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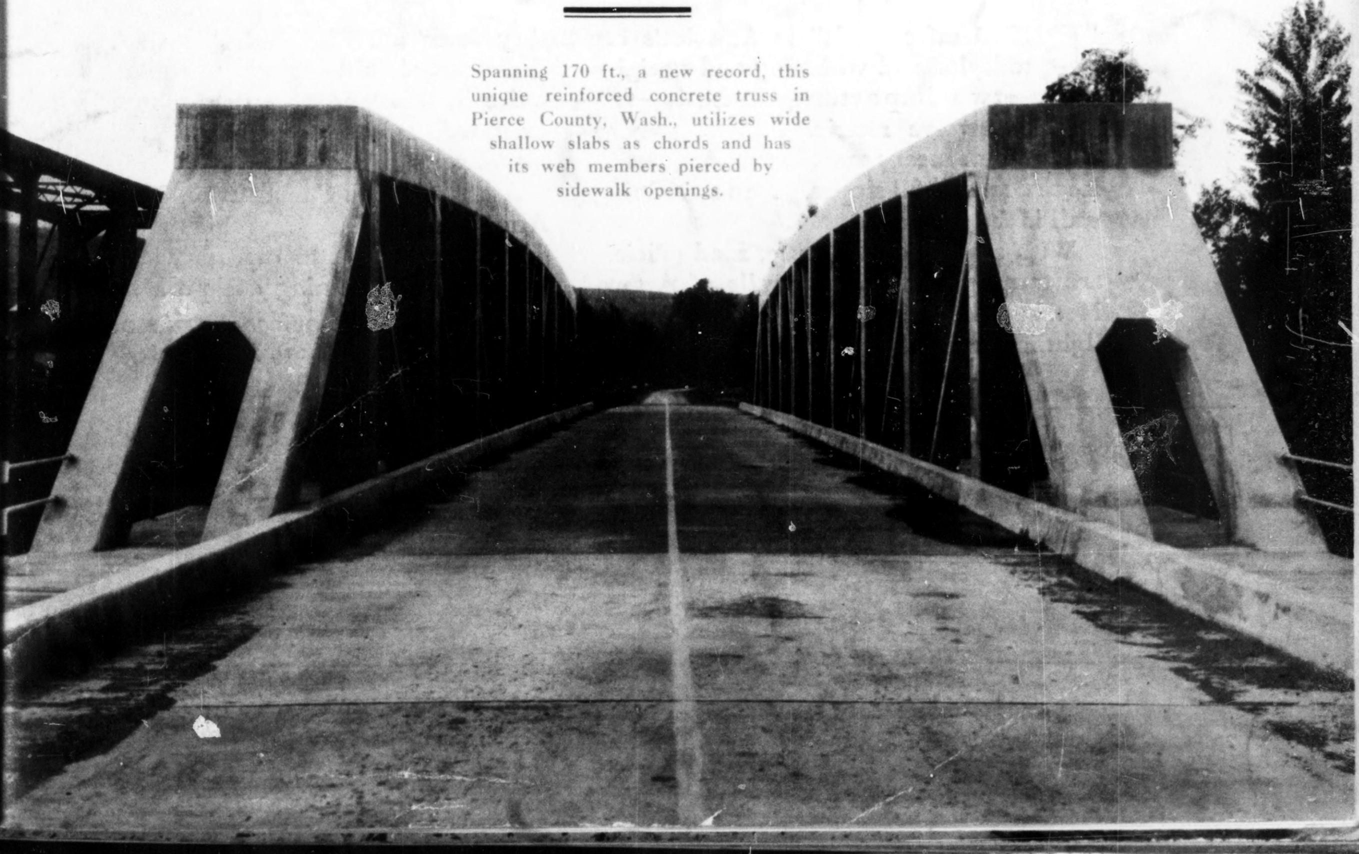

Spanning 170 ft., a new record, this unique reinforced concrete truss in Pierce County, Wash., utilizes wide shallow slabs as chords and has its web members pierced by sidewalk openings.

