Old Suspension Bridge Used in Erecting New Arch

Main 350-ft. Ribs of New Highway Bridge at Oregon City, Ore., Swung Into Place by Means of Cables of Bridge Being Replaced—Other Erection Methods Studied

By C. B. McCullough
Bridge Engineer, Oregon State Highway Dept., Salem, Ore.

In the erection of the 350-ft. steel arch bridge across the Willamette River at Oregon City, Ore., several different schemes for placing the large arch ribs were studied and the one selected seems to be unique in bridge erection, namely utilizing the old suspension bridge at the site to carry the sections of the ribs until the arch was closed. The adopted method and the alternate schemes are the subject of this article, together with some notes on the foundations.

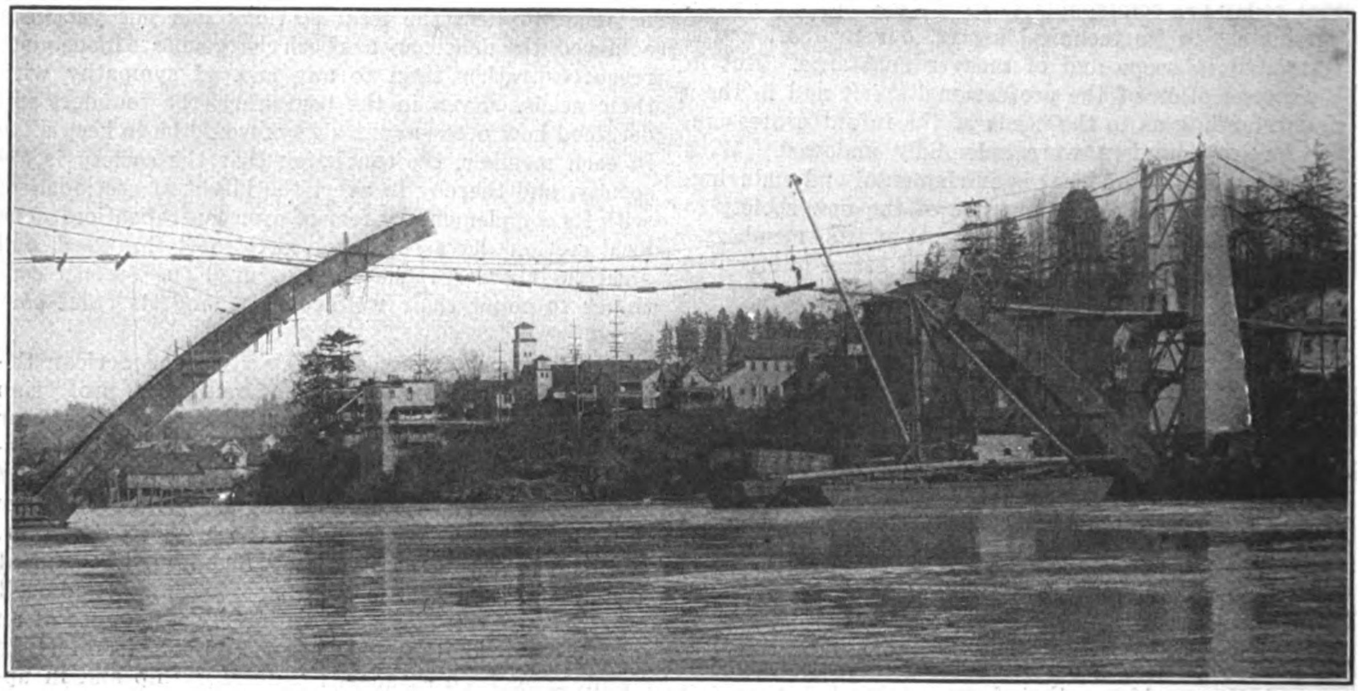
The Oregon City bridge, as described in *Engineering News-Record*, June 8, 1922, p. 942, has as a main span a steel arch 360 ft. between pier centers and 350 ft. between skewbacks, with a 5 per cent grade roadway halfway up the arch so that it is suspended at the center and supported by posts at the ends of the arch. The ribs are 22 ft. center to center and are each of box-girder section, which at the springing line is filled with concrete and which is covered with concrete applied by cement gun. Erected as a three-hinged arch, the hinges are replaced after erection by through splices.

The main channel of the Willamette River at the site of this crossing is nearly 100 ft. deep which fact precluded the consideration of false work in the channel, thus rendering it necessary to adopt some other method of erection. The several different schemes considered in this connection are shown in one of the drawings and may be described as follows:

Alternate Methods—Scheme A—Falsework on floating barges and a battery of hydraulic jacks for elevation adjustment. Since a program of this kind could only be employed during the summer season (owing to the severity of fall flood water conditions), this scheme could not fit in with the program of operations demanded by the time limit on the job. This scheme was never seriously considered by any of the contractors bidding upon the work.

Scheme B—This schedule contemplated the erection of section 1 of the rib, the construction of the abutments or piers and the subsequent erection of sections 2, 3, 4 and 5 from a derrick placed on top of the piers. The floor stringers in this case were to be employed as a temporary anchorage and the balance of the work (sections 6, 7, 8 and 9) placed with the derrick moved out to the end of the temporary cantilever.

Schemes C and D—Both of these schedules contemplated the erection of one-half of the arch rib in one operation. Scheme C contemplated first the placing of the lower section (section 1), next the erection of the piers and a timber erection tower surmounting the same. One-half of the balance of each rib was to be riveted up next and floated to the site in "position No. 1," raised by jacks to "position No. 2" and connected at the lower erection pins. Each rib was then to be raised to a position sufficiently above the true position to allow the other half span to clear when raised; and subsequently, the two sections were to be lowered simultaneously and pinned at the center. Scheme D was somewhat similar except for the fact that lifting was to be done from a



GENERAL VIEW OF OREGON CITY ARCH UNDER ERECTION

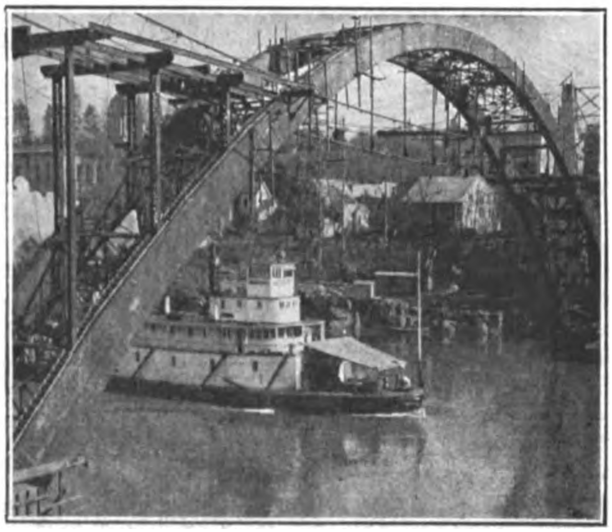
Rib sections are held in place by stays from towers of old suspension bridge being replaced. Old footway used for temporary construction crossing. The main channel of the Willamette River at the site of this crossing is 100 ft. deep.

Digitized by Google

vertical tower on a large barge at the center, thus dispensing with a shore line for lifting. In this scheme, the entire half span including lateral bracing was to be lifted at one operation.

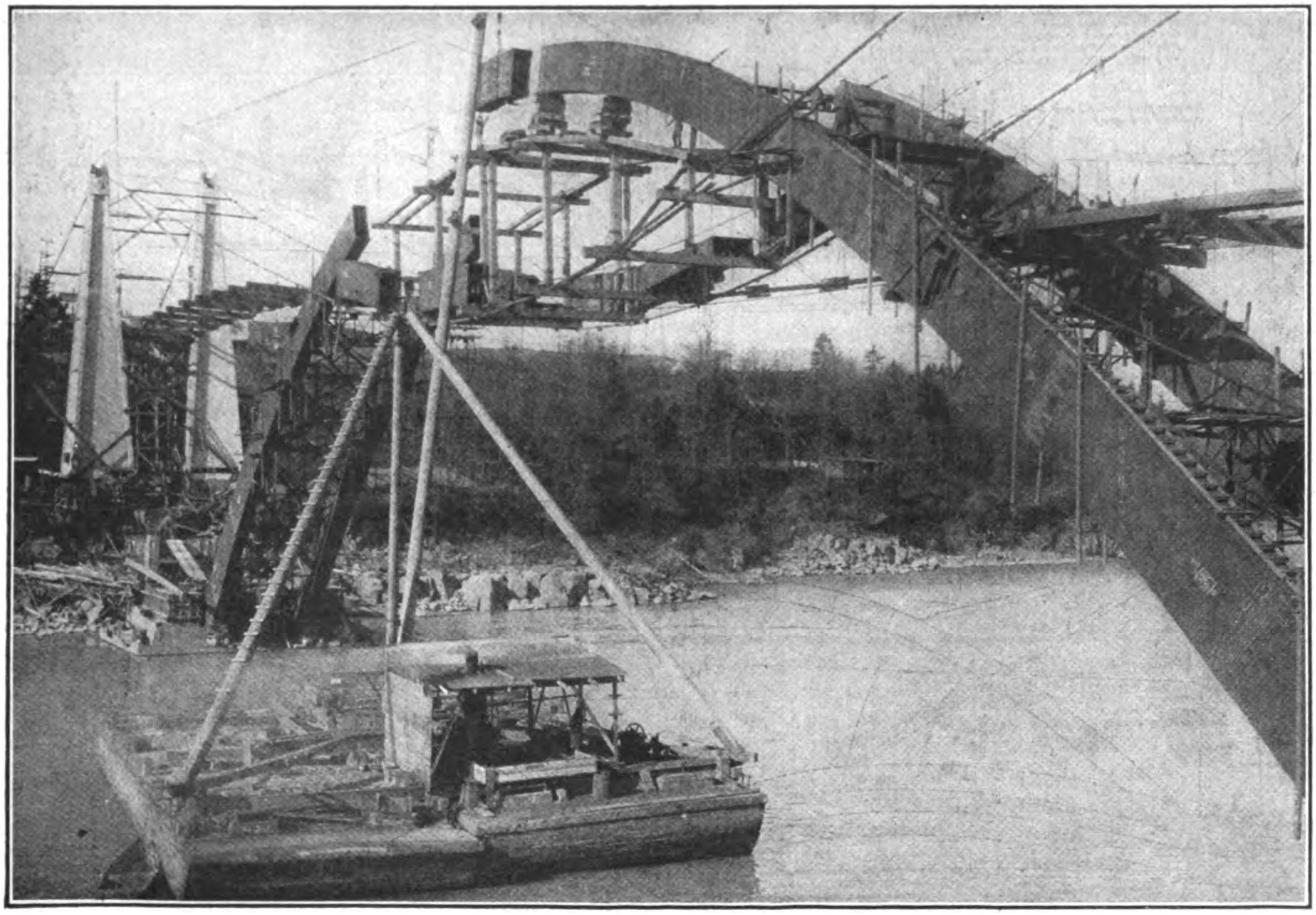
Method Adopted—Utilizing the cables of the old suspension span. The erection schedules described hereinabove were worked out in the office of the State Highway Department, and submitted to the erectors upon request before bids were taken. The successful bidder, however, proposed a fifth scheme which was finally approved and adopted for the work. This erection schedule was worked out by the firm of Gerrick and Gerrick, of Seattle, Wash., who were sub-contractors for the erection and is largely the work of A. Münster, of Seattle, who was retained by the above firm as consulting engineer. This scheme utilized the main cables and towers of the old suspension span (which was to be replaced by the present structure) for the support of rib sections.

