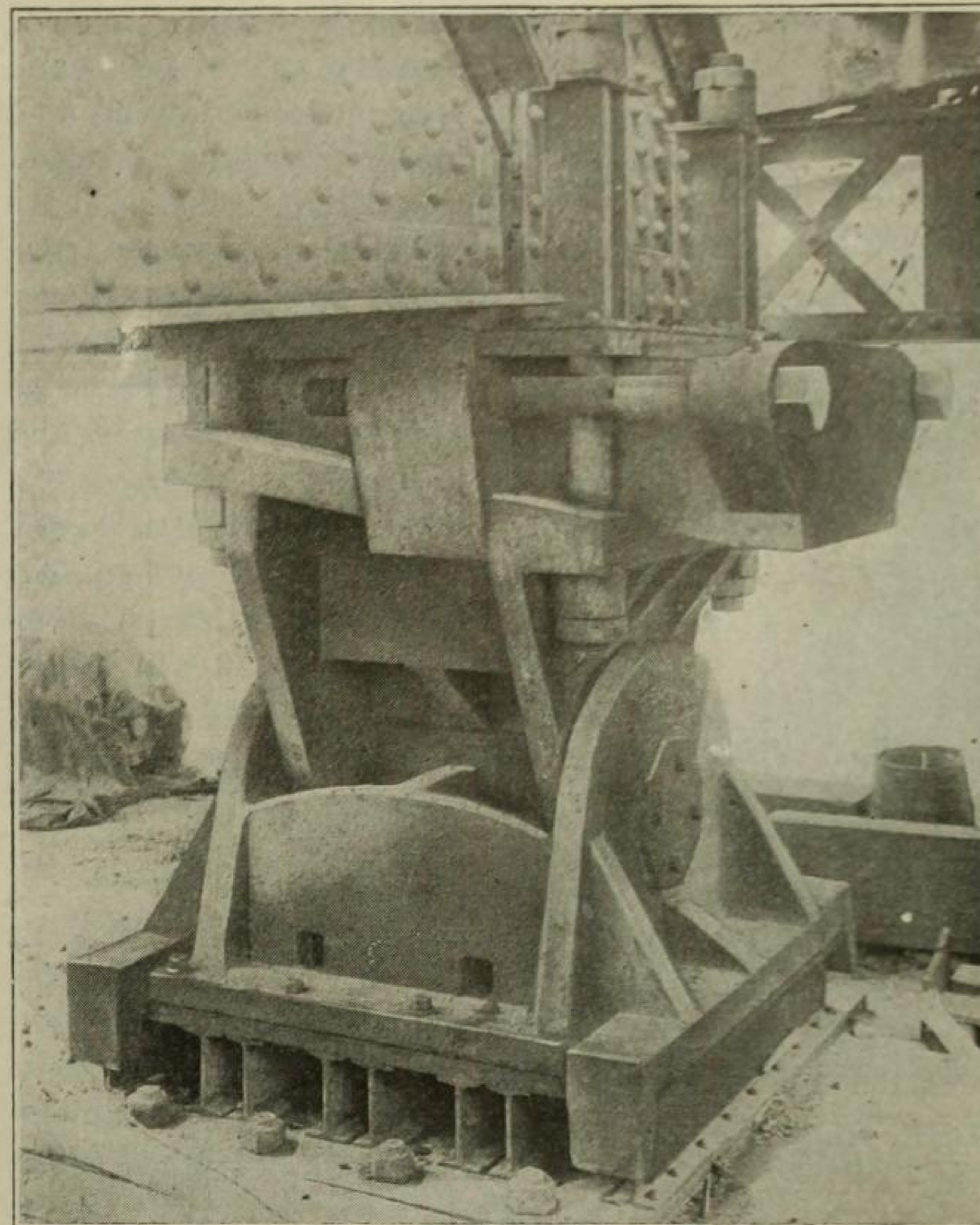