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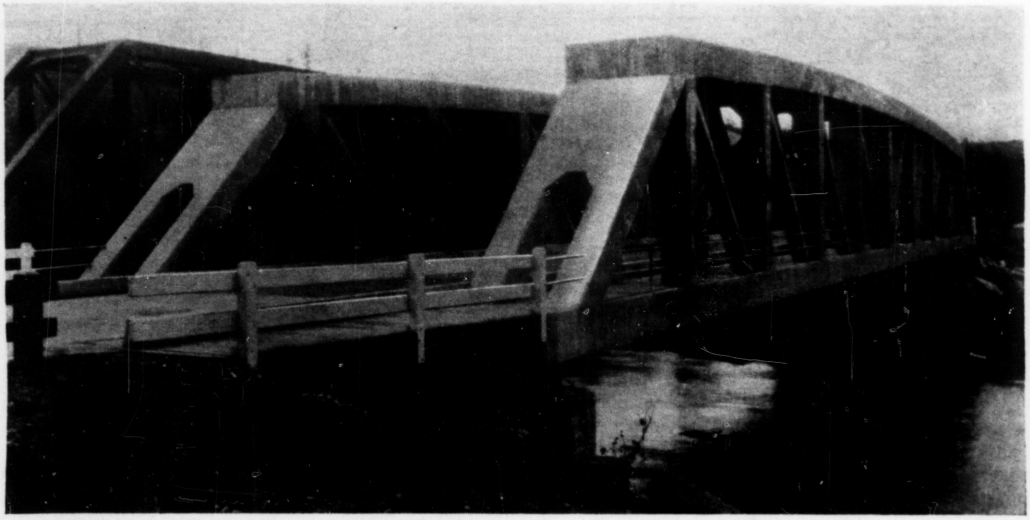


FIG. 1—COMPLETED STRUCTURE of new McMillin Bridge near Tacoma, Wash. Reinforced-concrete main span, 170 ft. long, provides largest practical water opening for Puyallup River floods.

Through Concrete Trusses, 170 Ft. Long Used on Low-Cost Highway Bridge,

Unique structure near Tacoma, Wash., built to replace old steel bridge, is longest concrete truss or beam span in United States —Sidewalks accommodated by openings through truss members

By W. E. Berry and Geo. Runciman
Former Pierce County (Wash.) Engineer and President, W. H. Witt Co., Respectively

THE recently completed McMillin Bridge, 15 miles southeast of Tacoma, Wash., has as its main span the longest reinforced-concrete span, exclusive of arches, that has been built to date in the United States. It employs a through truss of novel design whose breadth and stiffness are such that lateral bracing of the trusses above the roadway is dispensed with. Sidewalks on both sides of the roadway are neatly handled by running them through the trusses on their longitudinal center lines. A most important feature of the design is its simplicity from a construction standpoint. The bridge was built of concrete because alternate bids taken on designs of both structural steel and concrete showed the concrete bid to be $826 less than the lowest bid on steel.

During the winter of 1933-34 a flood in the Puyallup River undermined an abutment of the old 150-ft. steel span, known as the McMillin Bridge, between the towns of Sumner and Orting in Pierce County, Wash. The highly precarious condition in which the bridge was left, as well as its narrow 16-ft. roadway, made a new bridge necessary. Like all rivers in the Puget Sound Basin, the Puyallup is subject to frequent severe floods, and bridges in its lower valley require the largest practical waterways and so must be of through type.

For the new bridge, which adjoins that of a branch of the Northern Pacific Railway, it was decided to increase the span to 170 ft. and to provide also short 20-ft. approach spans at both ends. The piers were taken down about 15 ft. below the ground where they are supported by steel H-piles.