Rib sections No. 1 at either end were first placed on the skewback support by means of a barge derrick. These sections were rested on a temporary erection hinge and held to position by means of a radial cable passing upward over the old suspension bridge towers and back to an anchorage on the approach. Sections 2, 3, 4, 5 and 6 were successively placed, each rib section being held by radial cables passing over the towers and down to the anchorage. One of the photographs shows six rib sections on the left held in this manner and the fifth section on the right being lifted from the scow for placement.



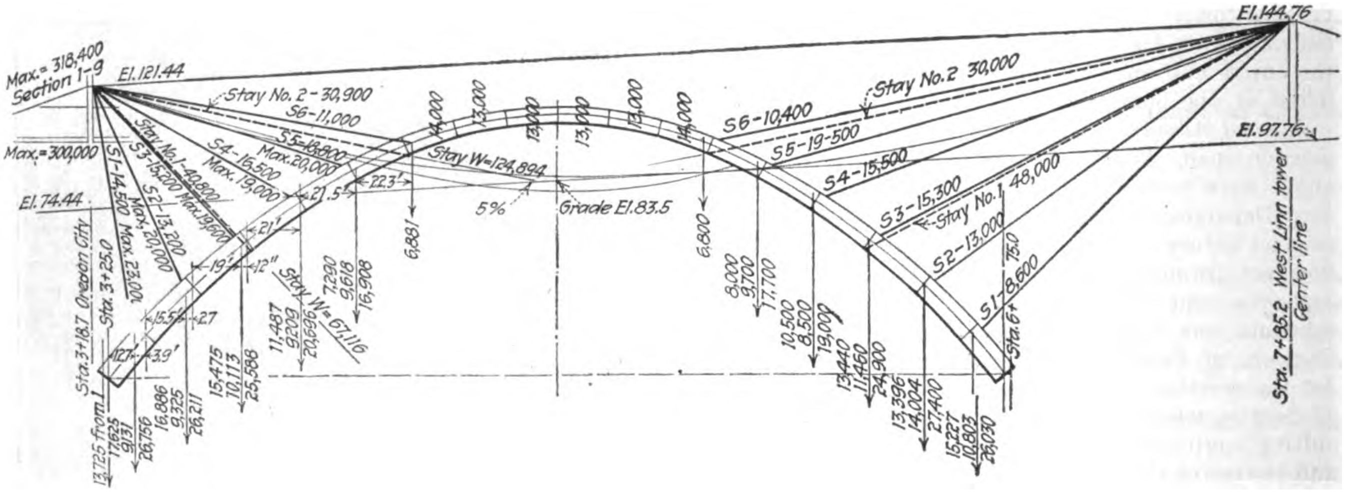
OREGON CITY ARCH RIB SWUNG INTO PLACE

There is also shown herewith a graphic analysis of cable stresses which is self-explanatory. Although each radial cable was amply capable of sustaining the maximum stress induced therein by virtue of the rib loads, it was considered the part of prudence to require the use of auxiliary *stay* cables to insure against overstress in any one of the radial cables due to a breakage or any undue deflection in any of the others. The graphic analysis indicates the scheme first outlined and contemplates the use of *two* stay cables for each cantilever; one at the



PUTTING CROWN SECTIONS OF RIB IN PLACE ON OREGON CITY ARCH

Old cables being used to carry rib sections prior to swinging in place. Haunch sections stayed back to suspension bridge tower. Crown sections built up on falsework on cables.

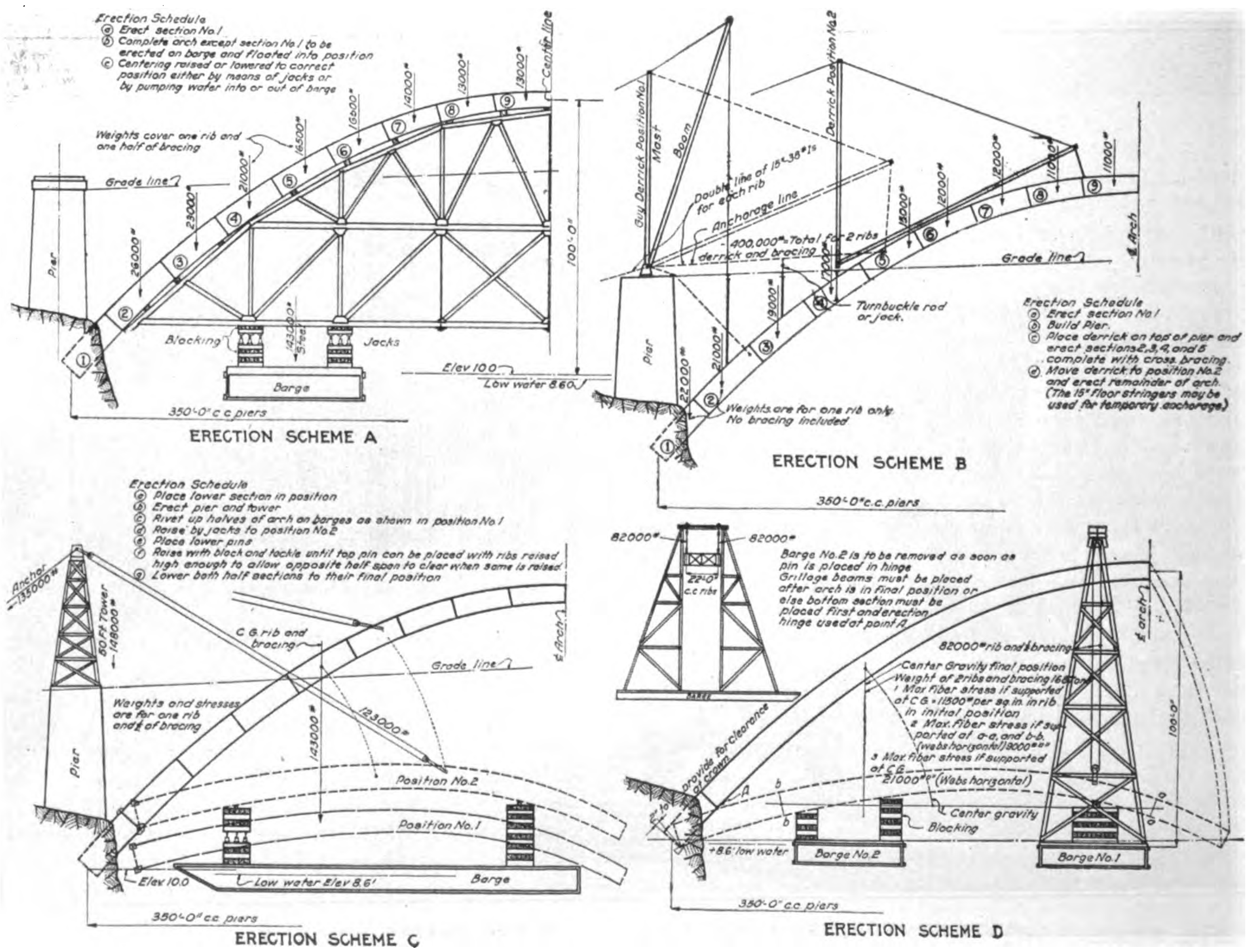


GRAPHIC REPRESENTATION OF THE CABLE STRESSES IN ERECTING THE ARCH

extreme end of section 3 and one at the extreme end of section 6. These stay cables were designed to carry the entire six rib sections independently of cables, S1 to S6 inclusive. The two stays shown were afterwards discarded in favor of one heavy stay cable at the end of section 4.

The central portions of the rib span (sections 7, 8 and 9) were placed on timber bents carried directly on the main cables of the old suspension span. One of the views shows the construction of these bents, the last section being placed at the time of the taking of this photograph. There is also a view of the completely swung rib with the central falsework removed from the main cable and a portion of the column and floor steel in place.

