Engineering News-Record

A Consolidation of Engineering News and Engineering Record

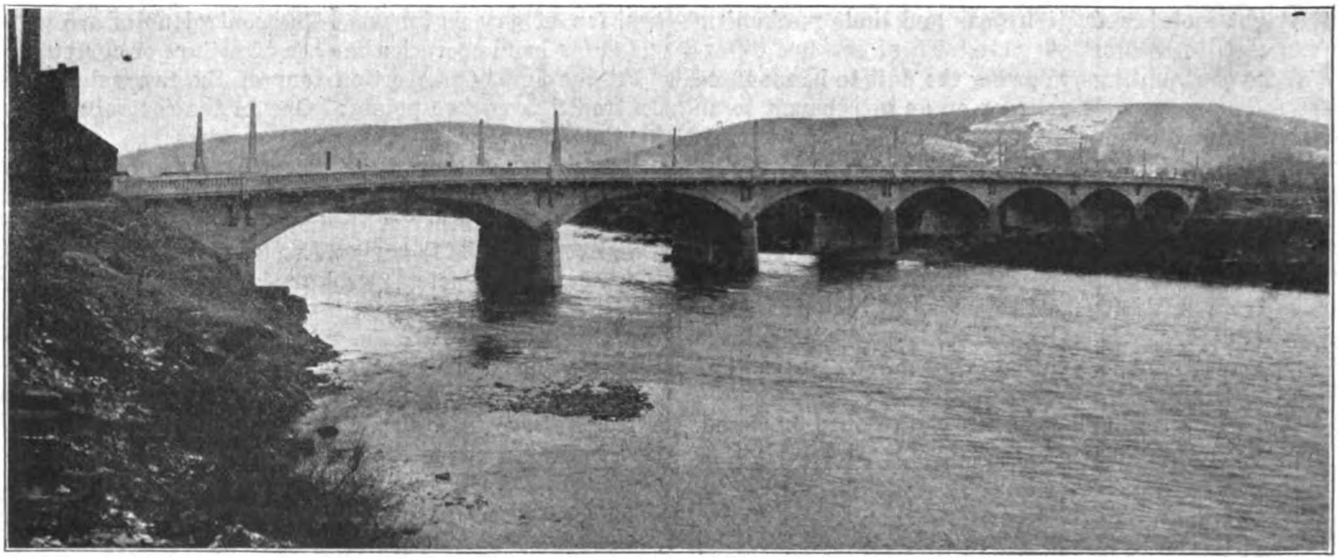
A JOURNAL OF CIVIL ENGINEERING AND CONSTRUCTION

ISSUED WEEKLY

VOLUME LXXXVIII

January 1 to June 30, 1922

McGRAW-HILL COMPANY, INC.
10TH AVENUE AT 36TH STREET
NEW YORK



Some Features of the Chemung River Concrete Bridge

Careful Balancing of Design Alternatives and Definite Purpose in Construction Planning—Reinforcement Interlocking, Waterproofing Details and Flood Protection of Plant

PILE foundation design and the problems of expansion and construction joints, drainage and waterproofing, were given especial attention in constructing the seven-arch, reinforced concrete bridge at Corning, N. Y. Incidentally all parts of the bridge are reinforced concrete poured in place even to the foundation piles and the lamp posts on the parapets. The reinforcement is, consequently, interlocked from the piles up through footings, pier shafts, arch rings, spandrel walls and sidewalks to the hand railing and lamp posts. Architecturally, simple, pleasing lines and masses were sought.