Consideration of various suggestions advanced led to plans being prepared and alternate bids being taken on designs of structural steel and reinforced concrete. The steel layout was of standard type, commonly used for light highway work. It provided a roadway 24 ft. wide from curb to curb, with a lateral clearance of 25 ft. 6 in. between trusses and a vertical clearance of 15 ft. below the top lateral system. It had no sidewalks. The concrete layout had a 22-ft. roadway (curb to curb), 24-ft. clearances between trusses, unlimited vertical clearance, and 3-ft. sidewalks on both sides of the bridge. Since money received from state gas-tax funds was used in the work, plans for both designs were approved by the bridge division of the Washington state highway department.

Bids were opened on Aug. 23, 1934. Six bids were offered on the steel design, and only one on the concrete. The

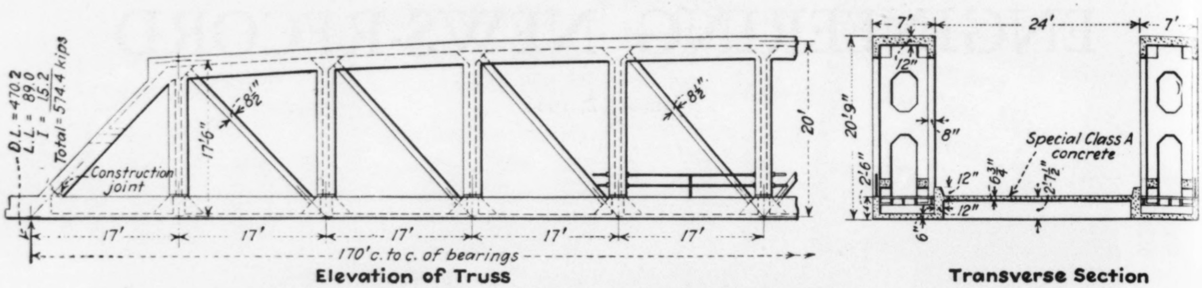


FIG. 2—ELEVATION and cross-section of through concrete truss span, the longest in the United States. Design provides for eliminating lateral bracing of trusses above the roadway.

lateness of the season and the hazard of fall floods deterred several contractors from quoting on the concrete. The steel bids for the main span and approach spans, constructing and maintaining temporary detour bridge, removing old bridge, etc., ranged from $36,738 to $48,250. The lowest bidder on steel also submitted the concrete bid of $35,912. As this was $826 lower than the steel bid and as there were maintenance advantages as well, the contract was awarded for concrete.

Work was started in September after the old span had been shifted downstream 32 ft. and had been provided with wood approaches to serve as a detour during construction. A heavy flood in October caused the work to be shut down for the winter. It resumed in April. The bridge was opened to traffic last September.

Design assumptions

In designing this bridge it was assumed that: (1) Joints of the trusses were free and without restraint; (2) the trusses were loaded axially; and (3) with the rich concrete employed, cracks in the concrete encasement of tension members would be of no practical significance.

Concerning (1) it should be said that the diagonal members were not given their concrete encasement until all shoring had been removed and they were subjected to practically their full stress by the dead load; that construction joints at the top and bottom of the verticals offered opportunity for slight elastic adjustments; that it was anticipated that deflections would be slight; finally, that on completion of the bridge a few fine cracks in the upturned outer flanges of the bottom chords—cracks for which ordinary shrinkage in the concrete may have been largely responsible—were the only indications of impropriety in an assumption which was here quite as convenient as it is in the case of joints in riveted steel trusses.

Assumption (2) departs from actuality in the fact that although 60 per cent of the dead load is in the trusses, the remaining 40 per cent is in the roadway. This roadway dead load, together with the live load, is delivered to the inner side of the trusses. This eccentricity is real and unavoidable and is not to be circumvented merely by designing the floor beams on an assumed span c. to c. of trusses. Recognizing this condition, the reinforcement of the panel-point floor beams was extended to the outer side of the trusses, diagonal steel was provided over the doorway openings in the verticals, and then the assumption was made.