Tests of material cut from the main cable made in the laboratory at the time of erection disclosed an ultimate strength of 157,000 lb. per square inch. This, with an area of 13.85 sq.in., gave a breaking load of 2,174,450 lb.; while the maximum erection load under



ALTERNATE METHODS STUDIED FOR ERECTION OF OREGON CITY ARCH

Digitized by Google

the worst condition amounted to 351,000 lb., thus giving an ample factor of safety. The radial cables were 1-in. plow steel having a breaking load of 38 tons. The stay cables were of 2-in. plow steel having a breaking strength of 140 tons.

In order to eliminate the difficulty of blocking the center sections to correct elevation, which would be caused by the varying deflection in the main cable, the six center sections were first loaded onto a platform between the bents and supported by the main cables causing the cable to take its final (or nearly final) position before the bent blocking was placed.

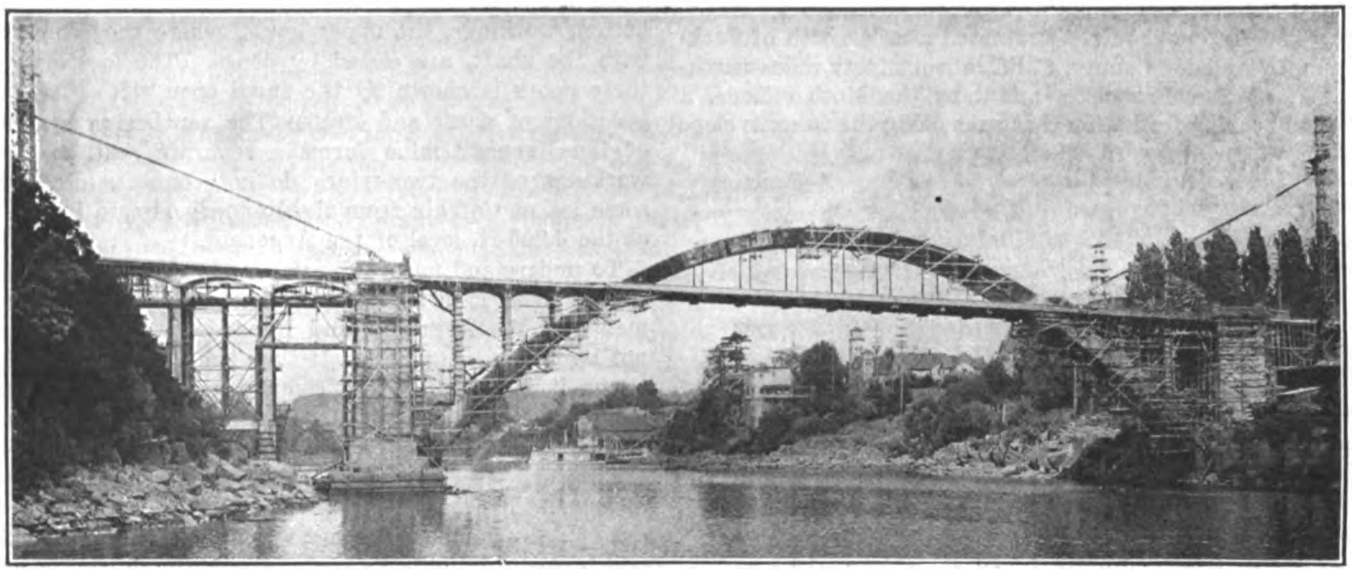
After the rib was swung, a portion of the remaining dead-load was added to the structure acting as a three-hinged arch. Subsequently, the ribs were fixed at crown and at skewback thereafter acting under live-load and the balance of the dead-load as fixed arch ribs.

Foundation Work—Some of the details of the construction of the main piers are worth describing. These foundations were on solid basaltic rock, one abutment being entirely in the dry and the other in about 8 ft. of comparatively still water. No problem out of the ordinary was presented by either foundation, the wet abutment being concreted in the dry through the unwatering of a double wall puddle cofferdam of ordinary type. Core drillings were taken for a distance of from 25 to 35 ft. below the surface to disclose the presence of any seams or pockets in the foundation material. The drillings, however, indicated nothing of this kind, yielding a very uniform and continuous core. Tested in the laboratory, these core specimens gave the following results:—

Maximum compressive strength — 15,300 lb. per sq.in. = 1,101.6 tons per sq.ft.
Minimum compressive strength — 11,050 lb. per sq.in. = 795.6 tons per sq.ft.
Average of maximum and minimum strength — 13,175 lb. per sq.in. = 948.6 tons per sq.ft.

In view of the fact that the resultant base pressure on the abutment, even under the worst possible live-load condition, falls within the middle third of the base and that the maximum extreme toe pressure amounts to but 4.9 tons per square foot, this type of foundation leaves little to be desired. The base rock was lightly shot and about 2 ft. of material removed in order to roughen the surface and remove any shattered or slightly seamed surface material and concrete placed thereon in the dry.

Personnel—Contract for this structure was awarded in June, 1921, to A. Guthrie & Co., Inc., of St. Paul and Portland, and construction work begun during the following July. The schedule of erection herein outlined operated with the utmost satisfaction to all parties concerned and from start to finish the program was carried through without a hitch. Credit for the admirable solution of the many erection problems encountered is due to the excellent engineering staff maintained by the contractors' and sub-contractors' organizations, to A. Münster, their consulting engineer, and to R. A. Furrow and C. P. Richards, who were in charge of the construction work for the Oregon State Highway Department, by whom the bridge is being built.



VIEW OF BRIDGE IN LATER STAGES OF COMPLETION

Canadian Canal Traffic Shows Increasing Tonnage

The July summary of Canadian canal statistics issued by the Dominion Bureau of Statistics, shows increased traffic in nearly every case. Total traffic through the Canadian and American locks at Sault Ste. Marie increased more than 26 per cent, over July, 1921, and also over June, 1922. The large increases were in wheat and iron ore; the latter totalling 8,942,659 tons, compared with 4,356,760 tons in July, 1921; 9,235,086 tons in July, 1920, and 8,912,609 tons in July, 1919. Wheat shipments for the month were 17,777,377 bushels as against 7,838,878 in July, 1921; 7,838,470 in July, 1920, and 7,100,008 in July, 1919. Coal shipments continued light.

Welland canal traffic showed a slight decrease from July, 1921, mainly in soft coal shipments and in American corn. Wheat shipments increased 191,291 tons or 6,376,000 bushels, while barley, iron and steel and pulpwood also showed increases.

On the St. Lawrence canal passenger traffic increased more than 44 per cent over that of July, 1921. Freight traffic also increase 23,256 tons or 4 per cent despite large decreases in coal and in corn and oats. Wheat, iron and steel, pulpwood, lumber and timber all showed substantial increases.

Digitized by Google

Gunite Retains Integrity on Oregon Road Bridges

Incasement placed in 1922 and 1929 shows no deterioration with the exception of a few hair cracks as disclosed by careful survey made in 1932

By C. B. McCullough
Assistant State Highway Engineer, Salem, Ore.

THE TWO BRIDGES described in this article exhibit both a recent and a fairly old example of steel incasement with gunite. Recently a careful inspection was made to determine the condition of this incasement, and the results appear of interest, particularly in the case of the structure at Oregon City. This bridge was described in *Engineering News-Record*, Nov. 2, 1922. A view of the bridge is shown in Fig. 1 and of the method of applying the incasement in Fig. 2.

Oregon City Bridge

The Oregon City Bridge carries the traffic of the Pacific Highway across the Willamette River at Oregon City and consists of one central fixed-arch span 360 ft. long flanked by a reinforced-concrete viaduct approach at either end, the entire structure being 745 ft. long. The arch ribs are closed box-girder sections braced at frequent intervals throughout their length by radial diaphragms. Suitable openings are provided in the rib sections at the crown for inspection, and the solid steel diaphragms are fitted with manholes to permit easy access from end to end of the rib sections on the inside. The structure replaced a suspension bridge with timber towers and stiffening trusses, and the erection program contemplated the use of the old cables, as indicated in Fig. 4.