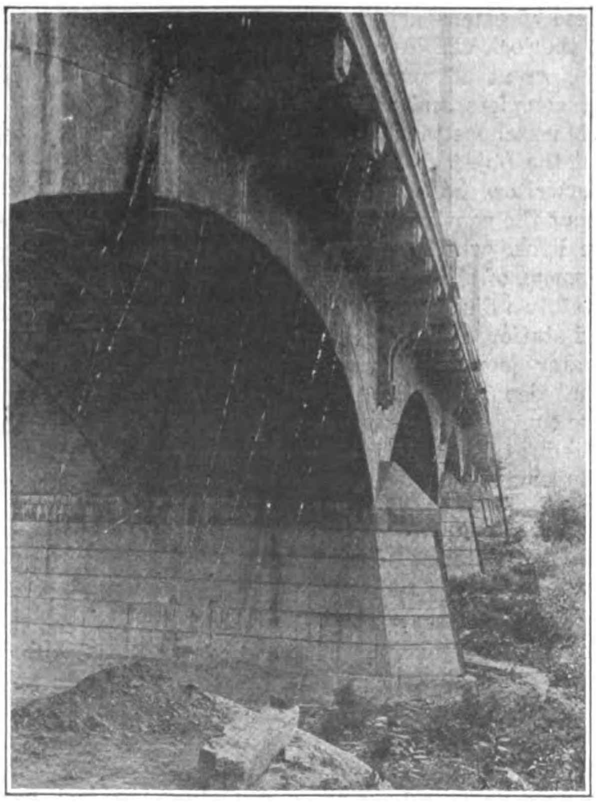


FIG. 1. LOOKING ALONG THE SEVEN SPAN MEMORIAL BRIDGE AT CORNING, N. Y.

The structure is 752 ft. long and is composed of seven three-centered segmental barrel skew arches 92-ft. 3-in. span and 11-ft. 3-in. rise. The arches are 41 ft. in width face to face of parapet walls and support a 36-ft. roadway on earth back fill, and two sidewalks 6 ft. in the clear which cantilever 5 ft. beyond the face of the parapet walls. The total width of structure face to face of coping beams is 51 ft. The depth from the crown of the road to the crown of the arch is 1 ft. 8 in., a minimum requirement in placing a 10-in. water main in an earth cushion under a concrete pavement.

Three streams, draining about 4,200 square miles, come together near Corning to form the Chemung River, which is crossed by the bridge. These streams are flashy, and floods come quickly and subside almost as quickly. In the maximum recorded flood of March 14, 1918, the river rose 19 ft. The rate of rise has, at times, been 1 ft. an hour. In computing waterway, the length of the bridge 752 ft. was set as the distance between the dikes, which have contained past floods, and the roadway was set at an elevation which gave an aggregate waterway between piers below springing lines sufficient for the record flood, leaving the arch spaces above springing lines as a factor of safety in case of greater floods.

With the foundation conditions, 2 to 6 ft. of gravel and clay, then stiff clay down to 16 ft., and then hardpan, the first problem was to choose between shallow footings on piles and deep foundations protected by coffer-dams. Because of the flood conditions and the favorable bottom, the advantage was in favor of piles. With piles adopted, the alternatives were wood and concrete and, in case of concrete, they were precast and cast-in-place construction.

The ground was so compact that it was thought that it would be impossible, or at least impracticable, to attempt to drive wood or precast concrete piles. In fact, the piles adopted required 8 blows for a 1-in. penetration under a No. 1 Vulcan Steam hammer, and in some cases this was raised to as many as 12.

While the unit cost of concrete piles was higher than that of wood piles, about half as many were required,

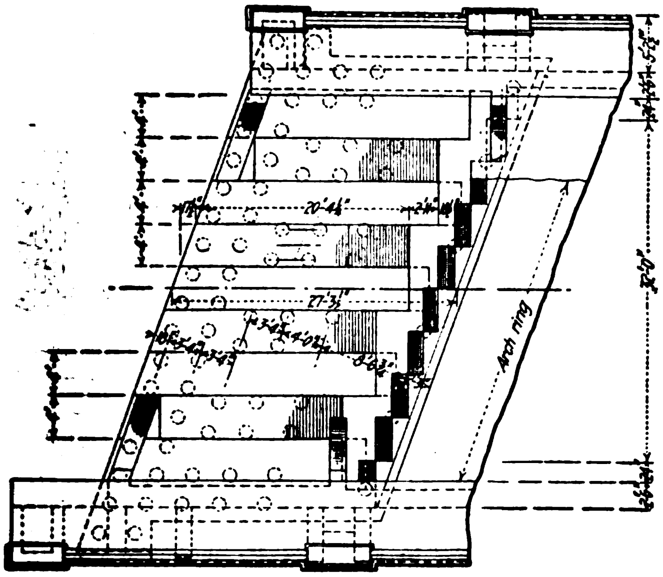


FIG. 2. PLAN OF ABUTMENT SHOWING PILE GROUPING

assuming a bearing power of 35 tons for concrete and of 18 tons for wood. Using fewer piles reduced the area of the pier footing; this saved excavation and also concrete and reinforcement, since the smaller footing could be plain concrete but the large footing, required by the wood piles, must be heavily reinforced. Wood piles would also have necessitated a deeper foundation at some of the piers. The great superiority of the concrete pile appeared, however, in the design of the abutments.