The validity of the third assumption is attested by the condition of tension members in a considerable number of through tied concrete arch bridges which have in the past ten years been built in the northwest.

Loading and design details

The bridge is designed for H-15 loading, A.A.S.H.O., with 30 per cent impact on floor slabs and 16 per cent on trusses.

The wide trusses would have greatly increased the up and downstream dimensions of piers constructed as a unit in conventional manner. Consequently, the equivalent of individual pier shafts was built for each truss, and these companion shafts at each end of the span were joined at their tops by deep connecting diaphragms. To save weight and concrete, the pier shafts, which are of rectangular section with truncated corners, are hollowed with two 3-ft.-diameter circular shafts, from top of base to a point 3½ ft. below the truss seat. Steel H-piles extend an average length of 25 ft. below the bottom of the footings. The foundations at the north end proved to be softer than anticipated, and therefore additional wood piles were driven under each shaft, and the steel piles were increased in length to 40 ft.

The length of span required that dead load be kept to a minimum. Since the standard Class A mixture for bridge decks in Washington employing 1.81 bbl. of cement per cu.yd., is assumed to develop a cylinder strength of 3,000 lb. at 28 days and is permitted a maximum stress in flexure of 1,000 lb. per sq.in., the use of higher stresses logically required a richer mix of concrete. Therefore, in the deck and in the top chord of the trusses a 2-bbl. mix was used with a flexural working stress of 1,200 lb. per sq.in. and a direct stress of 900 lb. per sq.in. Elsewhere in the span Class A concrete was employed. A maximum-size aggregate of 1½ in. was used everywhere except in piers, which used 3-in. aggregate.

In elevation the trusses are of Pratt type, with vertical web members in compression and diagonals in tension. The span is 170 ft. c. to c. of bearings, divided into ten panels of 17 ft. The top chord is curved, the height c. to c. of chords at the first panel from the end is 17½ ft. and at midspan 20 ft.

In section the novelty of the truss is revealed. The top chord is 12-in. slab, 7 ft. wide, having 8-in. flanges at its edges, extending 18 in. downward from the bottom of the slab. The bottom chord is a 6-in. slab with similar flanges turned upward. The verticals are 8-in. wall sections holed out as shown in one of the accompanying views, with 8-in. flanges at both sides to give an overall dimension of 24 in. The diagonals are in pairs in each panel, framing into the top and bottom chords and into the verticals through large 45-deg. fillets or brackets.

All tension members are formed of reinforcing bars, of intermediate-grade steel. Bottom chord and some diagonal bars are 1½ in. square. On account of the long lengths required, welding of chord steel was necessary. Welds were made by butting the bars, placing 2¼x½x9-in. plates on the sides, and making four ⅜-in. fillet welds, top and bottom, at the junction of the plates and bars. Midspan bars which did not have to extend through to the supports were terminated by extending them at least 5 ft. beyond the panel point where they ceased to be needed, and then welding them to adjoining bars. At the end joints the chords were made of full solid section to the top of the upstanding flanges, i.e. 30 in. deep. The longitudinal bars were anchored by bending them in a full semicircular sweep up to the top of the chord and then returning them downward to the bottom at an angle of about 30 deg. directly beneath the main end posts of the truss.

Anchorage of the diagonals is obtained by bond only. Embedment of at least 40 bar diameters from the point at which the bars enter into the large fillets is everywhere provided. Addi-