The exterior of the box-girder arch ribs was protected by an incasement

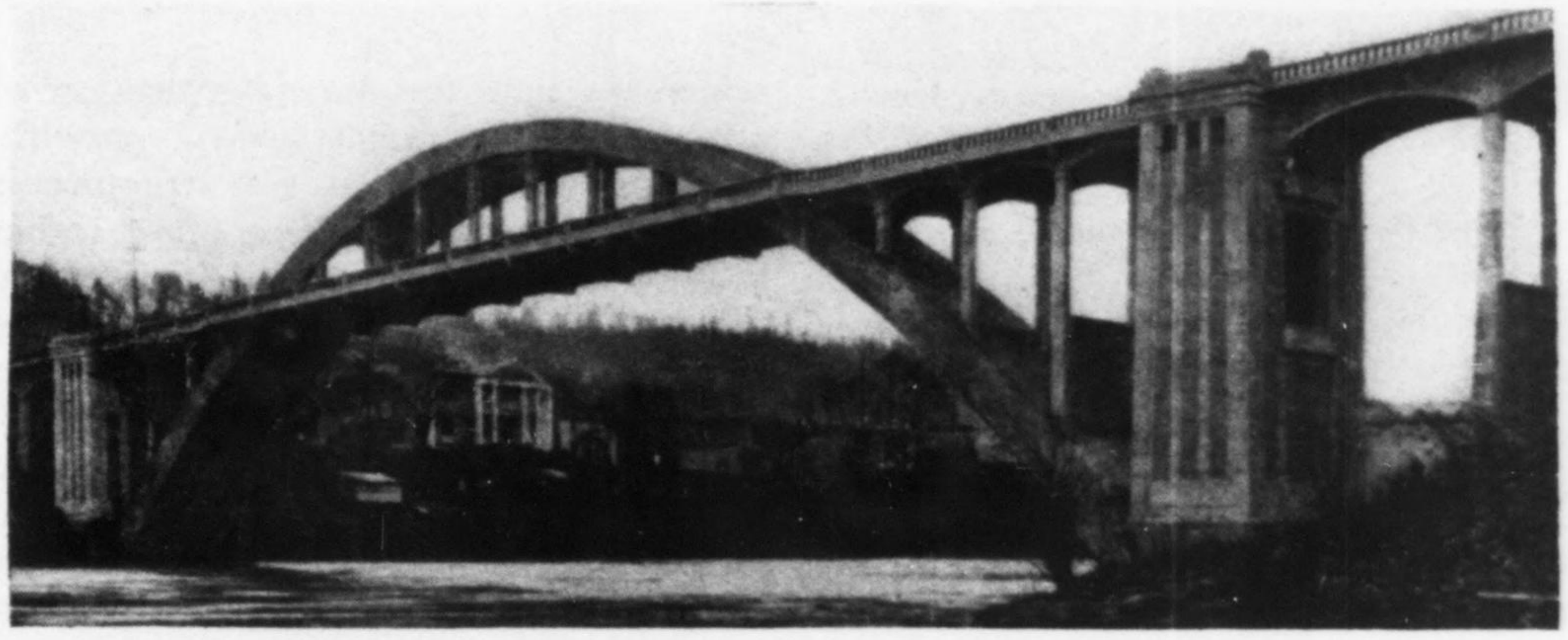


Fig. 1—Steel box-section arch-rib bridge completed in 1922 at Oregon City, Ore., with incasement of all steelwork.

1½ in. thick, the method of application being substantially as follows: The entire exterior was covered with a network of 3.8 in. round rods spaced 12 in. center to center both ways, and spot-welded to each other and to the structural steelwork at each intersection. This system served as an anchorage for a layer of triangular wire mesh subsequently placed and securely wired thereto. Below the deck the ribs were braced at each panel point by means of steel trusses the full depth of the rib. The individual truss members were incased, and the entire system was then converted into a solid girder by means of a gunite diaphragm shot against a temporary timber backwall.

Shortly after the construction of the bridge a series of drawings, as indicated in Fig. 3, was prepared for the purpose of carefully locating any cracks that might be observed to develop as the structure progressed in service. It will be noted on the sketch that the entire arch-rib surface is coordinately ruled so that the exact location of any particular crack or blemish may be quickly obtained. Fig. 3 indicates the formation of hair cracks as disclosed by two inspections: May 12, 1924, and Sept 21, 1932. This figure represents only one side of one-half of one vertical-rib surface; however, this particular part of the rib is typical of the entire structure.

An inspection of Fig. 3 will disclose the fact that there are no cracks, blemishes or defects of any nature on that portion of the work below the deck. This is true for the entire structure. Above the deck a few cracks have developed, but in nearly every case the cracking observed in 1932 was disclosed by the 1924 inspection. In other words, very few new cracks have developed during the eight-year period between these two inspections.

In order to ascertain the degree to which the cracking observed above the roadway deck might operate to impair the service life of the structure, the gunite was chiseled away in several places, and it was found that the cracks could not be traced beyond a depth of from ⅛ to ¼ in. below the surface, indicating that the trouble was undoubtedly a surface condition. The incasement was placed in two layers, the last one being a very thin flash coat employed for the purpose of smoothing up the surface, and it seems quite apparent that very few, if any, of the hair cracks observed extend beneath this finish or flash coat.

During the course of construction, tests were made on several gunite slabs, approximately 1½ in. thick, shot onto steel plates 8x½ in.x4 ft. These specimens were placed, with the steel side down, over knife-edge supports 46 in. apart and downwardly acting loads applied midway between supports. The table on page 260 indicates the observed modulus of elasticity and extreme fiber stresses at failure for five of the slabs thus tested. The beams were designed so that the gunite would fail at a comparatively low working stress in the steel, and it is interesting to observe

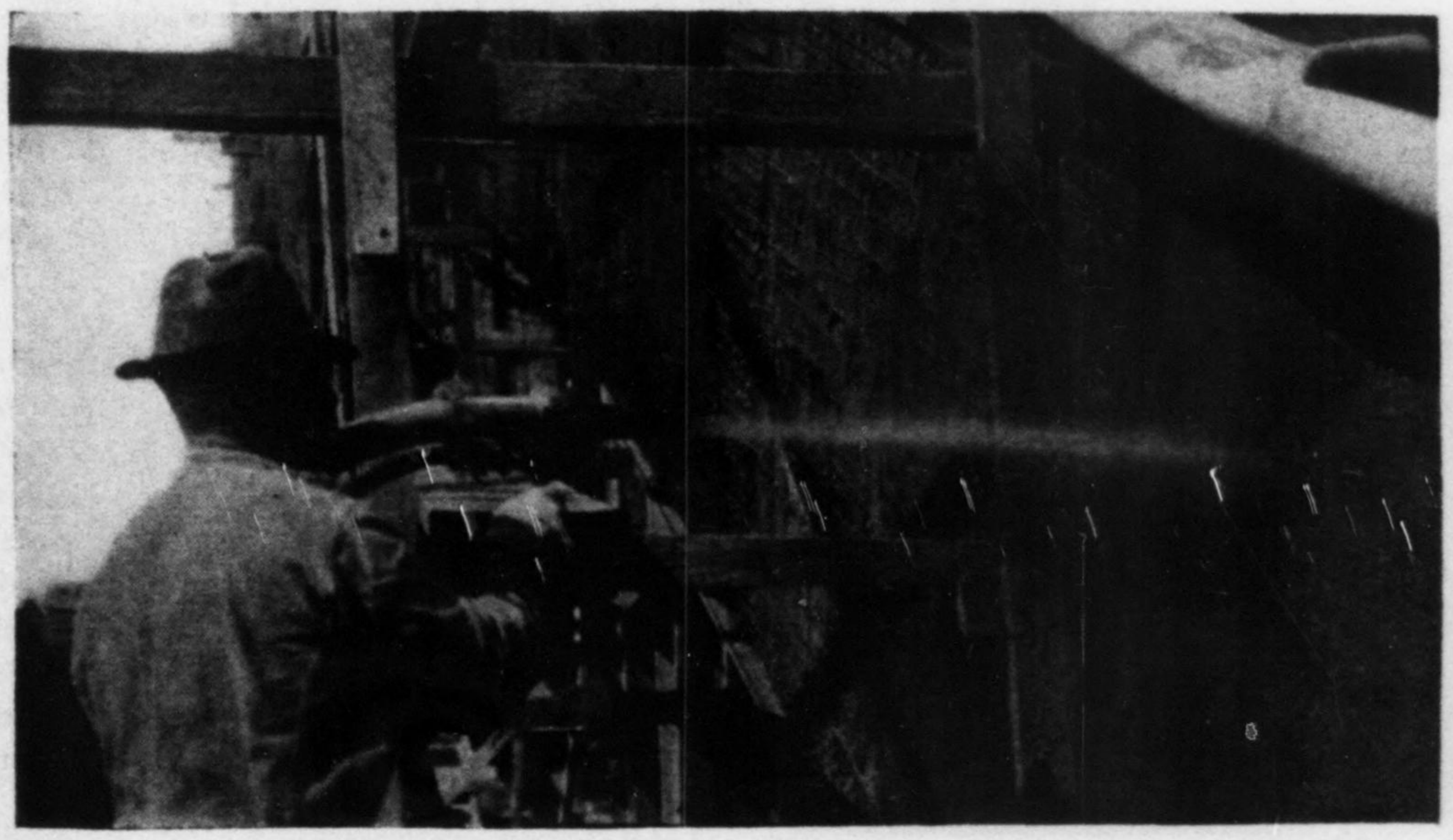


Fig. 2—Guniting arch rib of Oregon City Bridge; ⅜-in. bars 12 in. apart both ways, spot-welded at intersections to arch steel, anchored the reinforcing mesh.

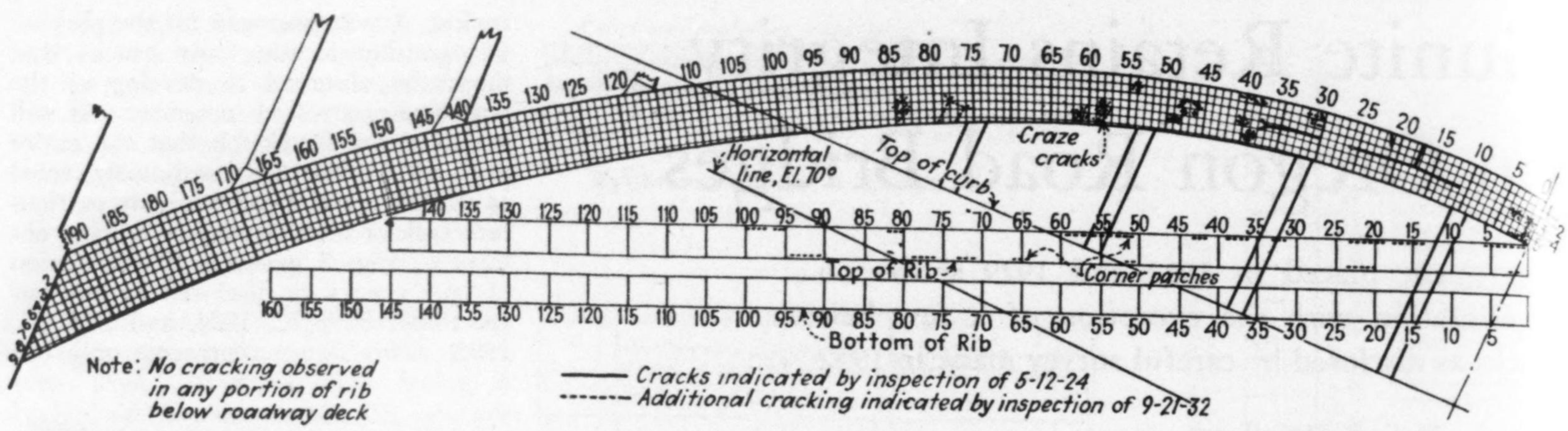


Fig. 3—Typical sketch employed in recording cracks in incasement. Sketch shows crack record of 1924 and 1932 surveys.

the relatively high elastic modulus (4,670,000 lb. per sq.in.) and the high average ultimate strength (8,000 lb per sq.in.) of the material.