With concrete piles selected, the cast-in-place type had advantages over the precast type as follows: (1) No time was required for curing and the work of driving could be started immediately after the excavation for the first abutment was completed. (2) Reinforcing rods could be inserted immediately after the piles were poured, to bond the footing course to the piles. Additional stubs placed in the footing course lapped the vertical bars in the pier shaft and also in the face walls of the abutments. This arrangement was effective in resisting eccentric thrusts in constructing the arch ring.

The advantage of the concrete piles in the design of the abutments was signal. Here any reduction in length saved a similar length of foundation excavation to a depth of 19 ft., since the abutments were set back into the dikes. The thrust of the arch against the abutments was fixed in amount and direction and to get the vertical load of abutment and fill necessary to resist the arch thrust required a width of abutment of approximately 28 ft. With the amount of eccentric pressure on the outer toe of the abutment, it was impossible to place enough wood piles within the pressure area of the 28 ft. width, without greatly exceeding the safe loading on the piles of the outer row.

In designing the abutment, concrete piles were so spaced under the varying pressure area of the 28 ft. width that each pile was loaded uniformly to carrying capacity. The center of gravity of the cluster of piles under each 4-ft. buttress, Fig. 2, was made to coincide with the resultant pressure of the external loads. It is remarked here that the outer face, of the abutment, was made to act as an abutment buttress in addition to its function as a retaining wall and a support for the sidewalk. The resultant pressure of the abutment and arch thrust is outside of the middle third of the abutment base in the contracted area of the concrete pile plan. Therefore the six piles in a row along the inner face of the abutment adjacent to the arch, theoretically are not taking any vertical load, but these six piles tied to the abutment footing by projecting bars offer a factor of safety against overturning and satisfy a possible discrepancy between the actual and theoretical action. Three 1-in. square reinforcing bars were placed on the side of every pile facing the arch to reinforce the pile in shear and these bars also projected into the footing course. The ends of the buttresses, due to their shallow section in the area of maximum pressure, were reinforced in shear by $\frac{3}{4}$-in. stirrups spaced 12 in. apart.

Two features of the arch design stand out. One is the provision for subsequently interlocking the walks and balustrades to the arch ring and spandrels, and one is the provision for expansion, drainage and waterproofing. In general, the bridge was designed to carry 150 lb. per square foot over the roadway. The sidewalks were designed to carry 100 lb. per square foot. The maximum stress in the concrete was set at 650 lb. per square inch and in the steel at 16,000 lb. per square inch.

The arch ring, Fig. 3, 1 ft. 8 in. at the crown and 2 ft. 6 in. at the haunch, was made three centered to conform to the pressure curve of the load. The parapet wall, Fig. 4, 1 ft. 6 in. in thickness supporting the sidewalk and balustrade, is of the buttress type and is reinforced in the longitudinal direction. The reinforcement of the buttress, which is anchored to the arch ring, is carried through the upper plane of the sidewalk beams in line with the buttresses and furnishes the necessary reinforcement for these beams. The depths of these beams, to satisfy architectural considerations, gave a very low shearing stress and no additional reinforcement was necessary. The outer 1 ft. 6 in. lamina of the arch ring, upon which the spandrel wall and sidewalk sit, was made 8 in. deeper than the arch