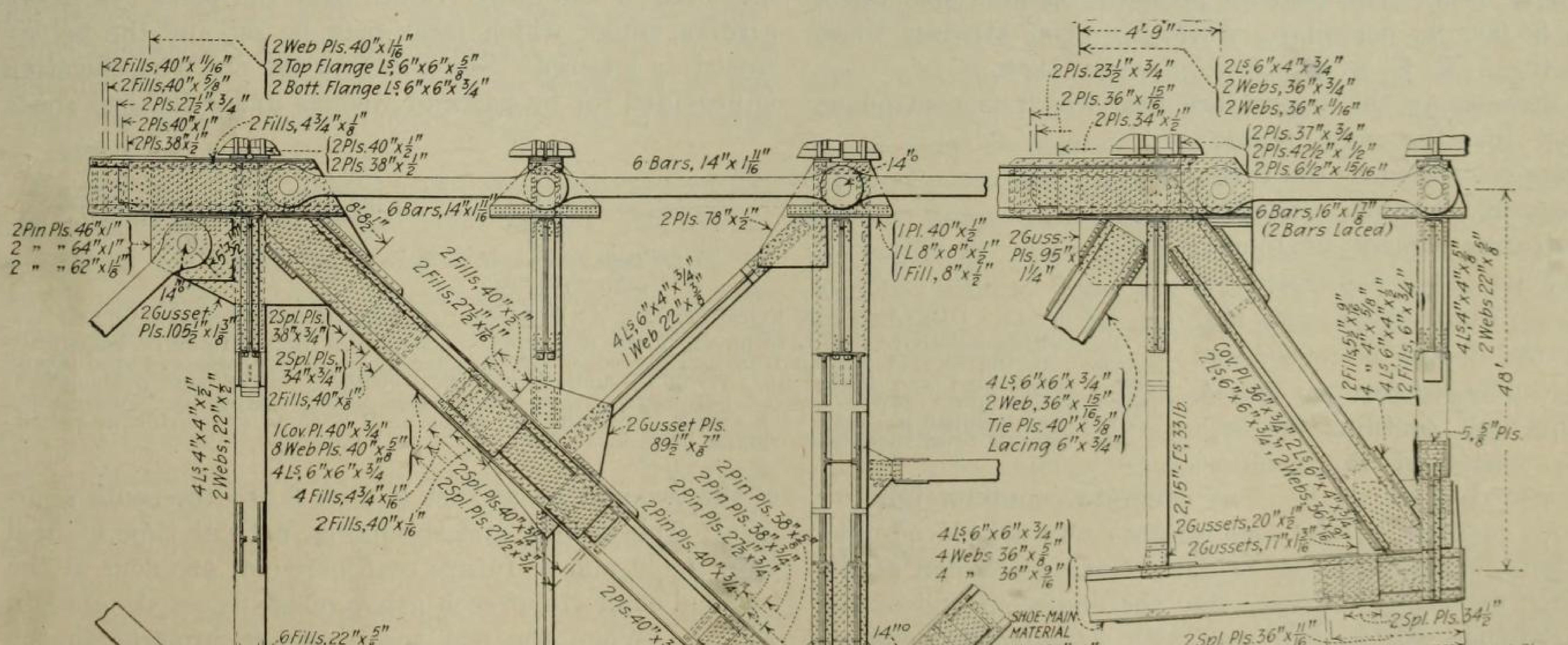


