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MOVEMENT OF PIERS DURING THE CONSTRUCTION OF MULTIPLE-SPAN REINFORCED CONCRETE ARCH BRIDGES

BY

WILBUR M. WILSON

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MOVEMENT OF PIERS DURING THE CONSTRUCTION OF MULTIPLE-SPAN REINFORCED CONCRETE ARCH BRIDGES

BY

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Introduction

1. *Object and Scope of Investigation.*—The design of reinforced concrete arches is based upon the assumption that the piers supporting the arches are fixed. Knowing that in some cases piers of arch bridges have moved while the superstructure was being built, observations were made on a number of multiple-span arch bridges during construction to determine what, if any, movement of the piers took place. This bulletin is a report of the observations made under the supervision of the author on five multiple-span arch bridges. The piers for two bridges rest upon rock; for one bridge some piers rest upon rock and others upon piles driven into gravel; and for two bridges all piers rest upon piles driven into gravel. Three bridges have twin-rib, open-spandrel, right arches; two bridges have skew, spandrel-filled barrel arches, one having a skew of 6 deg. 56 min. and the other a skew of 46 deg. 52 min. The piers for both skew bridges rest upon piles driven into gravel.

The observations on the twin-rib arches included tipping of the piers and, in the case of one bridge, vertical movement of the piers and changes in the arch span; the observations on the piers of the skew barrel arches included tipping, rotation about a vertical axis, and horizontal and vertical translation.

The rotation of a pier about a horizontal axis in the base produces stress in the rib or barrel due to the rotation at the springing and also to a change in the span resulting from the tipping of the pier.

The arch ribs and barrels were analyzed algebraically by the well-known elastic theory of arches, and the unit strain at the springing and the crown were determined for a unit change in span and for a unit rotation of one abutment. In this analysis, the effect of the deck, spandrel columns, and spandrel walls was neglected. The unit stress corresponding to unit strain can be determined only when the stress-strain relation is known. Unfortunately, the stress-strain relation varies with the age of the concrete, with the time interval during which the strain occurs, with the physical and geometrical properties of the aggregate, and with the degree of consolidation that occurs when the concrete is placed. The unit strain cannot, therefore, be considered as an accurate measure of the unit stress in the concrete, although it does give a rough indication of the quantitative value of the stress. The pier movements have been converted into stresses because most readers think in terms of unit stress rather than in terms of unit strain. The translation of strain into stress has been made on the basis that the modulus of elasticity, E, equals 1 000 000 lb. per sq. in., not because this is believed to be the correct value, but because the stress due to a given strain can be determined so easily for any other value of E, when the stress for E equal to 1 000 000 is known.

2. *Acknowledgment.*—This investigation has been a part of the work of the Engineering Experiment Station of the University of Illinois, of which DEAN MILO S. KETCHUM is the director, and the Department of Civil Engineering, of which PROF. W. C. HUNTINGTON is the head. It also constitutes a part of the program of the Committee on Concrete and Reinforced Concrete Arches of the American Society of Civil Engineers.*

The direct expense of the investigation was paid from funds provided by the American Society of Civil Engineers and the Engineering Foundation.

The Yorkville bridges and the Rockton bridge were designed by the Illinois Division of Highways; FRANK T. SHEETS, Chief Highway Engineer; G. F. BURCH, Bridge Engineer; G. N. LAMB, District Engineer for the Yorkville district; and O. F. GOEKE, District Engineer for the Rockton district. The field observations on the Yorkville bridges were made by R. H. UNDERHILL, and on the Rockton bridge by JAS. F. GOLDSBERRY, Resident Engineers of the Illinois State Highway Department for the respective bridges.

The Oliver Avenue and Kentucky Avenue bridges, Indianapolis, were designed by DANIEL B. LUTEN and constructed under the supervision of F. C. LINGENFELTER, City Engineer for the City of Indianapolis. The observations were made by PAUL H. JOHNSON, an engineer whose services were paid for from the funds provided by the American Society of Civil Engineers and the Engineering Foundation.

II. RESULTS OF OBSERVATIONS

3. *Fox River Bridge at Yorkville, Illinois.*—Illinois State Highway, Route 18, is carried over the Fox River at Yorkville by means of a

*The members of this committee are C. T. Morris, Chairman; E. H. Harder; A. C. Janni; George E. Beggs; and W. M. Wilson.

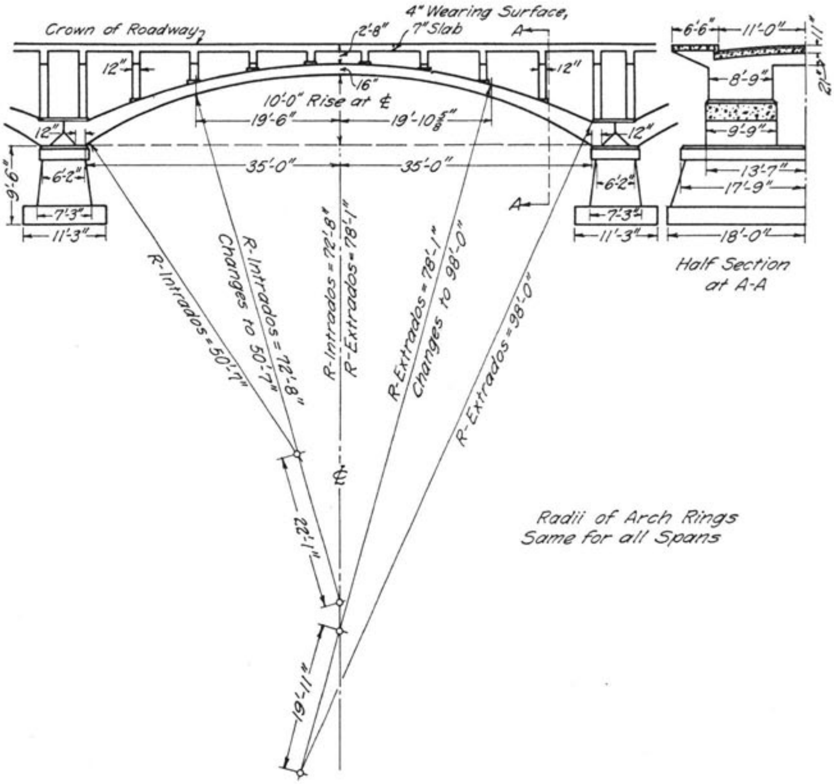

Fig. 1. General Dimensions, South Bridge, Yorkville, Illinois

series of reinforced concrete arches. An island in the river divides the crossing into two three-span bridges, designated in this report as the south bridge and the north bridge, respectively.