The incasement as applied on the Oregon City Bridge resulted in an exceptionally dense and hard protective coating and, with the exception of the few hair cracks noted, a service record of ten years indicates no deterioration of any character whatsoever, notwithstanding the fact that the structure is exposed to an atmosphere often rather heavily laden with sulphite fumes from two large paper plants in the immediate vicinity.

The employment of a steel rib of hollow box-girder section incased in gunite not only operated to save a considerable sum as against the cost of a solid concrete rib section but also greatly simplified certain erection problems that had arisen.

MODULUS OF ELASTICITY AND ULTIMATE STRENGTH OF TEST SPECIMENS OF GUNITE USED ON OREGON CITY BRIDGE

Specimen No.	Elastic Modulus (Lb. Per Sq.In.)	Ultimate Strength at Failure (Lb. Per Sq.In.)
1	4,800,000	6,360
2	3,800,000	7,980
3	5,350,000	9,250
4	5,100,000	9,780
5	4,300,000	6,620
Average	4,670,000	8,000

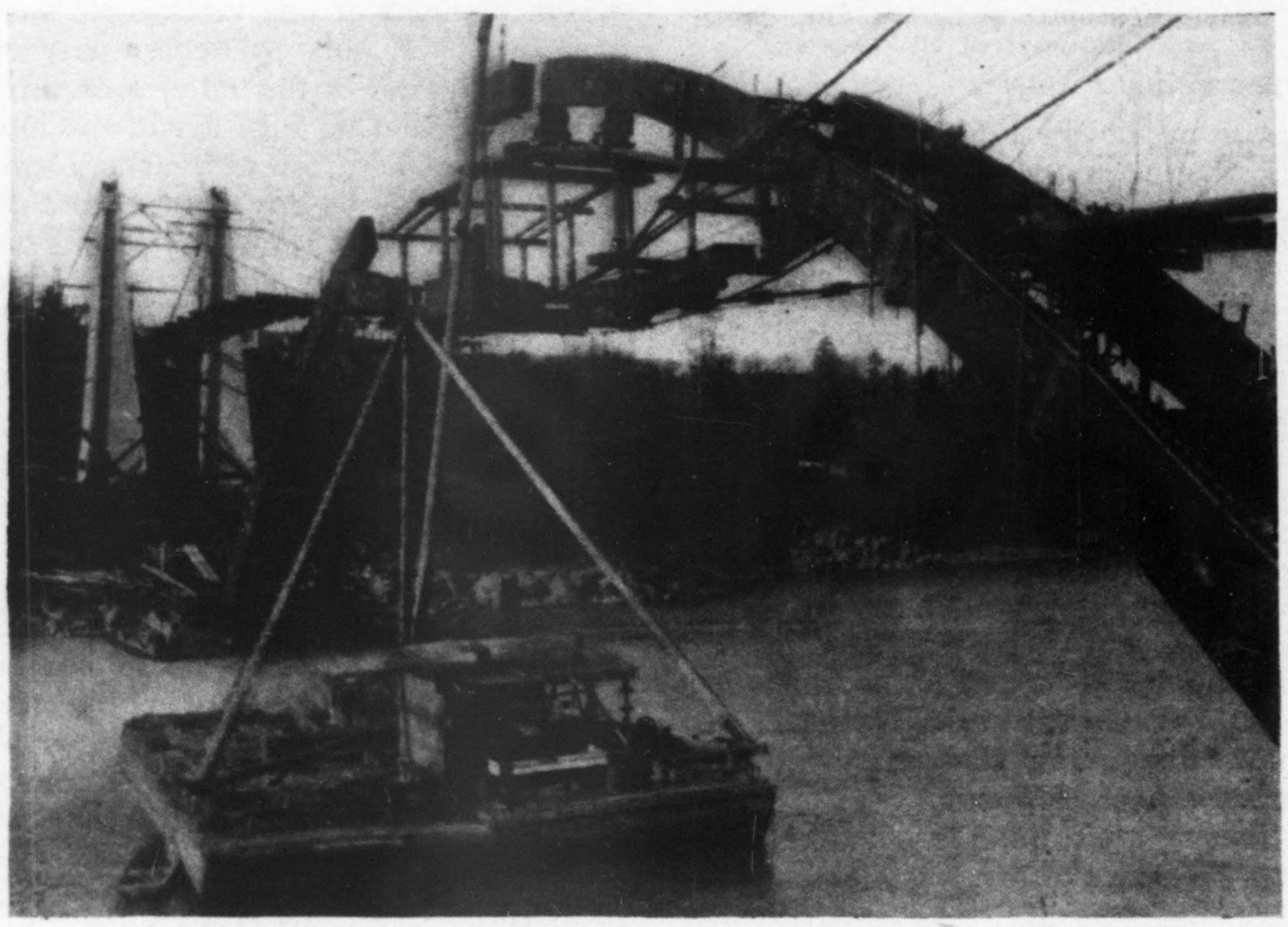


Fig. 4 (above)—Old suspension bridge, replaced by new arch, was utilized in erecting the new steelwork for the Oregon City Bridge in 1922.

Fig. 5 (below)—Under-side view of McKenzie River Bridge, showing incasement of steelwork with no cracks after three years.

McKenzie River Bridge

The bridge across the McKenzie River was built in 1929. It consisted essentially of a curved-chord, steel pony truss of 100-ft. span supported on cantilever arms forming a main central span of 130 ft. The cantilever arms are of reinforced concrete, with suitable structural steel anchorages to carry the negative bending back into the adjacent flanking approach spans. The suspended span is, as above stated, an ordinary curved-chord, steel pony truss, the top chord and diagonals of which were incased in concrete, while the bottom chord and lower lateral system were gunite-incased. Fig. 5 indicates the general appearance of the incasement around the lower chords, lateral system and sidewalk brackets. A very careful inspection of this structure failed to disclose any evidence of hair cracking, checking or any surface disintegration after three years of service.

Santiam River Water Power

A study of the potential water power of the Santiam River basin in Oregon has been completed by Benjamin E. Jones, hydraulic engineer, and Arthur M. Piper, geologist, of the U. S. Geological Survey. Without storage the river can produce 143,000 hp. for 90 per cent of the time and 447,000 for 50 per cent of the time. With storage reservoirs outlined in the report to the Survey, these figures will be increased to 351,000 and 492,000. Fourteen power sites are listed and seven reservoir sites. Reconnaissance of the geology at many of the dam sites was made by the geophysical method. Copies of the reports are available in the district office at Portland, Ore., or in the Washington office.

Large Steel Arch Bridge Ribs Encased in Gunite

Oregon Highway Bridge Has Central Span of 350 Ft. Made Up of Structural Steel With Thin Concrete Covering—Comparison of Alternate Designs Considered

BY C. B. MCCULLOUGH
Bridge Engineer, Oregon State Highway Commission, Salem, Oregon

A STEEL arch, 360 ft. between pier centers, 350 ft. between skewbacks, and encased in concrete applied with a cement gun is now being built by the Oregon State Highway Commission, as the main span of a concrete and steel bridge across the Willamette River at Oregon City, Ore. This unique type of bridge structure was decided upon only after an extensive comparative investigation into different types for the crossing. The considerations which governed the objection of the other types and the selection of this one, and the details of the bridge now being built are the subject of this article. The methods of erection, which also required some comparative study, will be discussed in a later article.