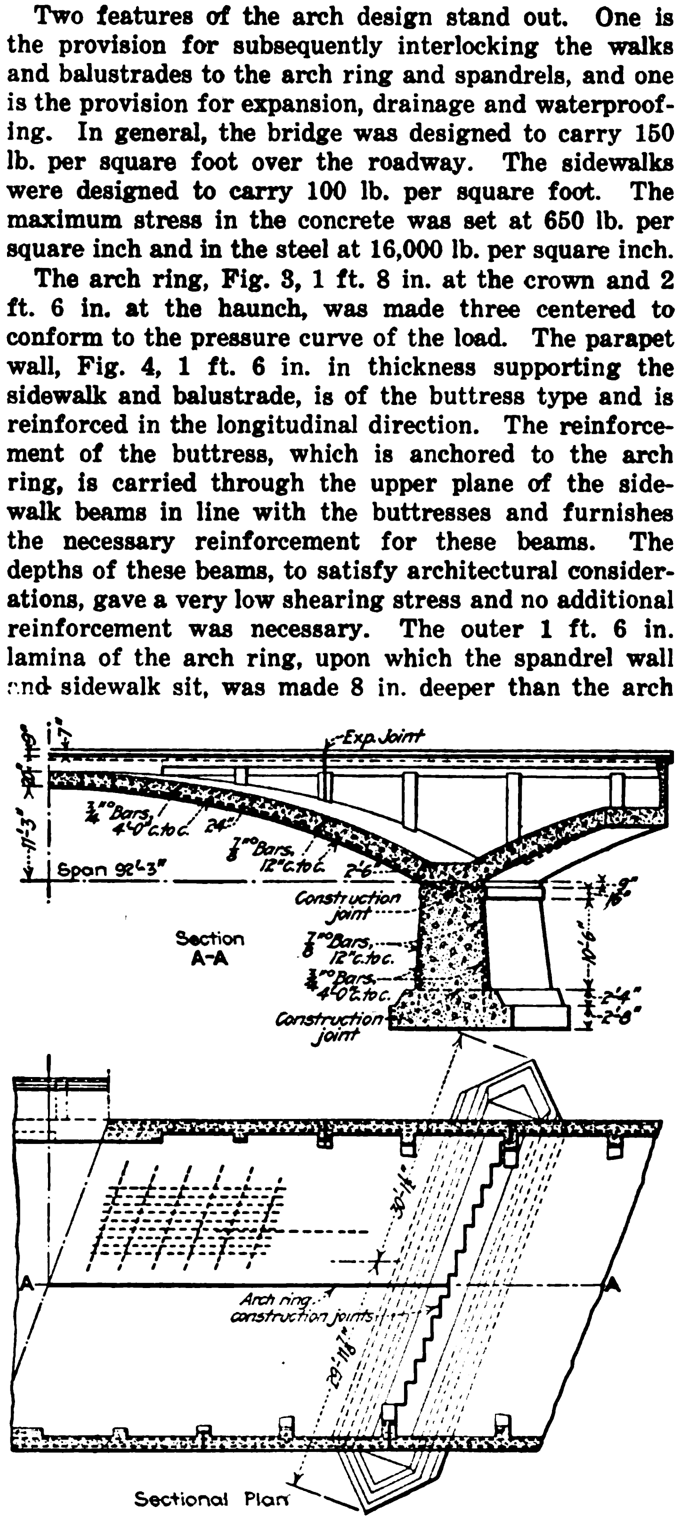
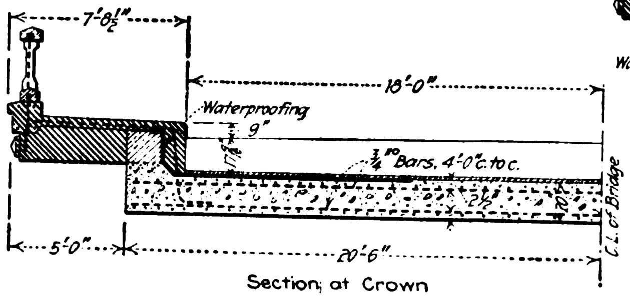


FIG. 3. ARCH SECTION AND PLAN EMPHASIZING CONSTRUCTION JOINTS

ring proper at the crown, and 12 in. deeper at the haunch to carry this extra loading. Poured simultaneously with the half width of the arch ring, it formed a suitable construction joint from which the spandrel wall could be run in a separate operation. Also by raising the construction joint above the extrados, a curb was formed that precludes seepage to the face of the wall, if the waterproofing membrane fails. The wall was carried up to the underside of the sidewalk, with recesses to receive the sidewalk beams.