FIG. 8. FIXED-END SHOE WITH ADJUSTING WEDGES

When the three spans of a group were completely erected, hydraulic jacks were set under the jacking girders at the ends of the group; after lifting until the desired reaction is obtained, the wedges are drawn up.



COMPACT CONNECTIONS OBTAINED BY USE OF LARGE RIVETS
NEW MASONRY REQUIRED SPLICED PIER GIRDERS

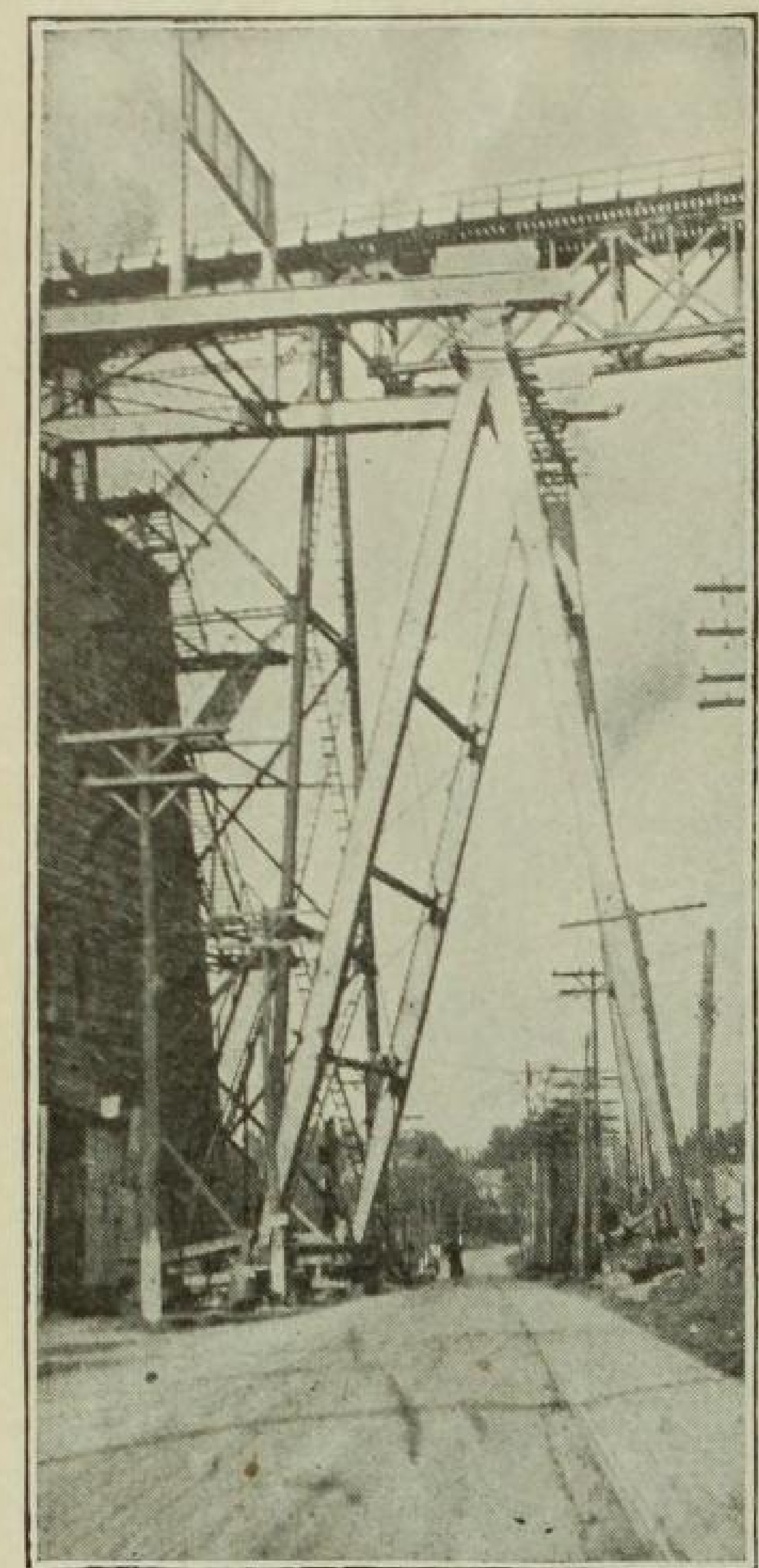


FIG. 10. SUPPORT OF NORTH SPAN DURING ERECTION

those normally allowed by the specification, and for combined wind and load during erection conditions, 50% above normal. The single-track deck used in cantilever erection was computed as weighing 500 lb.; with one pair of stringers and bracing, it formed the floor load on the cantilever. The live-load consisted of a derrick car taken as weighing 212,000 lb. When the new bridge is completed, the old bridge will be supported from it by bracket connections, while being dismantled; stresses due to this weight, with E50 loading on the far track of the new bridge, were considered erection stresses. Carbon steel was used wherever rigidity or minimum section governed: In vertical hangers supporting the bottom chord, horizontal struts (secondary members in the truss system), safety stringers, shoes, grillages, and all lateral and sway bracing. Members over the abutments also were made of carbon steel. The rivets (as already noted) and all pins were of the grades ordinarily used. The bearing pressure on pins was taken at 30,000 lb. per sq.in., while for rivet stresses those of the A. R. E. A. specification were taken.