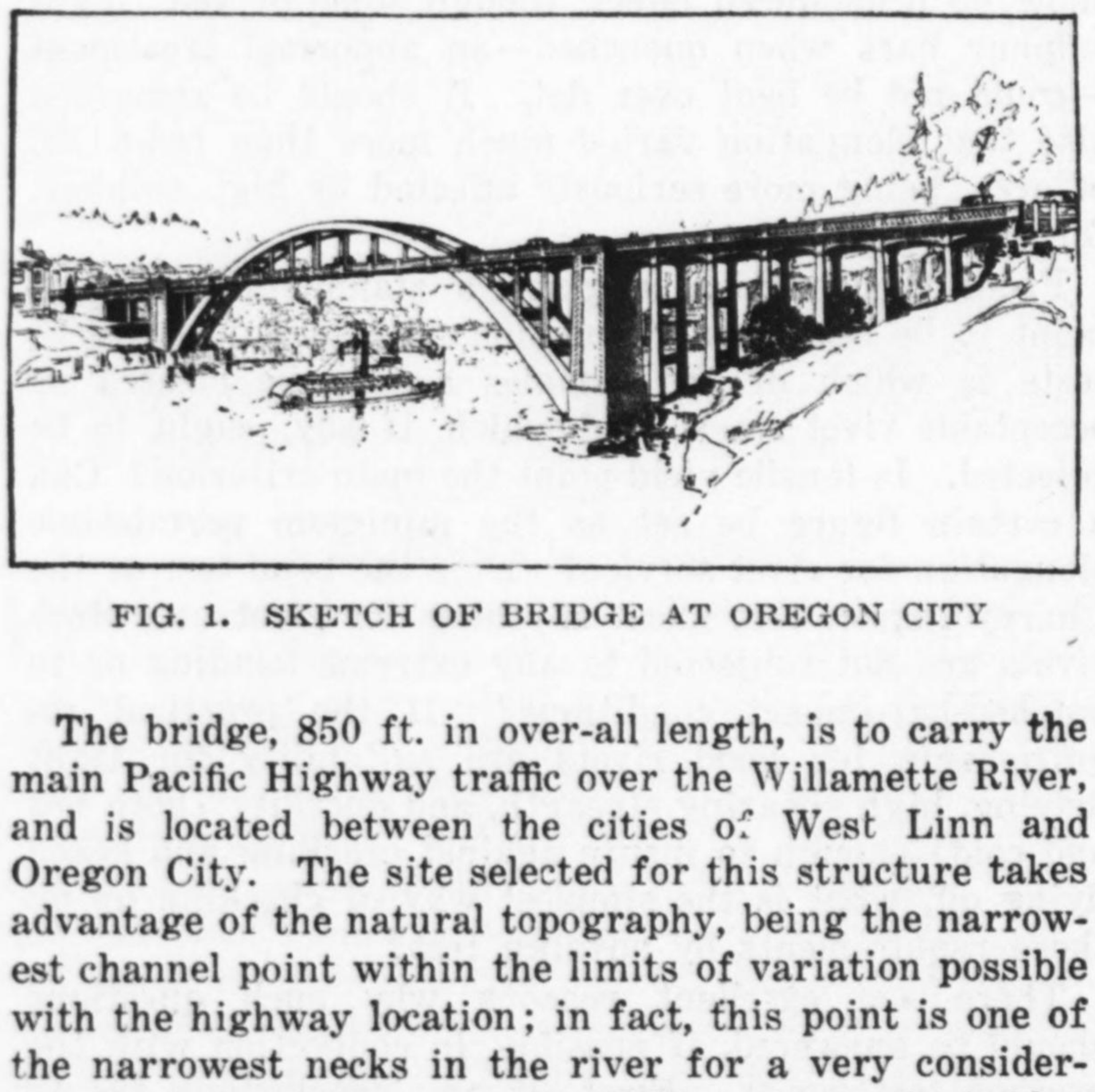


FIG. 1. SKETCH OF BRIDGE AT OREGON CITY

The bridge, 850 ft. in over-all length, is to carry the main Pacific Highway traffic over the Willamette River, and is located between the cities of West Linn and Oregon City. The site selected for this structure takes advantage of the natural topography, being the narrowest channel point within the limits of variation possible with the highway location; in fact, this point is one of the narrowest necks in the river for a very considerable distance either way. The Willamette at the bridge site flows in a well defined and very deep channel bounded by high cliffs of basaltic formation, the maximum stream depth at mean low water being over 100 ft.

The bridge crossing is, unfortunately, located in the immediate vicinity of certain large industrial plants employing sulphurous acid as a reagent, with the result that, at times, the atmosphere is more or less densely charged with sulphurous anhydride (SO_2). Corrosive action in this atmosphere appears to be considerably stimulated, and it was thought that an unprotected steel structure might impose an undue burden of maintenance upon the state. The city of Oregon City is located but 15 miles from Portland and is one of the biggest industrial and manufacturing towns in the state. The bridge, which will be a link in a highway surfaced on either side with heavy traffic pavement, will, therefore, be subjected to exceptionally heavy truck traffic as well as a large volume of pleasure traffic and pedestrian travel to and from the large paper mills that lie on the West Linn side. The waterway is navigable and fairly busy as regards river traffic, requiring a horizontal and vertical clearance not less than that shown in Fig. 1.

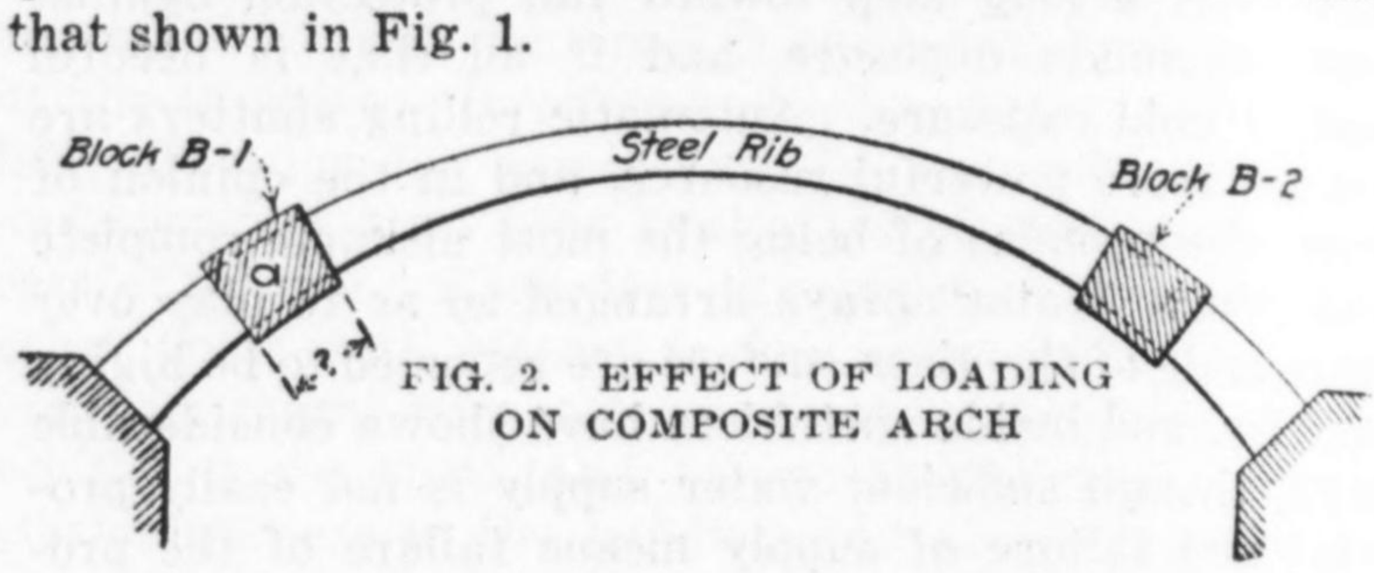


FIG. 2. EFFECT OF LOADING ON COMPOSITE ARCH

In order to meet the conditions above outlined and, at the same time, take advantage of the narrow and rock bound character of the stream crossing, preliminary studies were made for seven distinct types of construction and covering a period of nearly two years, with the following results:

Type A. Suspension Span—Abandoned because of the cost of making such a structure rigid enough to carry the heavy truck traffic without injury to the roadway pavement, and also because no feasible method of protecting the metal against corrosive influences presented itself.

Type B. Cantilever Span—Type C. Simple Truss Span and Type D. Spandrel Braced Framed Arch, were each tried successively, and were abandoned owing to the difficulty encountered in securing a suitable type of encasement or protective covering at a reasonable cost.

Type E. Reinforced-Concrete Arch—This type was abandoned owing to the great cost of supporting the centering for the extremely heavy and massive concrete rib. The great depth of the water coupled with the fact that the channel must be kept open at all times for navigation rendered the use of falsework out of the question. In this connection investigation was made of the use of suspended centering which was also discarded in favor of the more practical and positive method outlined under Type G below.

Type F. Steel Arch Rib With Poured Concrete Encasement—This type was considered a long time and complete details were at one time worked out for the same. Some very interesting problems were encountered a very brief mention of which may be in order.

The first scheme for this type employed an encasement designed to take a portion of the rib stress, in other words, the steel rib was simply utilized for supporting the formwork for a nearly solid concrete rib. The rib sections were to be poured block by block loading each rib simultaneously and symmetrically. It is readily seen that, as this load is placed, stresses of ever increasing magnitude are set up in the rib steel. When the rib itself is swung the stress at any point *a* has a certain value S_1 which may be easily determined. (See Fig. 2). Having the rib swung, the first voussoir or rib block B-1 may now be poured. As soon as concrete pouring commences the rib starts to deflect under the additional dead-load so that at the end of this pour the stress in the steel is at some new increased value S_2. The concrete in the meantime is in a semi-plastic condition so that, if precaution is taken to thoroughly spade and "rod" the mass we may reasonable assume that at the end of the pour the stress in steel $= S_2$ and the stress in concrete $= 0$.