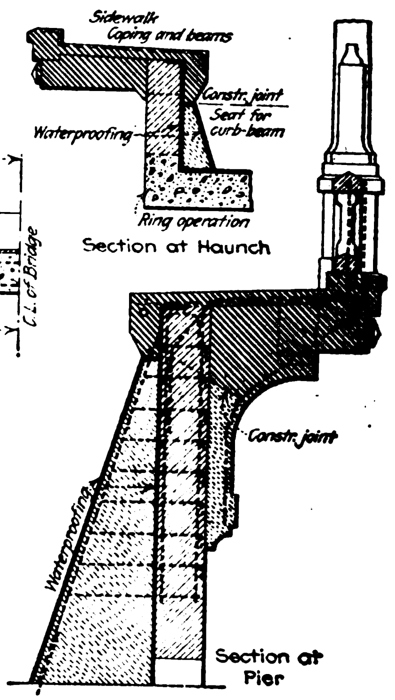


FIG. 4. SPANDREL, SIDEWALK AND PARAPET DETAIL

Immediately after the forms for the spandrel walls had been removed, the waterproofers covered the backs of the arch rings with a membrane composed of two plies of an asphaltic saturated cotton cloth laid in three swabbings of hot asphalt. This membrane, Fig. 5, was carried up the back of the parapet walls, around the buttresses and flashed over the tops of the walls and into the sidewalk beam recesses. This operation was followed immediately by a protective layer of 1:3 sand mortar 1½ in. thick over the backs of the arches to protect the membrane from being punctured in subsequent construction operations and in back filling.

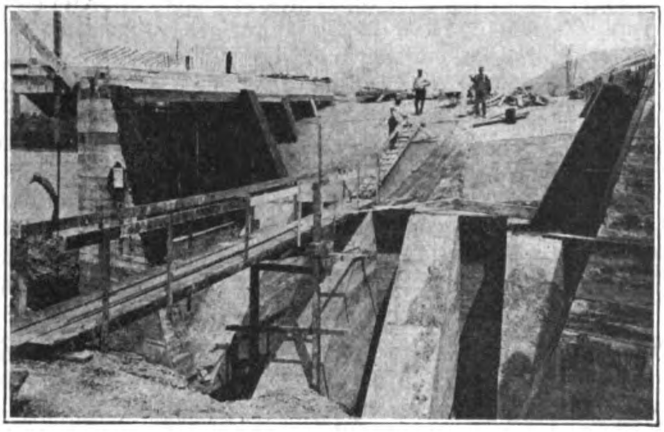


FIG. 5. SOUTH ABUTMENT BUTTRESSES AND WATERPROOFING

The next concreting operation was the simultaneous pouring of the curb beam supported on the 6-in. ledge top of the wall counterfort, the sidewalk and the fascia coping beam. With this concrete operation, the waterproofing is positively secured and flashed, with no danger of being pulled away from the wall by settlement of the back fill or of water working behind the membrane, since it is completely protected by the curb beam. The heavy curb beam built with the sidewalk also gives a certain balance to the sidewalk overhang and furnishes a sort of lock giving added strength by the tensile strength of the concrete. The top of the coping beam is 7 in. above the sidewalk avoiding a construction joint on the sidewalk level where the drainage of the sidewalk might seep through to the face of the coping. The coping is reinforced to act as a beam carrying the balustrade and half the sidewalk load to the sidewalk cantilever beams. The sidewalk slab, therefore, is reinforced by light triangular mesh in the transverse direction extending to the edge of the curb. Vertical rods were embedded in the coping beam and projected the full height of the balustrade to hold the latter substantially to the coping.

Expansion joints were placed at the quarter points of each span, approximately at the point of contraflexure in the arch ring. To avoid the uncertain action of a sliding joint of any description, the superstructure at this point was cut clearly in two by a bulkhead. The sidewalk beam and counterfort, therefore, are built in two separate units and reinforced to take the separate end reactions. The joint was built up by multiple layers of tar paper. A steel plate covers it through the sidewalk and it is further protected from seepage by copper flashing embedded in the concrete.

In planning the equipment, the frequency of the floods suggested mobility and retreat and the high ground on the north bank of the river, with the protection afforded by the dike, offered the haven of safety.