South Bridge

The south bridge is shown in Fig. 1. The three arches are alike, the span and rise being 70 feet and 10 feet, respectively, measured on the intrados. Each span is a twin-rib open-spandrel arch supported on low piers that extend to bedrock. Observations were made on the two intermediate piers to determine the rotation that took place during construction, the rotation being measured with a 20-in. level-bar* set on steel plugs embedded in the concrete at or near the springing of the arch. The results of the observations are given in Fig. 2.

*This instrument is described in detail in "The Effect of Climatic Changes Upon a Multiple-Span Reinforced Concrete Arch Bridge," Univ. of Ill. Eng. Exp. Sta. Bul. 174, 1928.

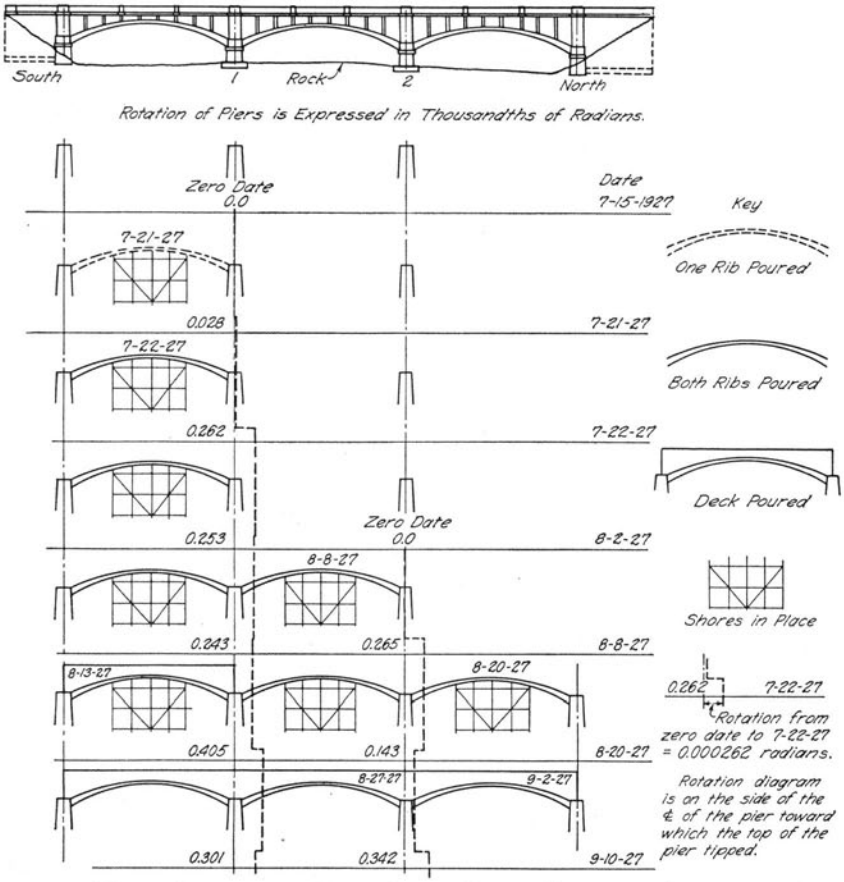

FIG. 2. ROTATION OF PIERS, SOUTH BRIDGE, YORKVILLE, ILLINOIS

The sketches show the construction stage of the bridge at the time each observation was made. The distance from the vertical broken line to the centerline of a pier represents to scale the magnitude of the rotation. The numerical value of this rotation, expressed in thousandths of radians, is given just below the sketch of a pier for each date on which observations were made. The maximum rotation recorded is for pier 1 on August 20, 1927, when the rotation was 0.000405 radian. A rotation of 0.001 radian produces a maximum unit strain in the rib at the springing of 0.000048 in. per in.* With E assumed to be 1 000 000 lb. per sq. in., the unit stress produced by a

*The strains resulting from pier movements for the five bridges were computed and checked by Nathan Newmark, Glenn Murphy, and Charles O. Harris, Research Graduate Assistants.
Calculated value determined by the elastic theory applied to the rib alone, the effect of the deck and spandrel columns being neglected.

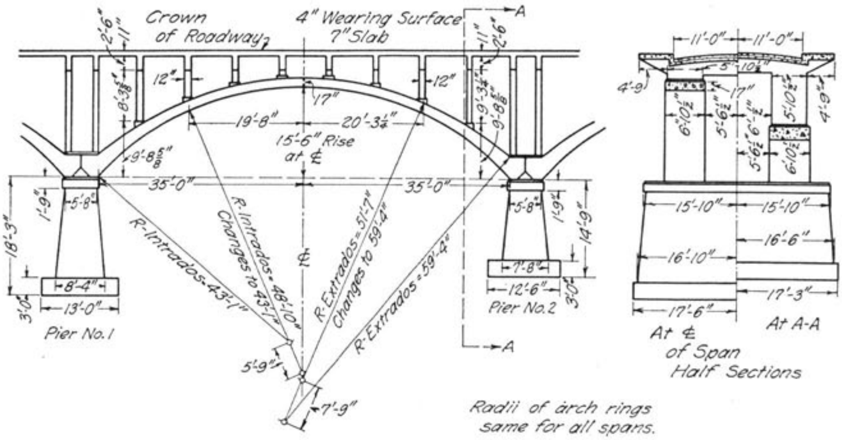

FIG. 3. GENERAL DIMENSIONS, NORTH BRIDGE, YORKVILLE, ILLINOIS

rotation of 0.000405 radian would therefore be only 19 lb. per sq. in. Apparently the stress is so small as to be of no particular interest to the designer. This is the stress due to rotation of one end of the rib, the other end being fixed and the span constant.

If the whole pier is assumed to have tipped 0.0004 radian about an axis in the base, a point at the springing 200 in. above the base will move horizontally 0.08 in. A change in span of 0.08 in. will produce a unit stress, with $E = 1\ 000\ 000$, of 34 lb. per sq. in. at the springing, and 45 lb. per sq. in. at the crown.

North Bridge

The north bridge over the Fox River at Yorkville is shown in Fig. 3. It is similar to the south bridge except that the rise is greater relative to the span. For this bridge also the piers are low and rest upon rock.