Stresses in the lateral bracing, which is continuous just like the main trusses, were calculated by the three-moment formula, assuming uniform moment of inertia and no sheer deflection. A stress sheet of the south half of the bridge is reproduced in condensed form in Fig. 4. This half contains the 520-ft. span and the counter-weighted south span. The concrete counterweight already mentioned (350 tons weight) was not placed until after completion of the bridge. During erection of the south half of the 520-ft. span the abutment end of the shore span was loaded down with a temporary counterweight of steel rails weighing 120 tons.

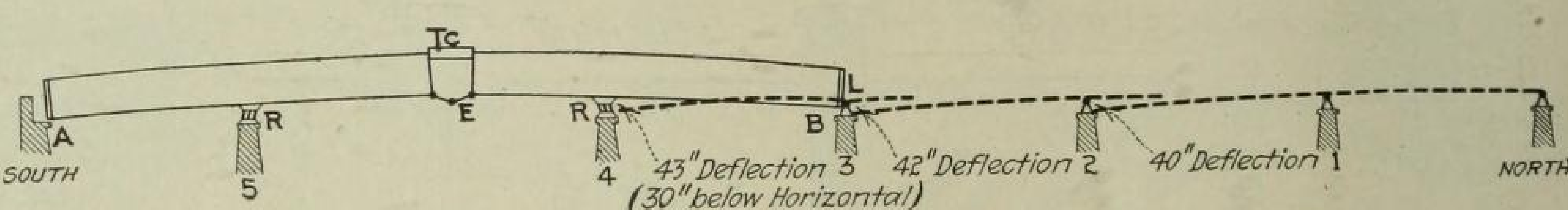


FIG. 11. DEFLECTION CONDITIONS AND FINAL CLOSURE OF CANTILEVERS

By making top-chord link L $1\frac{1}{4}$ in. short, the deflection at Pier 4 was limited to 30 in. below the horizontal, the total deflection from unstressed condition being 43 in. Link L was removed when span was landed and jacked up. To close south group, the cantilever halves were pushed forward and points A and B lowered. Eyebars E were slack when placed. After the closure section T was dropped in, jacks at A and B tipped the two arms until T came to bearing and then pulled bars E taut, rockers R coming back to vertical.

As in other cases where high-strength steel has been used there was some perplexity concerning the riveting. Search and experiments did not bring to light any material affording with definite certainty a higher rivet strength, so as to bring rivets and body material into the same relation as in work using ordinary steel. The only way to avoid ungainly and expensive connections was to use large rivets, and therefore $1\frac{1}{4}$-in. rivets (of ordinary rivet steel) were adopted for the main connections.