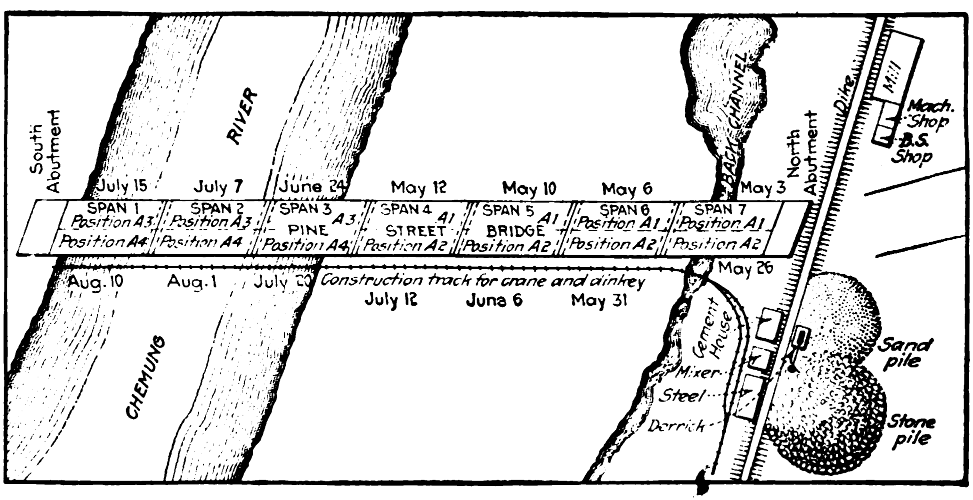


FIG. 6. PLANT LAYOUT AND PROGRESS SCHEDULE

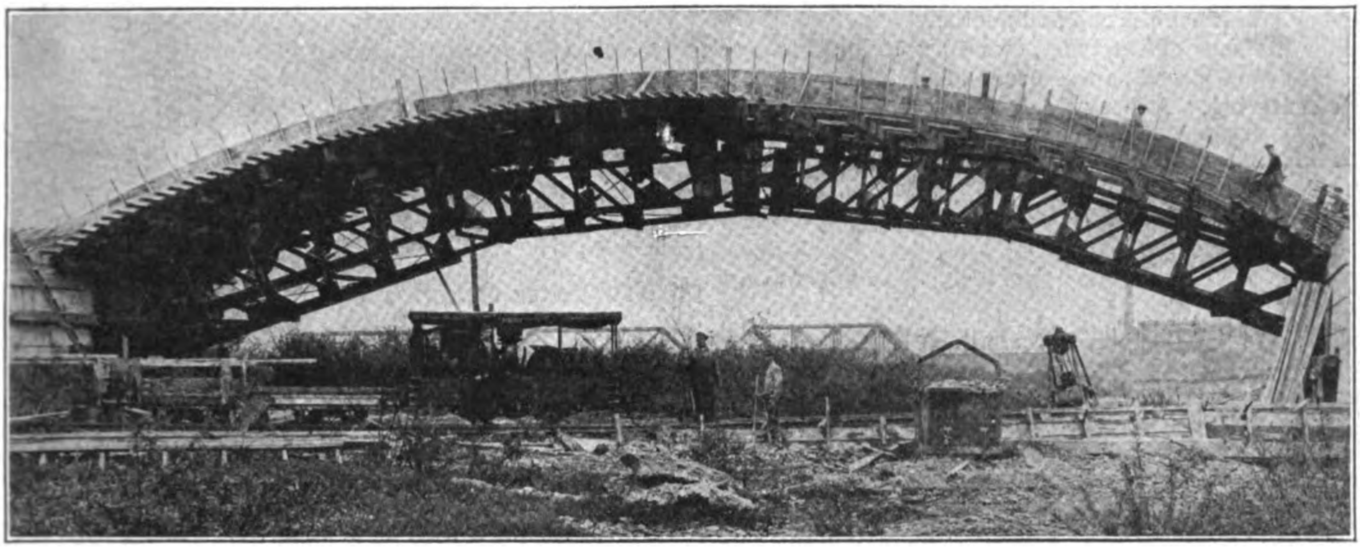


FIG. 8. CENTERS AND ARCH RING FORMS IN PLACE

Therefore the construction of the bridge was planned around a locomotive crane, with a 60-ft. boom, and a construction plant as indicated by Fig. 6. A derrick, having a 75-ft. mast and 70-ft. boom was placed within reach of the stock piles, the cement and steel store houses and the concrete mixing plant. The wooden forms were built near by so that they could also be handled by the derrick. These forms were built in heavy units, Fig. 7, so that they could be reused.