The maximum rotation recorded was for pier 1 on July 8, 1927, when the pier was found to have rotated 0.00112 radian from the position it occupied April 30, 1927, just before the adjacent ribs were poured. This rotation produces a unit strain* at the springing of 0.000068 in. per in., which, with E taken at 1 000 000 lb. per sq. in., corresponds to a stress of 68 lb. per sq. in., the direct effect of the rotation. If a pier rotates about some point in the base the rotation causes a horizontal movement of the pier at the springing parallel to the axis of the bridge. Assuming the center of rotation to be 200

*See note bottom of page 8.

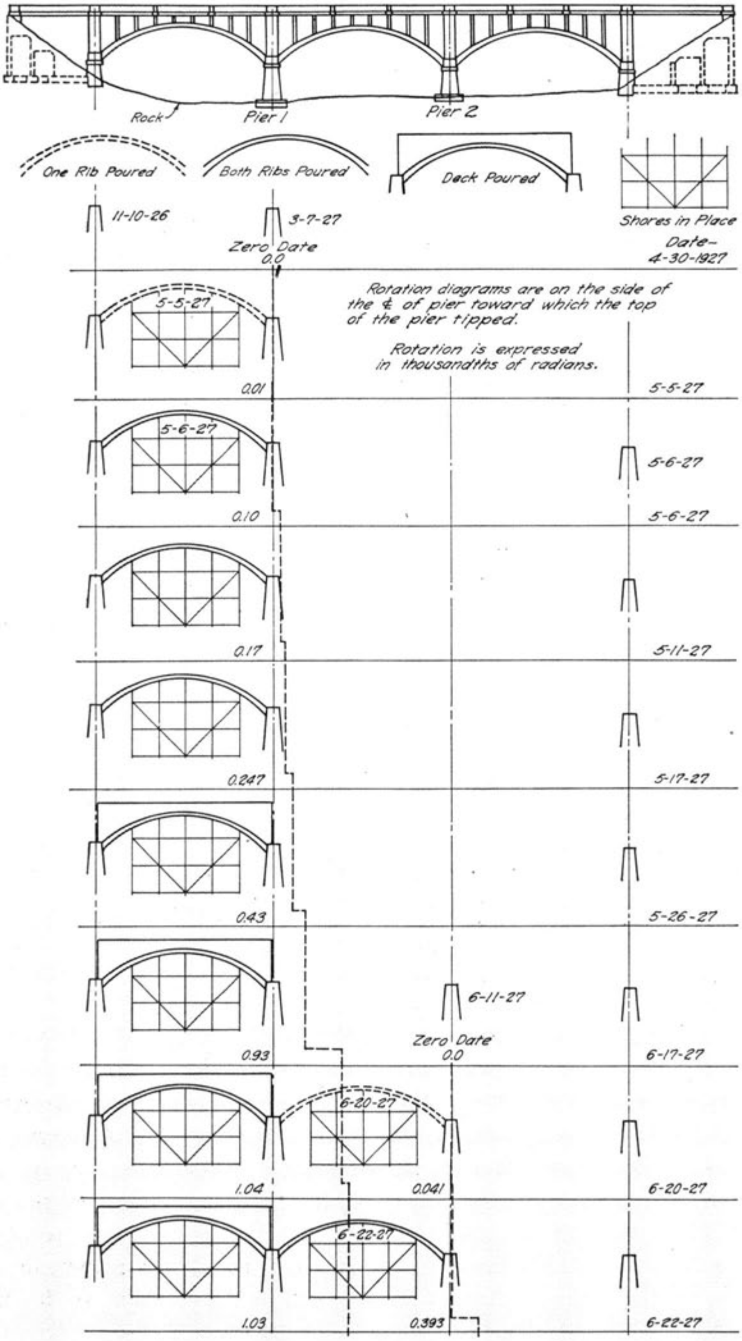

FIG. 4. ROTATION OF PIERS, NORTH BRIDGE, YORKVILLE, ILLINOIS

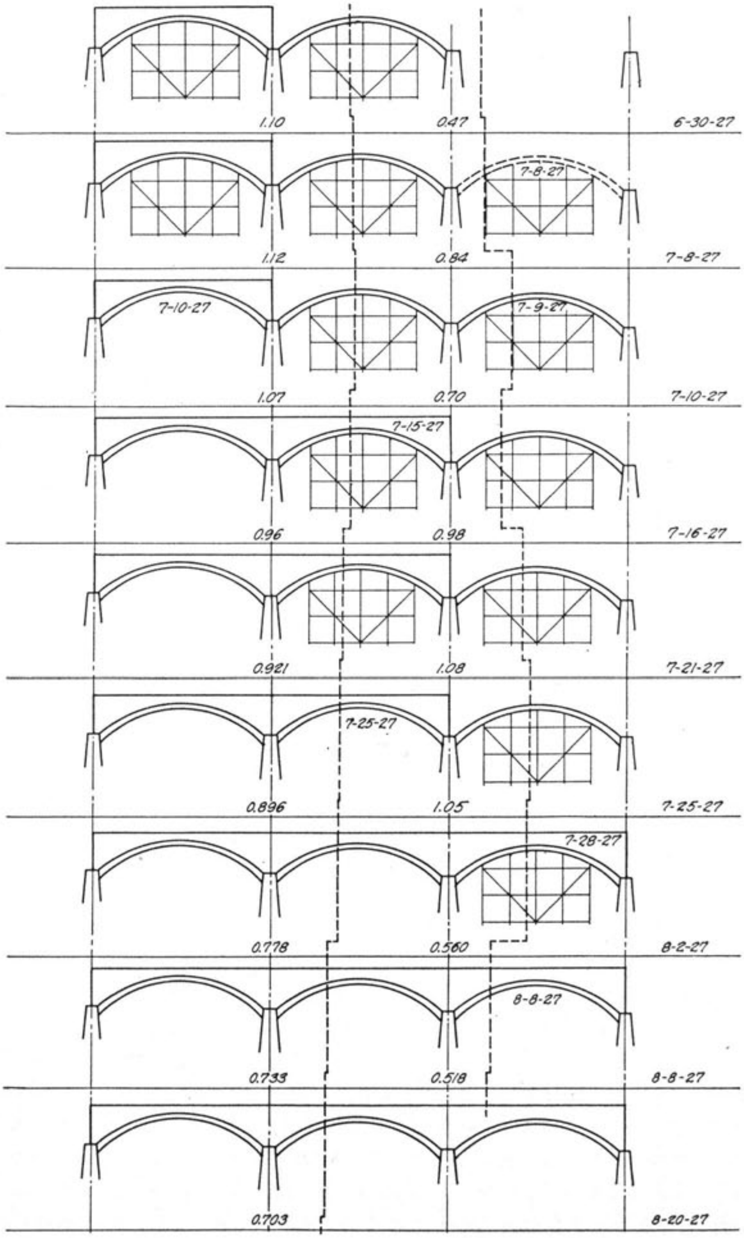

FIG. 4 (Concluded). ROTATION OF PIERS, NORTH BRIDGE, YORKVILLE, ILLINOIS

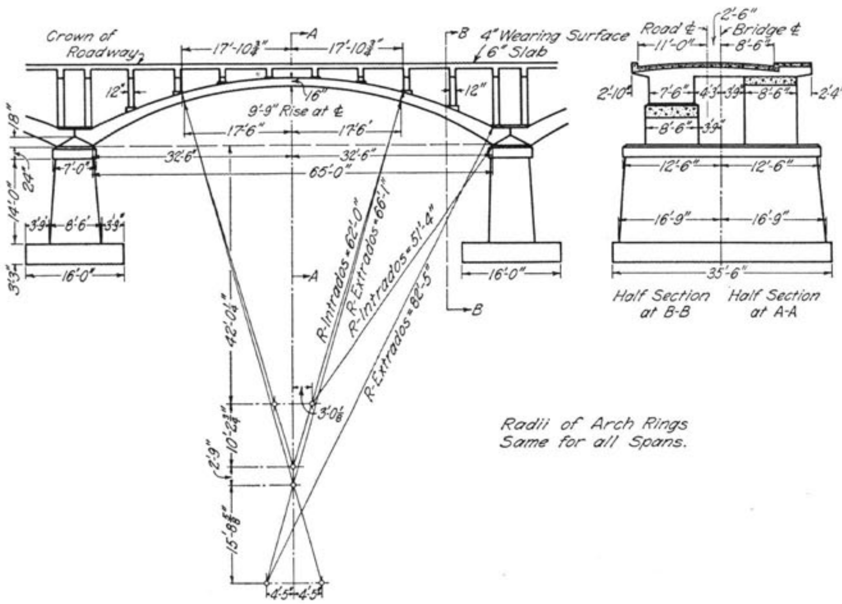

FIG. 5. GENERAL DIMENSIONS, WASHINGTON STREET BRIDGE, ROCKTON, ILLINOIS

in. below the springing, a rotation of 0.00112 radian will produce a horizontal motion of 0.224 in. This change in span will cause a stress* of 70 lb. per sq. in. at the springing and 73 lb. per sq. in. at the crown. If one pier tips out and the other remains stationary the change in span and the rotation produce stress of the same sign at the springing. In the case of this bridge both piers tipped in the same direction and there was probably little or no change in span.

4. *Washington Street Bridge at Rockton, Illinois.*—The highway bridge at Rockton, Illinois, shown in Fig. 5, consists of eight twin-ribbed, open-spandrel, reinforced concrete arches. The spans have a rise of 9 ft. 9 in. and a length of 65 ft., measured on the intrados. The foundation conditions are unique in that the north abutment and piers 1 to 4, inclusive, are supported on piles driven into gravel whereas the other piers and the south abutment are founded on rock. The rotation of the piers at different stages of construction is given in Fig. 6.