The addition of a second block B-2 anywhere along the rib now operates to increase the stress in the steel at

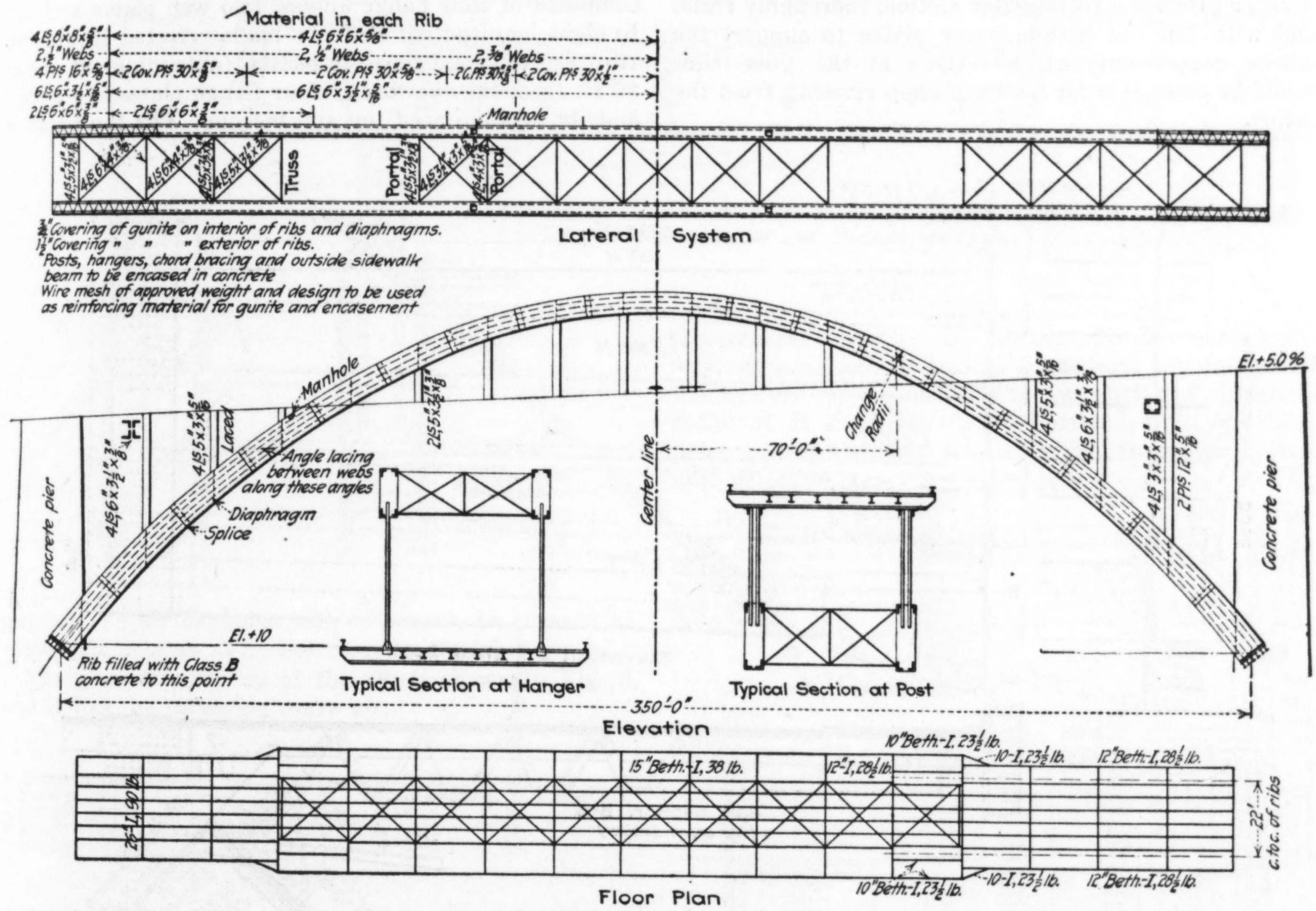


FIG. 3. STRUCTURAL STEEL MAKE-UP OF MAIN ARCH

point a to a new value S_3, the rib length L_1 is therefore shortened and a stress S_c, where

$$S_c = \frac{(S_3 - S_2)\,(E_c)}{E_s},$$

is at once set up in the concrete. Stresses of this kind, imposed upon *green* concrete undoubtedly shatter and break the initial set for which reason it would seem essential that each block be poured and allowed to set before any additional load was placed upon the rib. This scheme required so much time that it was abandoned in favor of a very thin, poured encasement, designed merely as a *protective coating* and not in any way as augmenting the stress carrying capacity of the steel rib.

This thin encasement presented a very difficult construction problem as regards pouring. It was felt that an encasement such as this would undoubtedly give the public a sense of security and that this must not be *false* security, in other words there must be no porous pockets with the consequent danger of the steel rusting underneath the concrete. An encasement, designed to insure the degree of density and positive protection considered essential, proved very costly and also imposed a very severe dead-load on the rib.

Before finally abandoning this type altogether, experiments were made looking toward the adoption of a *light-weight* aggregate. At one time this seemed to be the answer, however Type G described below had so much to recommend it, both in cost and serviceability that it was chosen as the final solution.

Type G—Steel Rib Arch Gunite Encasement—This design, which was the one finally adopted, contemplates the construction of a main channel span 360 ft. center to center of piers, flanked on either end by approaches of the reinforced column and girder type (see Fig. 1). At the Oregon City end is a retaining wall and fill, the wall being of the cellular type of comparatively thin concrete sections, heavily reinforced. The span carries a roadway 19 ft. clear width and two 4 ft. 6-in. sidewalks. It is designed as a half-through span to provide the required vertical and lateral clearances for navigation. The following loadings were used in the design:

Concentrated loading consisting of two trucks abreast of the following dimensions and weights:

1. Total weight .... 40,000 lb.
2. Weight concentrated on rear axle .... 28,000 lb.
3. Distance, c. to c. axles .... 10 ft.
4. Distance, c. to c. wheels .... 6 ft.
5. Length of roadway occupied .... 24 ft.

That portion of the roadway not occupied by the truck loading as above, was assumed as loaded with a uniform load of 70 lb. per square foot. The sidewalks were loaded with a uniform load of 60 lb. per square foot.

The stress analysis was made after the method of variable "rotation loads" or "elastic weights" described by the writer in *Engineering News-Record*, April 1, 1920, p. 675. Arch reaction and stress influence diagrams were plotted and the above loading placed in such manner as to produce the maximum rib stresses.

The main arch rib is of structural steel designed with particular attention to the requirements enumerated below:

1. To produce a rectangular section, thoroughly rigid, and with top and bottom cover plates to support the gunite encasement, which section at the same time would be accessible for field and shop riveting from the inside.

the temporary construction hinge provided in the design. A similar arrangement will be used at the crown.

As soon as the ribs are swung and the lateral system in place, 30 ft. of the rib at each skewback will be completely encased and filled with Class B concrete, thus rigidly fixing the ends. The crown hinge will next be replaced by a heavy splice connection designed to transmit the maximum thrust, moment and shear at the crown. The rib then becomes a fixed or hingeless arch under all additional dead and live loadings.

The main arch rib consists of a box girder section composed of four flange angles, two web plates and six to eight longitudinal stiffener angles riveted inside the rib. For the solid concrete section (extending out about 30 ft. from the skewback) four flange plates are added and the section laced top and bottom. For the balance

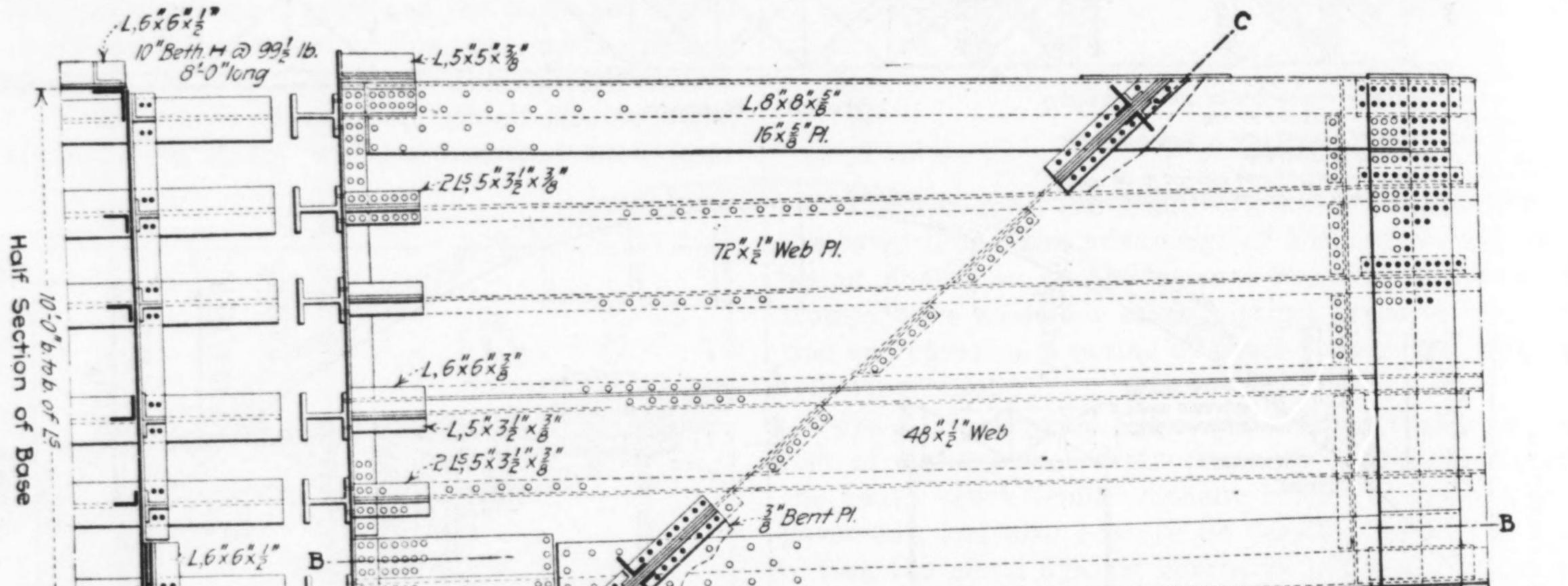


FIG. 4. DETAILS OF ARCH AT SKEW-BACK

2. To so detail the section that all diaphragm angles and stiffeners would be inside leaving a plane rectangular surface for the application of the wire mesh and gunite encasement.