Work started on Oct. 5, 1920, excavating for the south abutment. Building and grading a ramp from the top of the bank the crane worked its way down to the river's edge, and thence across on tracks laid in the bed of river during low water, driving the sheeting and making the excavations for the river piers as it proceeded. Immediately after the excavation was completed the pile-driving equipment was lowered into the hole and the driving of the piles commenced. It was estimated that 15 piles could be driven in a working day and this progress was made in driving 633 piles in 42 working days. About two days were required to move the driver from pier to pier. The driving schedule was interrupted for a short period in the early part of November due to an unprecedented flood for this particular month. The specifications required eight blows of a No. 1 Vulcan steam hammer to 1 in. penetration and where driving was especially difficult twelve blows were given to secure depth. The average driven length of the piles was 12 ft.

Work on the piers followed the pile driving. The foundations for the river piers were poured before floods of any consequence occurred, but the pier shafts were completed between floods, with a very narrow margin of time in each operation. Work on the piers was advanced so that the superstructure could be started from the north end and be completed on the high ground during spring months to the river's edge before the June flood. Fortunately, due to the unusual drought this flood did not materialize but the work was developed accordingly to the predetermined plan which contemplated that high water would cause delay but would not cause serious damage to the work or plant.

Fig. 8 shows the structural steel centering. One unit, forming a half-width arch barrel, was composed of two ribs made up of four separate Pratt truss sections, with parallel top and bottom chords. These sections were made to conform to the approximate curve of arch by wedge-shaped diaphragms at the crown and quarter points of the truss. The true curve for the lagging was formed by wood shims of variable height, which raised to the proper level and inclination the 6 x 12-in. purlins set parallel with the skew axis of the arch. Over these purlins 4 x 10-in. longitudinal riders, 3 ft. 3 in. center to center, were warped to the true curve of the intrados and to them the 2 x 6-in. lagging, laid parallel to the skew axis, was nailed. The weight of one rib assembled was 9.2 tons which was easily handled by the crane, Fig. 9, to its position on the timber bent bolted to the piers.

This supporting bent was capped by a 10- x 10-in.

FIG. 7. BALUSTRADE FORM UNITS BEING PLACED

timber. Between the cap and another 10 x 10-in. timber under the shoe of the truss, two 10 x 10-in. x 4-ft. oak wedges were placed as the means of raising or lowering the centering. A heavy oak thrust block, cut to the

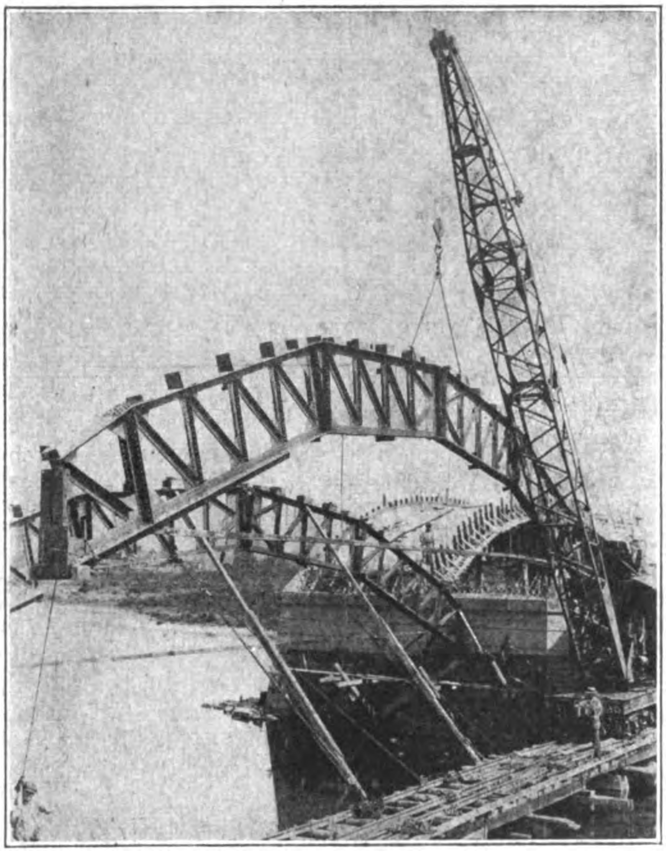


FIG. 9. DERRICK SETTING STEEL RIB FOR ARCH CENTER

skew angle of the bridge, was set back of the truss shoe to transfer the horizontal thrust to the piers. Rollers were placed between the two 10 x 10-in. timbers before the wedges were struck, on which the entire unit, truss, lagging and all, was jacked and rolled over to the second half of the arch ring.