The rotation of the south abutment and of piers 6 and 7 is very small, in no case being equal to 0.0002 radian. The rotation of the north abutment and of the other piers, although in no case excessive, is much greater. The former piers and abutment are supported on

*See note bottom of page 8.

TABLE 1
CHANGE IN SPAN OF ARCHES, WASHINGTON STREET BRIDGE, ROCKTON, ILLINOIS

Date	Temperature, deg. F.	Change in Span No.							Remarks
		2	3	4	5	6	7	8	
12–19–25	39	0	0	0	0	0	0	0	Initial reading. Before span 2 was poured.
12–29–25	15	+0.16	+0.01	+0.00	+0.06	+0.10	+0.07	+0.01	After pouring ribs of span 2 and before removing shores of span 4.
1– 5–26	33	+0.06	+0.02	+0.04	+0.05	+0.02	+0.00	+0.00	After removing shores of span 4.
2–18–26	22	−0.02	−0.08	−0.06	−0.08	−0.05	−0.08	−0.08	Before ribs of span 1 were poured.
2–28–26	45	+0.03	+0.05	−0.09	−0.07	−0.08	−0.02	+0.02	After ribs of span 1 were poured.

Changes in span are given in inches. Plus (+) sign indicates an increase, minus (−) sign indicates a decrease, in span.

rock; the latter, except pier 5, are supported on piles; pier 5 is located at the edge of the rock ledge. The maximum rotation recorded is 0.000597 radian for pier 3. The maximum rotation for the other piers founded on piles and for pier 5 is nearly as great. The direct effect of a rotation of 0.0006 radian with $E = 1\ 000\ 000$ lb. per sq. in. is 42 lb. per sq. in.* at the springing. The accompanying change in span, if one pier rotates about its base and the other is stationary, produces a stress of 87 lb. per sq. in.* In this case all piers tipped in the same direction and there was probably little or no change in span.

Readings were taken on some of the piers with a surveyor's level to determine the settlement. A complete set of readings was not obtained but the following fragmentary reports are of interest. The north abutment settled 0.04 in. between Feb. 18 and Feb. 28, 1926, the ribs of the adjacent spans being poured in the interim. Pier 1 settled 0.0625 in. between Dec. 19 and Dec. 28, 1925, the rib of an adjacent span being poured in the interim. Both the pier and the abutment on which level readings were taken rested on piles, and in no case was the settlement excessive.

*See note bottom of page 8.