An unusual rivet-hole specification was written, because of the use of silicon steel and extra large rivets. The intent of the specification is to limit the material punched to a thickness $\frac{1}{16}$ in. greater than the punch diameter, and at the same time have $\frac{1}{8}$-in. of metal left for reaming ($\frac{1}{4}$ in. on the diameter). The clause reads as follows:

> Material ½-in. thick or less shall be punched full size. Material over ½-in. thick, but not more than ¼ in. less than the nominal diameter of the rivet, may be punched to a diameter $\frac{3}{16}$ in. less than the nominal rivet diameter and reamed to $\frac{1}{16}$ larger than the nominal rivet diameter. Material thicker than above specified is to be drilled from the solid. No material over 1⅜ in. thick is to be used. The edges of all main gusset plates are to be planed.

Adjustment to Correct Stress Conditions

During erection, as soon as a span was landed it was jacked up to correct level—or rather to a level 8 in. above that shown by the drawings, so as to give room for inserting the track and rollers on which the new bridge is to be rolled along the piers into final position after the old structure shall have been removed. But adjustment to design elevation only approximated the desired condition of correct distribution of the reactions, as the assembly of the members is likely to make the shape of the structure depart slightly from the diagram shape. In the final adjustment, therefore, the jacking was governed by reaction rather than by amount of lift.

The reactions produced by the jacking were equal to those used in the design calculations. These were approximately but not exactly the same as the reactions corresponding to true continuity on supports exactly fitting the unstressed shape of the bridge. To allow readjustment in case of any subsequent pier settlement, provision is made for jacking and shimming. The lower members of the sway frames over the piers are heavy girders, under which jacks may be set and the bridge raised or lowered. Shimming to the changed elevation is provided for by pairs of wedges under the end shoes of the two groups, at Pier B, Pier 3 and the south abutment. The jacking and shimming, both at time of final adjustment and at future readjustment, are done at the two end supports of each group of spans, as sketched in Fig. 7. When the end reactions are brought to the proper amounts the middle reactions will also be right. However, jacking must be done at the intermediate supports in connection with the transverse rolling of the bridge, to free the blocking, lower the bridge down upon the roller track, later free the rollers and track and lower to final bearing. Jacking girders are therefore provided here as well as at the end supports (Fig. 6), but weighing the jack loads is to be done at the end bearings only. The operations of rolling the bridge to permanent position and making the final adjustment have not yet been performed.

Typical details of the bridge are contained in the panel-point elevations Fig. 9 and the cross-sections Fig. 6.

The designers aimed to use the maximum economic panel-length. In the long span the length was partly controlled by the desirability of having an odd number of main panels, to get the simplest conditions for central closure. The width of the deck, 31 ft., was determined partly by consideration of convenience of providing for the lateral plates outside of the outer track stringers, and partly by the desire to make provision for future gantleting of tracks, so as to avoid switching on the approaches when four line tracks are put in. Ties long enough to extend over the entire width of deck are used, of 10 x 12-in. section.

A safety stringer was placed at the outer edge of the deck, directly over the top chord. There is only one set of stringer laterals, placed between the center stringers. In erection, the cantilever construction was carried out with only one pair of stringers (the middle pair) in place, in order to reduce the weight of the overhang; as these stringers required a lateral system in order to carry the derrick-car loads safely, the adopted arrangement avoids subsequent shifting of stringers or laterals. The cross-frames which brace the unsupported stringer flanges are carried out to the edges of the deck and here connect with a safety stringer along each edge.

The shoes at the four points of end bearing are of cast steel, in conformity with normal practice. At the intermediate bearings, however, no separate shoes are used, but the truss panel-point serves as upper shoe and grillage on the pier girders as lower shoe. The lower and upper roller plates which inclose the rockers are in direct contact with these two parts. The panel-point assemblage being pin-connected to four of the abutting members, it was considered a degree of adjustability practically equivalent to that of a pin-shoe bearing is obtained.

Expansion on the middle pier must take care of the entire 2200-ft. length of the river bridge. Using the customary allowance of 1 in. expansion per 100 ft. length of bridge, 22 in. of expansion clearance is provided, half at each of the two adjoining shoes. The rockers are unusually large, being 24 in. in diameter.