The sequence of arch construction is shown in Fig. 6. The rings were poured in the order named and each one is a continuous operation, pouring alternately from either end. Including a 9-in. lift over the half area of the pier, there were 160 cu.yd. in each operation poured in eight hours with a one-yard mixer. The short time between arch ring operations was employed without reduction of force in the construction of the parapet walls, sidewalk and balustrade. When the last half of the arch ring was poured Aug. 10, the parapet walls were almost completed on all the spans and much of the sidewalk and railing was completed. In this manner operations were distributed over the entire work so that a maximum force could be economically employed up to the last three weeks. Remarkably fast work was done with the centers erection and concrete placing in the arches. Within one month after the arrival of the centers four halves of arches had been poured.

The bridge was planned and supervised by A. B. Cohen, consulting engineer, New York City. J. I. Bingham was resident engineer in field charge. The improvement was carried on through the office of W. O. Drake, Superintendent of Public Works, Corning, N. Y. Bush, Roberts & Schaefer Co., of New York, were the general contractors with J. E. Jones superintendent of construction. The piles were driven under separate contract by MacArthur Concrete Pile & Foundation Co. and the centers furnished by the Blaw-Knox Co.

Columns and Walls Lifted By Swelling Clay Under Floor

New Texas Church Built in Dry Year Damaged When Clay Enclosed by Foundation Walls Became Saturated

By F. E. Giesecke
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SHORTLY after construction of the University Baptist church in Austin, Tex., a change in the moisture content of a 6-ft. layer of clay under the building produced a vertical expansion of about 4 in. and thereby lifted and broke the concrete floor slab, lifted iron and wooden columns, with their superimposed loads, off the concrete piers originally supporting these columns, and lifted a brick wall together with a portion of its concrete foundation wall, after having overcome the tensile strength of the concrete foundation wall. The clay is decomposed Eagleford shale.

The building was erected in two installments; the first portion, which included the Sunday school department and the basement of the main building, was erected by Johnson & Chambers, of Austin, from April, 1918, to January, 1919; the second portion, which completed the building, was erected by Hedrick Construction Co., of Houston, from August, 1920, to October, 1921. The architect was Albert Kelsey of Philadelphia;

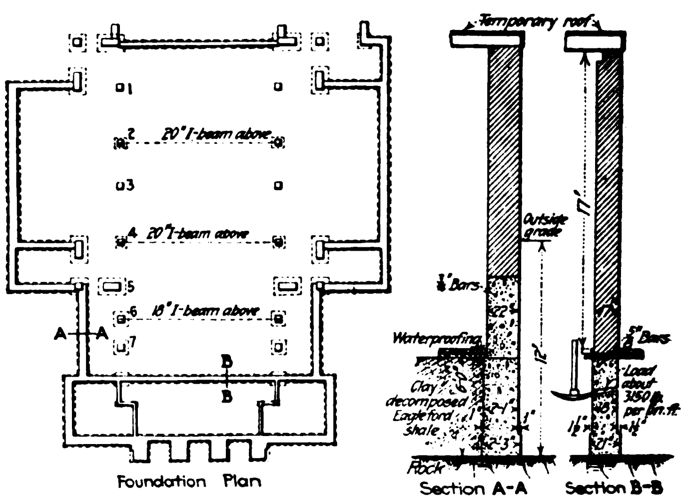

FIG. 1. PLAN OF DAMAGED CHURCH

the supervising architect of construction was Roy Thomas of Austin.

In Fig. 1, a plan of the basement of the main building, several walls are lettered for reference in the following description of the effect of the expansion of the clay. Fig. 3 shows the rainfall in Austin for 1917, 1918, and 1919; the first construction period of the building and the date when the failure of the concrete floor slab became noticeable are marked. As appears from this figure, the erection of the building followed a very dry year, the concrete floor slab was poured in July, when the ground was very dry, and the failure of the con-