3. To allow for inspection of the interior of the rib both during and after construction.

The steel ribs are to be erected first, and swung as a three-hinged arch. Fig. 4 illustrates the nature of

of the rib top and bottom cover plates are used in order to carry the encasement of gunite. Diaphragm plates are used to stiffen the rib as shown in Fig. 4, these being apertured with manholes to aid in construction and also to permit maintenance inspection from time to time. Manholes will also be left in the top cover plate at convenient points to permit access to the interior of the rib for future inspection of the condition of the

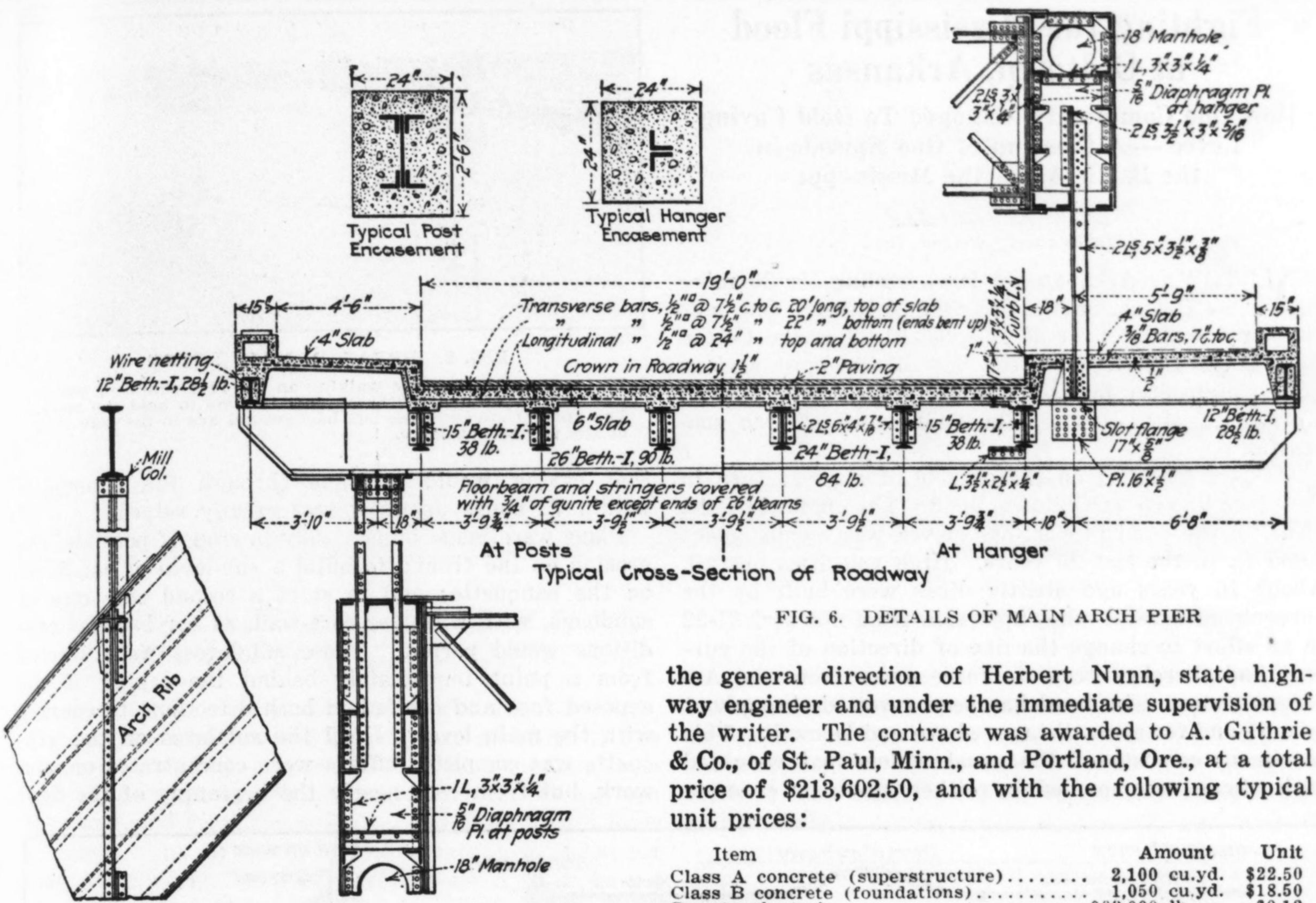


FIG. 6. DETAILS OF MAIN ARCH PIER

gunite. These manholes will, of course, be covered by removable hatches of non-corrosive material.

The general make up of the rib is shown in Fig. 3, and it has the following make up:

RIB

4 flange angles	8 x 8 x 5/8 in.
2 web plates	½ x 120 in. (approx.)
4 flange plates	16 x 5/8 in.
6 longitudinal stiffener angles	6 x 3½ x 5/16 in.
2 longitudinal stiffeners	6 x 6 x 3/8 in.

CROWN

4 flange angles	6 x 6 x 5/8 in.
2 cover plates	30 x ½ in.
2 web plates	72 x 3/8 in.
6 longitudinal stiffener angles	6 x 3 x 5/16 in.

The inside of the section is to be covered with a ¾-in. gunite encasement, the exterior is to be covered with wire mesh and given an encasement of gunite 1½ in. in depth, finished with a carpet float.

The hangers and columns as well as all portal and lateral frames are to be completely encased in a solid poured concrete encasement. The structural frame used for the portal is to be encased in a curved and panelled portal beam. The entire lateral system is of similar construction. The floor system is of structural members encased in gunite as shown in Fig. 5.

Fig. 6 illustrates the general design of the arch piers. Under each pier provision is made for comfort stations and observation balconys which are entered from the floor elevation by stairways leading from the upper balcony. The fixtures for these rooms will be furnished and installed by the cities of West Linn and Oregon City.

The design and construction of this bridge is under the general direction of Herbert Nunn, state highway engineer and under the immediate supervision of the writer. The contract was awarded to A. Guthrie & Co., of St. Paul, Minn., and Portland, Ore., at a total price of $213,602.50, and with the following typical unit prices:

Item	Amount		Unit
Class A concrete (superstructure)	2,100	cu.yd.	$22.50
Class B concrete (foundations)	1,050	cu.yd.	$18.50
Structural steel	900,000	lb.	$0.12
Metal reinforcement	270,000	lb.	0.055
¾-in. gunite	23,000	sq.ft.	0.17
1½-in. gunite	22,000	sq.ft.	0.29

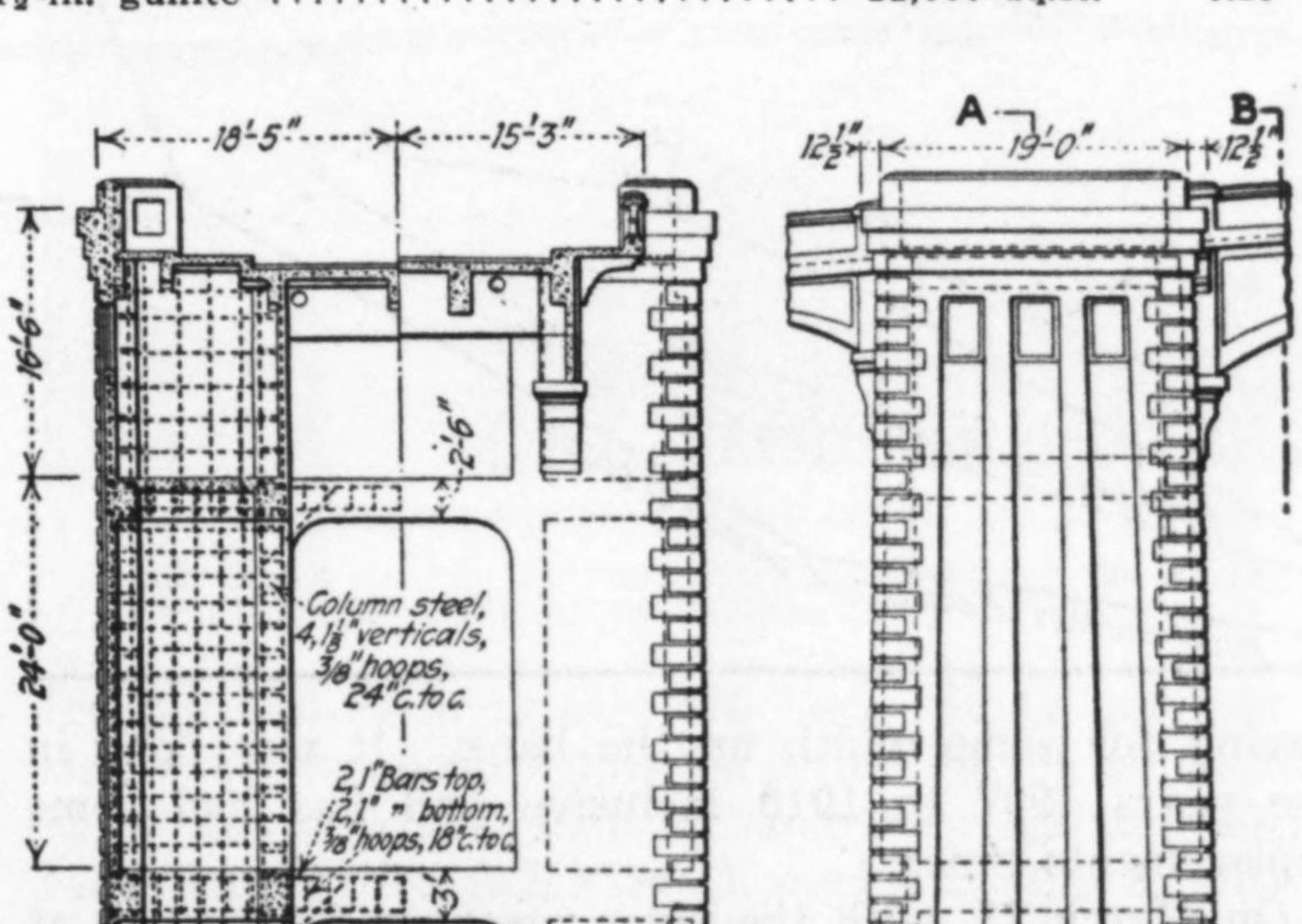


FIG. 6. DETAILS OF MAIN ARCH PIER