Fabrication of the bridge material involved no special features beyond those introduced by the rivet-hole drilling-and-reaming specification already noted. The truss members were built to full-load camber, each member being made of a length equal to its geometric length plus or minus the full-load compression or tension respectively, with a view to producing the true diagram outline under full load. The angles between the members at the connection points, however, were made as required for connecting up in the unstressed condition; that is, there was no attempt to eliminate secondary stresses. The trusses were in general not assembled in the shop yard for making the connections, but the gusset plates were drilled to iron templets.

Erection was handled by a 60-ton derrick car of the Mitchell type. The choice of equipment was partly controlled by the necessity of keeping the load on the overhanging spans in the cantilever erection down to a minimum in order to avoid excessive erection stresses.

The cantilever erection progressed from either end to closure at the middle of the 520-ft. span. This meant that the north span of the south group had to be erected from the north, or in other words had to be continuous with the north group for the time being. The continuity was brought about by eyebar links between the adjoining top chords, over Pier 3, and blocks between the bottom chords, which were removed as soon as the span was landed on Pier 4.

The links just mentioned were made short enough to hold the span 13 in. higher at Pier 4 than it otherwise would be (Fig. 11). The deflection of the span just before landing was 43 in., but the short links by tipping up the span held the end to 30 in. below the horizontal, giving more clearance between trusses and pier for placing the jacks. In jacking up here to release the links and blocking, however, the full 43-in. height had to be jacked, and the truss then lowered 13 in.

Large deflections occurred in the previously erected spans also. The second span from the north deflected 40 in. just before landing on Pier 2, and the next span deflected 42 in. at Pier 3.

In preparing for the cantilever work in the 520-ft. span, which was to close at midspan, the rocker bearings on Piers 4 and 5 were rendered fixed by bolting on interlocking side-plates. Just before this the two halves of the structure were pushed forward several inches, and the rear ends lowered to tip the forward ends up, so that when the meeting cantilevers came together they were too close at the bottom chord and too far apart at the top chord.

With this preparation, closing the span was an operation free from difficulty. The bottom-chord eyebars of the middle panel, which are jointed at the middle sub-panel point, were inserted and allowed to hang slack. The top-chord closing section was then dropped into place, and jacks were at once set to work raising the anchor ends of the arms. As soon as the top-chord joints came to bearing as a result of this jacking, the rocker joints on the main piers were released by taking off the fixing plates, so that the trusses might move backward at these points as the jacking continued. Further jacking drew the slack eyebars taut, and put them under tension corresponding to the desired simple-span condition.

There were some complications in the erection of the north anchor span on falsework due to local interferences. Two railway tracks pass under the span, and as traffic on them could not be interrupted a gap in the