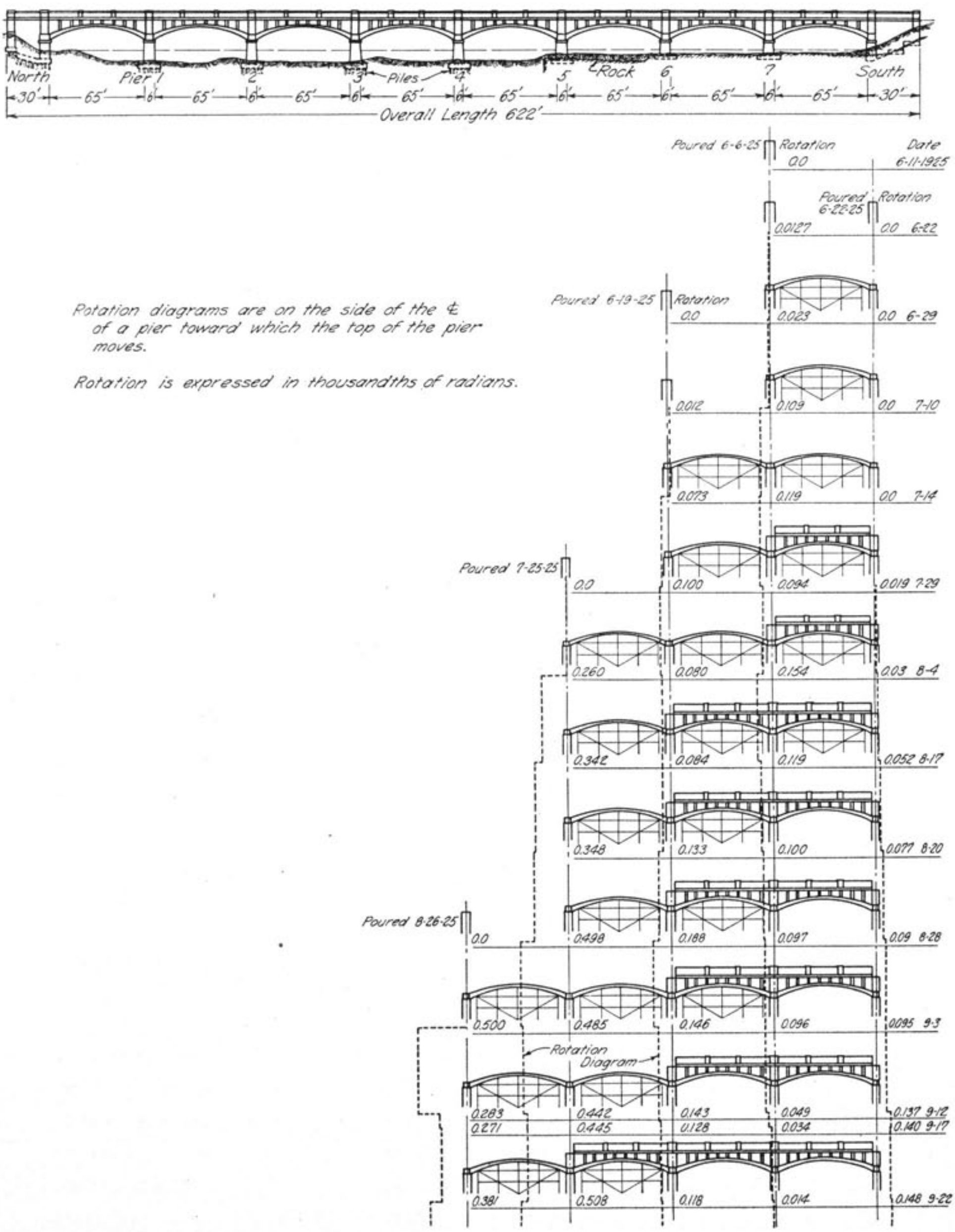

Fig. 6. Rotation of Piers During Construction, Washington Street Bridge, Rockton, Illinois