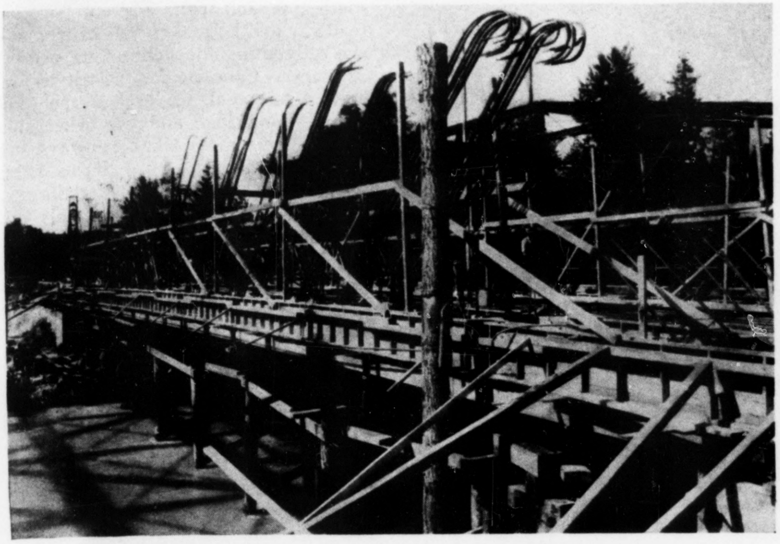


FIG. 3—REINFORCEMENT of truss diagonals, which are placed in pairs in each panel and frame into the top and bottom chords and into the verticals through large 45-deg. fillets or brackets.

tional anchorage is obtained from the large bends and sweeps at the ends of all diagonals and from the clamping compressive effect which the pressure in the verticals produces. At the end of the top chord where junction with the end posts is made, the joint is enlarged to provide room for the exceptionally large full hooks in which the bars of the last diagonals terminate.

An additional half-inch of thickness was added to the floor slab for wear. As chains on automobile tires are seldom, if ever, required anywhere along the Pacific Coast, wear on rich-mix bridge floors is practically non-existent, but in conformity with standard specifications it is carefully guarded against.

Besides the half inch of extra thickness required on the floor slabs for nonexistent wear, the amenities of conventional design were further observed by including an allowance in the dead load of 15 lb. per sq.ft. for surfacing, presumably to be applied when the extra concrete has worn away.

Commendable as foresight ever is in engineering work, it would seem that the gain in strength which comes to concrete in times subsequent to the 28-day age admitted and recognized by the specifications might safely be substituted for one or for the other or for both of the above safeguards.

The floor beams of the roadway are flush with the bottoms of the trusses. The truss verticals are simple wall sections. The top chord is a wide slab, practically horizontal, with simple marginal flange beams.

The span was given, more or less arbitrarily, a 6-in. camber, to cover deflection and to preserve as well a slight upward curvature in the bottom chord. A maximum deflection of $1\frac{7}{16}$ in. occurred at mid-span when the bridge became self-supporting.

The bridge is seated on the piers by means of steel bearings and rockers of conventional design. Had these been painted instead of being cadmium-plated as was required, they would not have cost 10c. per pound for the 8,850 lb. of metal so treated, and the concrete alternate would have shown a greater saving over the steel design than it did. Upon the removal of the falsework the panel points adjacent to the ends deflected an average of $\frac{3}{8}$ in. This deflection in the panel length of 17 ft. represents an angular change of 6 min. 5 sec., which is not a large amount. It would appear that plain steel bearing plates separated by asbestos packing as originally proposed would quite effectively have adjusted to such an angular change; they would, as shown by tests reported in *ENR*, June 25, 1931, p. 1058, have functioned very satisfactorily in allowing slight horizontal movements; and they would have been much less expensive to install, and in case of earthquake would be vastly superior structurally.

Weights and loads

The end reaction of one truss is:

	Lb.
Dead load	470,200
Live load, including impact	104,200
Total	574,400

Dead load thus constitutes 82 per cent of the total load for which the trusses are designed, and in view of the somewhat problematical nature of the impact allowance on a heavy structure of this character as well as the ample live load assumed, it is quite proper to say that for every pound of live load and impact there are 5 lb. of dead load.

What factor of safety against failure by exceeding the total design load there may be in these trusses cannot be definitely stated. However, as is generally the case, they are designed with the same maximum stresses for both dead and live loads. Probably the factor of safety is 4 or 5; but assuming it to be only 3, the following interesting relationship obtains:

Ultimate load = 3 live load + 3 dead load = 3 live load + (3) (5 live load) = 18 live load.