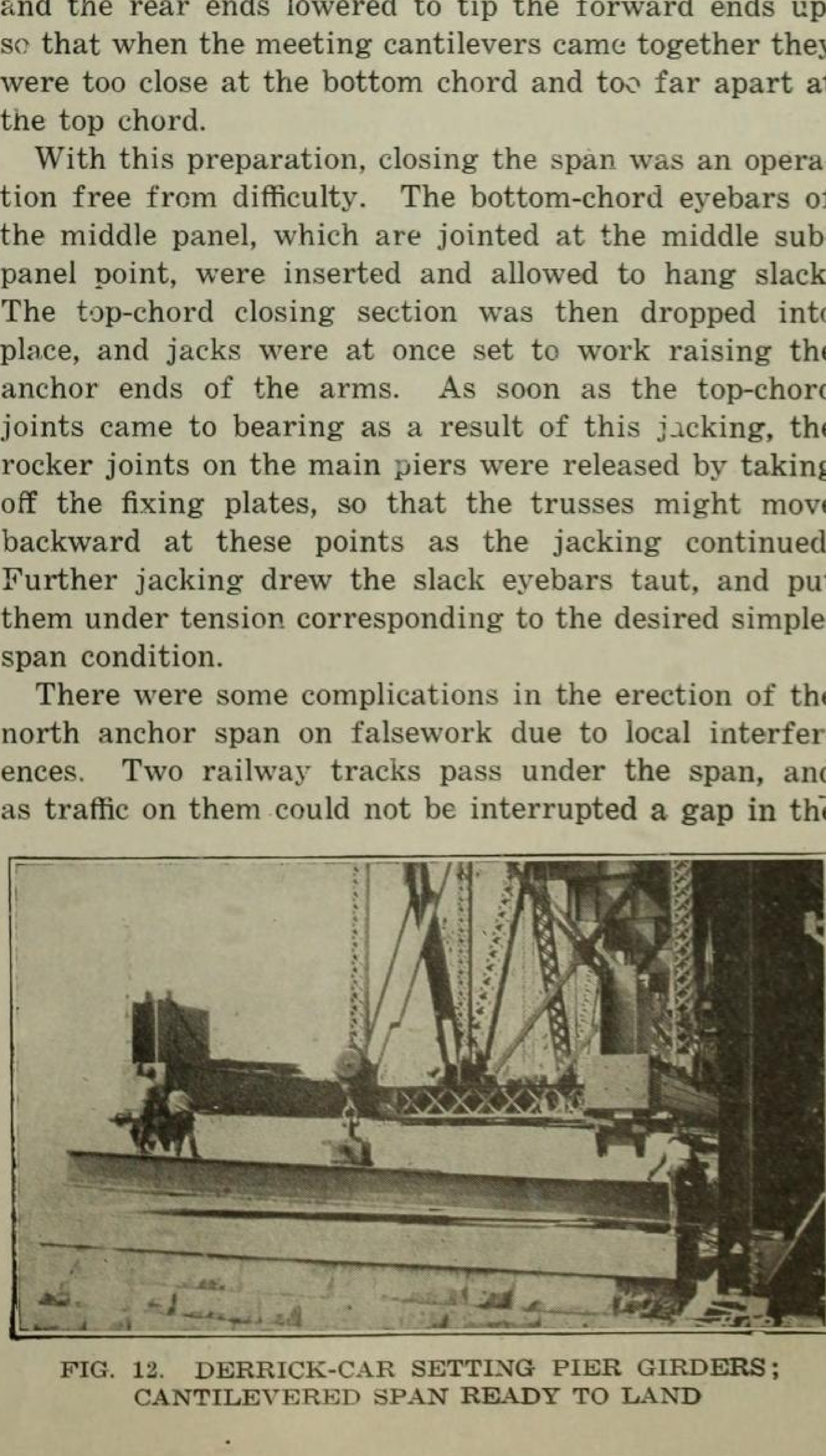


FIG. 12. DERRICK-CAR SETTING PIER GIRDERS; CANTILEVERED SPAN READY TO LAND

falsework resulted which had to be crossed by cantilever erection, requiring a steel fulcrum tower (Fig. 10) and a temporary tail counterweight.

Old Bridge Must Be Removed and New Structure Rolled to Final Position

The position in which the new bridge was erected, and its relation to the old bridge, may be seen in the cross-section sketch, Fig. 2. As the pier width is barely sufficient for both structures, pier girders (see Fig. 1) are used to prevent load concentration near the ends of the masonry and to tie the new and old masonry together. The same girders will carry the roller track required for shifting the new bridge to final position.

Before this moving the old bridge will be taken apart, after each panel-point is supported by connection to the new bridge through beam brackets and diagonal-rod hangers. Following this operation the old half of each pier will be built up to final coping elevation (made necessary by the change of grade), and the remaining sections of grillage girders placed, before preparation can be made for moving.