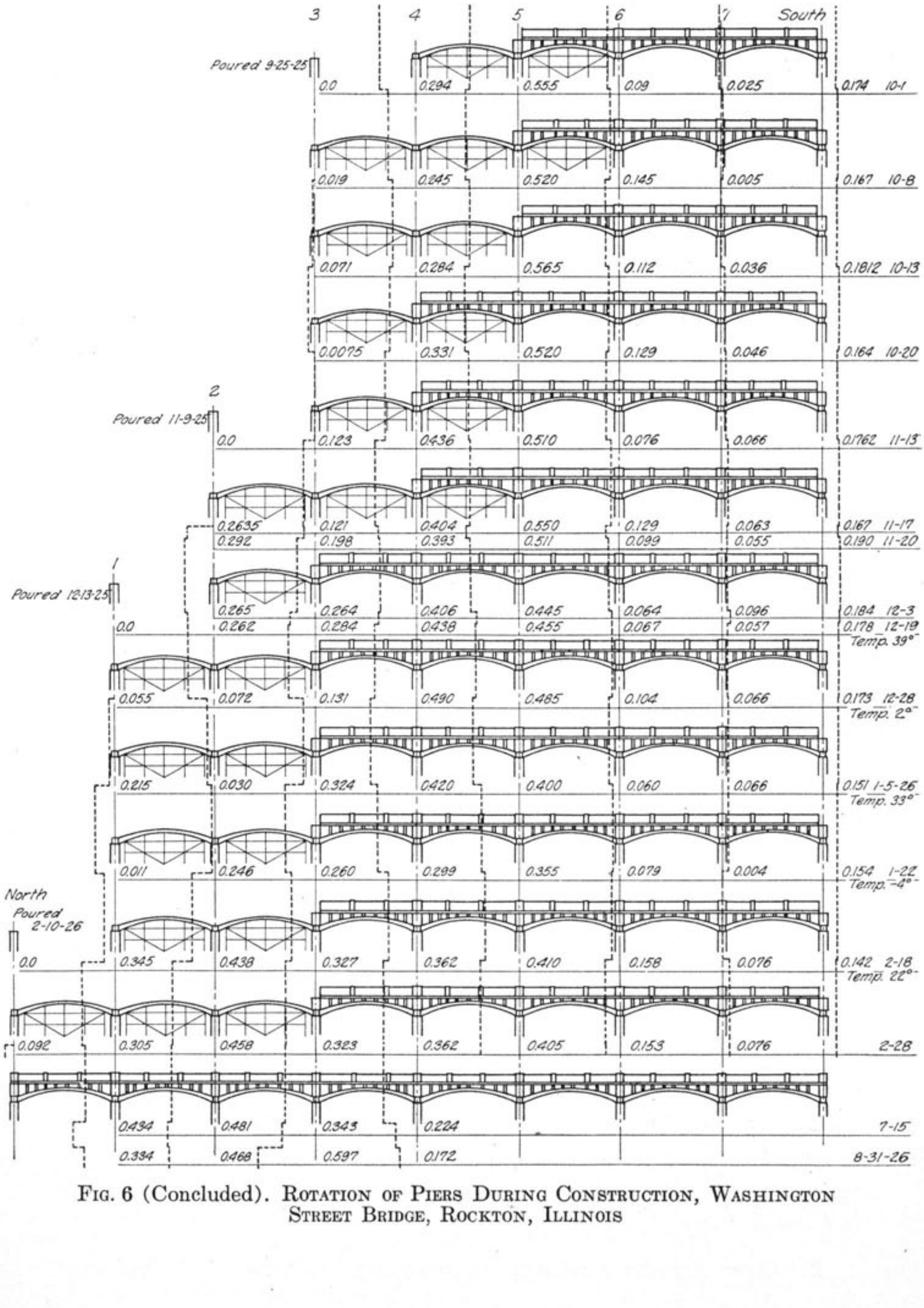

FIG. 6 (Concluded). ROTATION OF PIERS DURING CONSTRUCTION, WASHINGTON STREET BRIDGE, ROCKTON, ILLINOIS

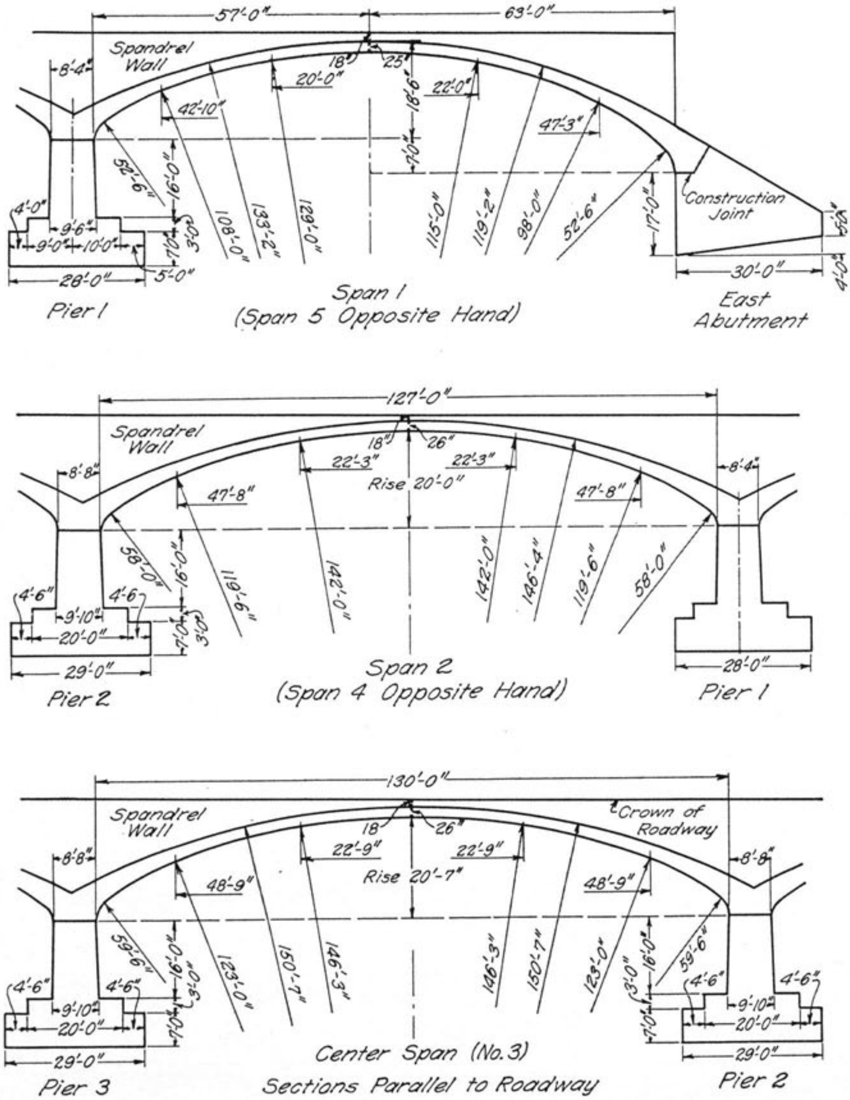

FIG. 7. GENERAL DIMENSIONS, OLIVER AVENUE BRIDGE, INDIANAPOLIS, INDIANA

Changes in span were measured with a piano wire subjected to a constant tension. The results of these observations are given in Table 1. Here, again, the record is far from being complete, but the observations indicate that there was no serious change in span even for the arches carried on piles.

5. *Oliver Avenue Bridge at Indianapolis, Indiana.*—The highway

bridge on Oliver Avenue, Indianapolis, Indiana, shown in Fig. 7, consists of five spandrel-filled, reinforced concrete, skew barrel arches. The span and rise vary from 120 ft. and 22 ft. for the end arches to 130 ft. and 20 ft. 7 in. for the middle one. The arches carry a 60-ft. roadway and two 10-ft. sidewalks, the barrels of the arches being approximately 60 ft. long. The piers make an angle of 83 deg. 4 min. with the centerline of the roadway. The piers and abutments are supported on wood piles driven into gravel, there being 168 piles under each pier, and 189 under each abutment. The bottoms of the piers are 26 ft. below the springing of the arch.

Observations were made during the construction of the arches to determine the rotation and the horizontal and vertical displacement of the piers. The rotation was measured with a level bar. The horizontal movement was measured with a transit, using a target on the pier, one on a monument set in the ground near the pier, and a back sight embedded in the bank of the stream a considerable distance from the monument.

The rotation of the piers is recorded in Fig. 8 and the horizontal translation in Fig. 9. The horizontal movement of the two ends of a pier are recorded separately, the movement of the north end being represented by a broken and that of the south end by a dotted line. The horizontal distance between the two lines represents the movement of one end relative to the other, and this relative movement divided by the length of the pier (about 60 feet) is the rotation of the pier about a vertical axis.

The maximum rotation about a horizontal axis occurred in the case of pier 4 and was equal to about 0.00088 radian. The maximum rotation for pier 3 was just a trifle less, and the rotation for the east abutment was 0.00075 radian. The rotation for piers 1 and 2 was much less. A rotation of one end of the barrel of span 4, the span connecting piers 3 and 4, of 0.00088 radian, unaccompanied by a rotation of the other end or by a change in span, with $E = 1\ 000\ 000$ lb. per sq. in., produces a stress of 42 lb. per sq. in.* at the springing.