Then external load to be applied to produce failure = ultimate load—dead

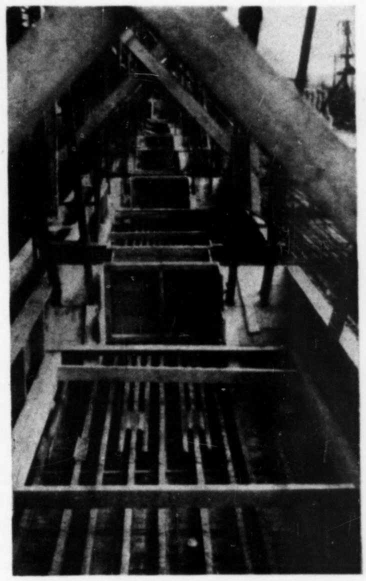


FIG. 4—TRUSS FORMS and reinforcing. Chord steel, before concreting, is spliced by welded plates.

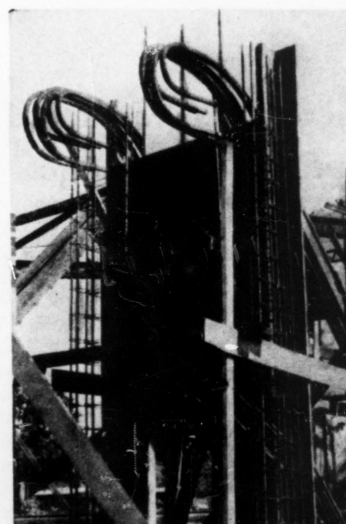


FIG. 5—DETAIL of first vertical, showing reinforcement and interior forms.

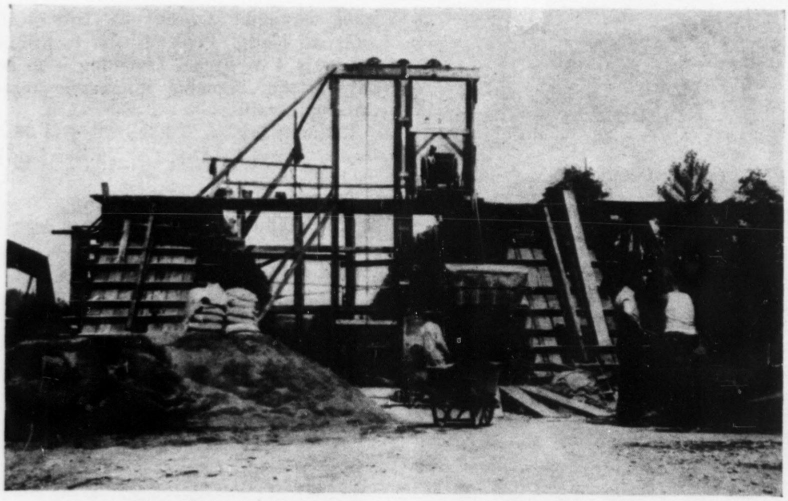


FIG. 6—CONCRETE MIXING PLANT and hoist. Mixing was performed with a 2-sack machine on the south approach span, concrete being wheeled along runways on the top chord.

load = 18 live load—5 live load = 13 live load.

If the factor of safety is greater, or if the ratio of dead load to live load is greater, then instead of the application of 13 times the design live and impact loads being required to produce collapse, a much greater load is required. For instance, with a factor of safety of 4 and a ratio of dead to live load of 6, it is 22 times the designed live and impact loads that must be applied to produce collapse. The grave apprehensions sometimes entertained lest the safety of heavy concrete bridges be jeopardized by unusually heavy trucks passing over them may be allayed or may at least be alleviated by the above considerations.