Rolling the new bridge to place will follow closely the precedent established by recent bridge-rolling operations. As the entire length of the movement will be on masonry, the increased resistance and frequent delay encountered when part of the support is on timber falsework is expected to be absent. The two groups of three spans will be moved successively. The shift is almost exactly equal to the track spacing on the new bridge, and it will therefore be possible to resume operation, if desired, when only one group has been moved. On the other hand the moving of the north group will be complicated by the necessity of skewing the end spans of the filling viaduct at the north approach (the filling track, at the grade of the revised profile, being then used temporarily as running track) to make connection between this viaduct on the line of the erection position and the river bridge in the final position, until the approach fill can be built up to grade.

The steel weight of the new bridge is about 10,150 tons, or 8700 lb. per lin.ft.

Both fabrication and erection were carried out by the American Bridge Co. Erection began in April, 1917, and the long span was closed February, 1918. The new bridge has carried regular traffic since Mar. 1 in its temporary position.

Designs of Workmen's Dwellings Exhibited

Industrial dwellings of the "model city" at Morgan Park, Minn., are a feature of an architectural exhibit at the Art Institute, Chicago, one room being devoted to a display of plans, photographs and paintings of the town and its buildings. This town was established a few years ago by the Minnesota Steel Co. to provide accommodation for employees at its large steel plant near Duluth. The buildings are of concrete block construction and range from single residences to long structures forming three or four residences. Wide streets with a curved layout and ample provision for gardens, playgrounds and other open spaces are characteristics of the general plan. The designs were made by Dean & Dean, of Chicago, architects for the Morgan Park Company.

Wood Tieplates Found to Be Poor Substitute for Steel

ECONOMY in track construction was the goal of extensive experiments begun some years ago in the use of wood instead of steel for tieplates to protect ties from being cut and abraded by the rails. These plates were used mainly in connection with treated ties, but were also used to some extent on untreated ties. One special advantage suggested was their application on branch lines, thus protecting the ties at much less expense than the use of steel would involve. This feature of track work was dealt with in *Engineering News* of June 21, 1906, page 694. Recent inquiry indicates that on the whole the plates have not been satisfactory, and have not given the expected service.

About 65,000 creosoted wood tieplates were applied in 1906 by the Cleveland, Cincinnati, Chicago & St. Louis (Big Four) Ry., and in 1917 only about 20% of these remained in service. They were mainly of beech and elm, 5 x 8 in. and ⅝ in. thick, fastened to the ties by sixpenny nails. Their failure has been due almost exclusively to splitting after five or six years service. Some of the plates were also worn or abraded. While they protected the ties as long as they remained intact, they did not prove as efficient as was expected. In 1907, about 500 creosoted white-oak tieplates 6 x 7 x ½ in., averaging ½ lb. of creosote each, were laid on a test track of the Northern Pacific Ry. in Montana. These were all removed in August, 1910. In service these plates soon split badly and worked out from under the rails, and they also cut into the softwood ties.

Creosoted Wood Tieplates Proved Unsatisfactory on Western Railroad

Creosoted wood tieplates were used extensively at one time on the Gulf, Colorado & Santa Fe Ry., but the results did not come up to expectations. The plates furnished were not all applied, as their replacement with steel plates was soon begun. That the results were somewhat unsatisfactory, however, is attributed to inadequate attention to the manner of carrying on the experiment. The main defect, and the one considered responsible for failure, was the omission of means of seating and fastening the plates properly. Unless the ties are adzed to give recessed and level bearings in which the plates may be inserted, the latter are likely to split and shift. At first they were secured only by the track spikes, but as they showed a tendency to creep in the direction of rail, each plate was afterward fastened to the tie by two small nails.

On this road the wood plates were regarded as a temporary experiment, and it was estimated that their life would be only a quarter to a third of that of steel tieplates. Furthermore, they were not advocated for main-line track carrying heavy traffic. But for branches and lines of medium traffic it was considered that if hardwood plates were inserted and fastened on ties properly adzed to receive them they would much more than pay for their cost by preventing the rails from cutting the ties. The cost of the plates in the track was about 1c. each, as compared with 0.6c. for those of the Cleveland, Cincinnati, Chicago & St. Louis Railway.