The maximum horizontal movement of a pier, shown in Fig. 9, occurred in the case of pier 2 and was 0.21 in. The east abutment had very little movement. A change in span of 0.21 in. unaccompanied by a rotation of the ends of the arch, with $E = 1\ 000\ 000$ lb. per sq. in., produces a unit stress of 45 lb. per sq. in.* at the springing and 62 lb. per sq. in. at the crown.

Unfortunately the targets were disturbed as the filling progressed so that the full effect of placing the fill and removing the shores could

*See note bottom of page 8.

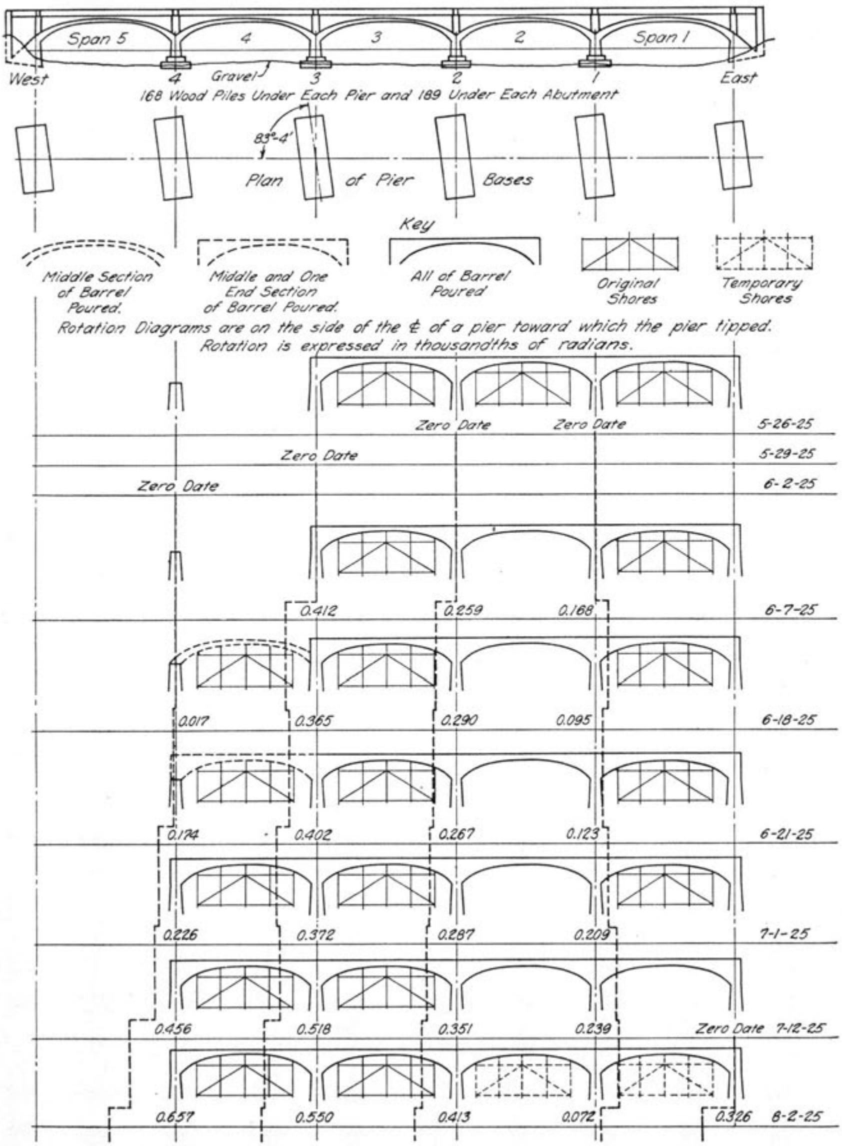

FIG. 8. ROTATION OF PIERS DURING CONSTRUCTION, OLIVER AVENUE BRIDGE, INDIANAPOLIS, INDIANA

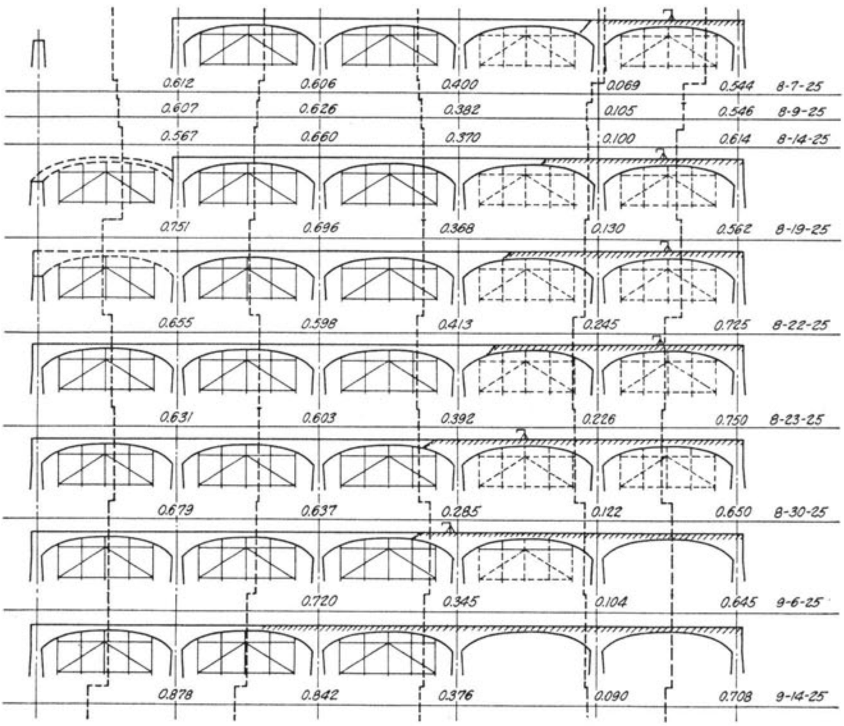

FIG. 8 (Concluded). ROTATION OF PIERS DURING CONSTRUCTION, OLIVER AVENUE BRIDGE, INDIANAPOLIS, INDIANA

not be ascertained. The temporary shores were left in place until the fill on the two spans adjacent to a pier had been placed; so there is no reason to suppose that movements particularly greater than the ones observed occurred as the fill progressed and the shores were removed.

The vertical movement of the piers reported is given in Table 2.