Order of construction

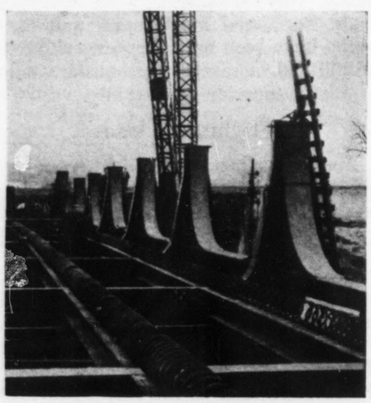
There were no unusual construction problems. Formwork for the deck was carried on pile bents, located at the truss panel joints. The reinforcement of the truss diagonals, anchored and bonded into the bottom chord, was supported in correct position on light wooden frames longitudinally and transversely braced while the concreting was being done. Steel for the verticals and end posts was stubbed into the concrete. Mixing was performed with a 2-sack machine on the south approach span. The entire deck—roadway and bottom chords of trusses—was placed in one continuous pour of 245 cu.yd. Concrete for the trusses was hoisted to top-chord level at the south end of the bridge and was wheeled to position on simple plank runways supported by the top-chord forms themselves, thus eliminating all trestling for runways. End posts and verticals constituted the second pour, top chords the third, and diagonals the fourth. An electrical internal vibrator was of material assistance in placement. In general, very excellent concrete was obtained.

Drains were provided in each of the bottom-chord sections formed between chord flanges and panel point beams. These areas were planked from flange to flange at the level of the top of the panel point beams and thus formed the floor of the sidewalks. Light pipe handrails above the outer flange protect the pedestrian on that exposed side. With the top chords of the trusses directly overhead, no umbrellas are needed, except when winds accompany the rain.

The major features and layout of this bridge were suggested by Homer M. Hadley, regional structural engineer of the Portland Cement Association. The W. H. Witt Co., Seattle, prepared the detailed plans. The co-author of this article was Pierce County engineer at the time that the designs were prepared and the contract was awarded. Most of the construction was performed under the charge of the present county engineer, Forest Easterday. F. E. Walters was resident engineer. Dolph Jones of Tacoma was the contractor.

Vierendeel Welded Trusses Used for Dutch Road Bridge

DUTCH ENGINEERS have recently completed a welded steel Vierendeel truss bridge which adds an interesting variation to the Belgian structures described in *Engineering News-Record,* July 25, 1935, p. 117. The new bridge, described in *L'Ossature Métallique,* September, 1934, carries a highway over a railway at Nuth, Holland, on a through truss span about 175 ft. long with a $25\frac{1}{2}$-ft. roadway and two outside sidewalks, each 8 ft. wide. The upper-chord outline and the floor construction are indicated as features of particular interest.



DECK STRUCTURE of Vierendeel truss road bridge at Nuth, Holland, showing spiral reinforcing for concrete floor and gusset pedestals for web verticals.

While in general practice it is usual to shape arched chords of this type of truss as true arcs, the designers of the Nuth bridge adopted a nine-sided polygon described in a parabola. This polygonal form, it is stated, is preferred by Holland government engineers for technical reasons, and also because they consider it more truly esthetic. Their argument against the true arc is that: (1) longitudinal stresses produce an additional relatively high bending moment requiring more steel; and (2) curved and generally parabolic forms require special shopwork, which is often laborious and expensive. A notable detail is the four-way welding of the lower-chord connections of the verticals to both chord and to floor and sidewalk beams and the plate cap, thus forming a pedestal on which the vertical rests and is welded. The illustration indicates this arrangement.

The floor is a concrete slab without expansion joints and 187 ft. from end to end, reinforced with spirals of $\frac{7}{8}$-in. steel welded to the floor stringers. The sidewalk slabs are similarly constructed. The floor-slab spirals are $9\frac{1}{2}$ in. in diameter and have a pitch varying in accordance with shearing stresses from about 4 to about 5 in. The floor system has no lateral bracing other than the deck slabs. The steelwork was mostly shop-assembled and welded in sections, to reduce field welding.