6. *Kentucky Avenue Bridge at Indianapolis, Indiana.*—The highway bridge on Kentucky Avenue, Indianapolis, Indiana, shown in Fig. 10, consists of seven spandrel-filled, reinforced concrete, skew barrel arches. The span and rise vary from 122 ft. and 22 ft. 7½ in. for the end arches to 138 ft. and 22 ft. for the middle one. The arches carry a 60-ft. roadway and two 10-ft. sidewalks, the barrels of the arches being approximately 60 ft. long normal to the roadway. The piers make an angle of 43 deg. 8 min. with the centerline of the bridge. The piers and abutments are supported on wood piles driven into

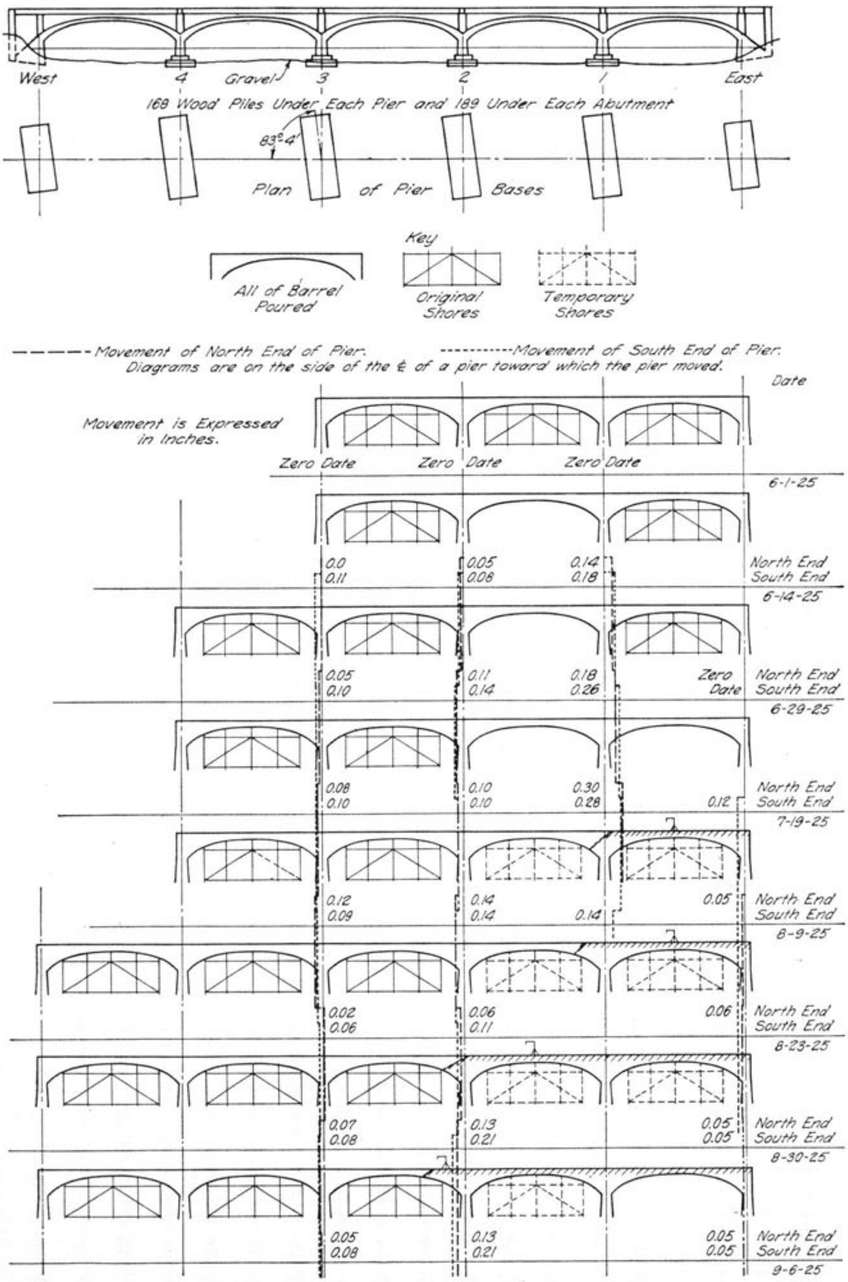

Fig. 9. Horizontal Movement of Piers During Construction, Oliver Avenue Bridge, Indianapolis, Indiana

TABLE 2
CHANGE IN ELEVATION OF TOPS OF PIERS, OLIVER AVENUE BRIDGE, INDIANAPOLIS, INDIANA

Pier No.	Date	Temperature, deg. F.	Change in Elevation of Top of Pier, in Inches	
			North End	South End
1	6-30-25	78	0.00	0.00
	7-12-25	90	−0.07	+0.22
	8- 8-25	88	−0.05	+0.22
	8-22-25	88	−0.04	+0.06
	8-30-25	92	+0.14	+0.19
	9- 6-25	92	+0.06	+0.06
2	6-30-25	78	0.00	0.00
	7-12-25	90	+0.05	+0.11
	8- 8-25	88	+0.12	+0.19
	8-22-25	88	+0.13	+0.14
	8-30-25	92	+0.14	+0.25
	9- 6-25	92	−0.13	−0.06
3	6-30-25	78	0.00	0.00
	7-12-25	90	+0.24	+0.30
	8- 8-25	88	−0.30	−0.13
	8-22-25	88	+0.17	+0.22
	8-30-25	92	+0.42	+0.41
	9- 6-25	92	+0.07	+0.01
4	6-30-25	78	0.00	0.00
	7-12-25	90	+0.46	+0.52
	8- 8-25	88	+0.44	+0.64
	8-22-25	88	+0.44	+0.52
	8-30-25	92	+0.78	+0.95
	9- 6-25	92	+0.32	......

gravel, there being 186 piles under each pier, and 244 under each abutment. The bottom of the piers is 26 ft. below the springing of the arch.

Observations were made to determine the rotation and the horizontal and vertical displacement of the piers. The rotation of the piers is recorded in Fig. 11 and the horizontal movement in Fig. 12.

The maximum rotation about a horizontal axis occurred in the case of pier 3, and was equal to 0.00109 radian. Most of this rotation occurred just after the adjacent span was poured. A rotation of one end of the barrel of span 3, one of the spans adjacent to pier 3, of 0.00109 radian when the other end is stationary and when the span remains constant, with $E = 1\ 000\ 000$ lb. per sq. in., produces a stress of 57* lb. per sq. in. at the springing. The comparatively small rotation of the two abutments is of interest especially as the observations were continued for more than a month after the fill in the adjacent spans had been completed and the temporary shores had been removed.

The horizontal movement of the two ends of the piers is given in Fig. 12. The maximum movement occurred at the north end of pier

*See note bottom of page 8.

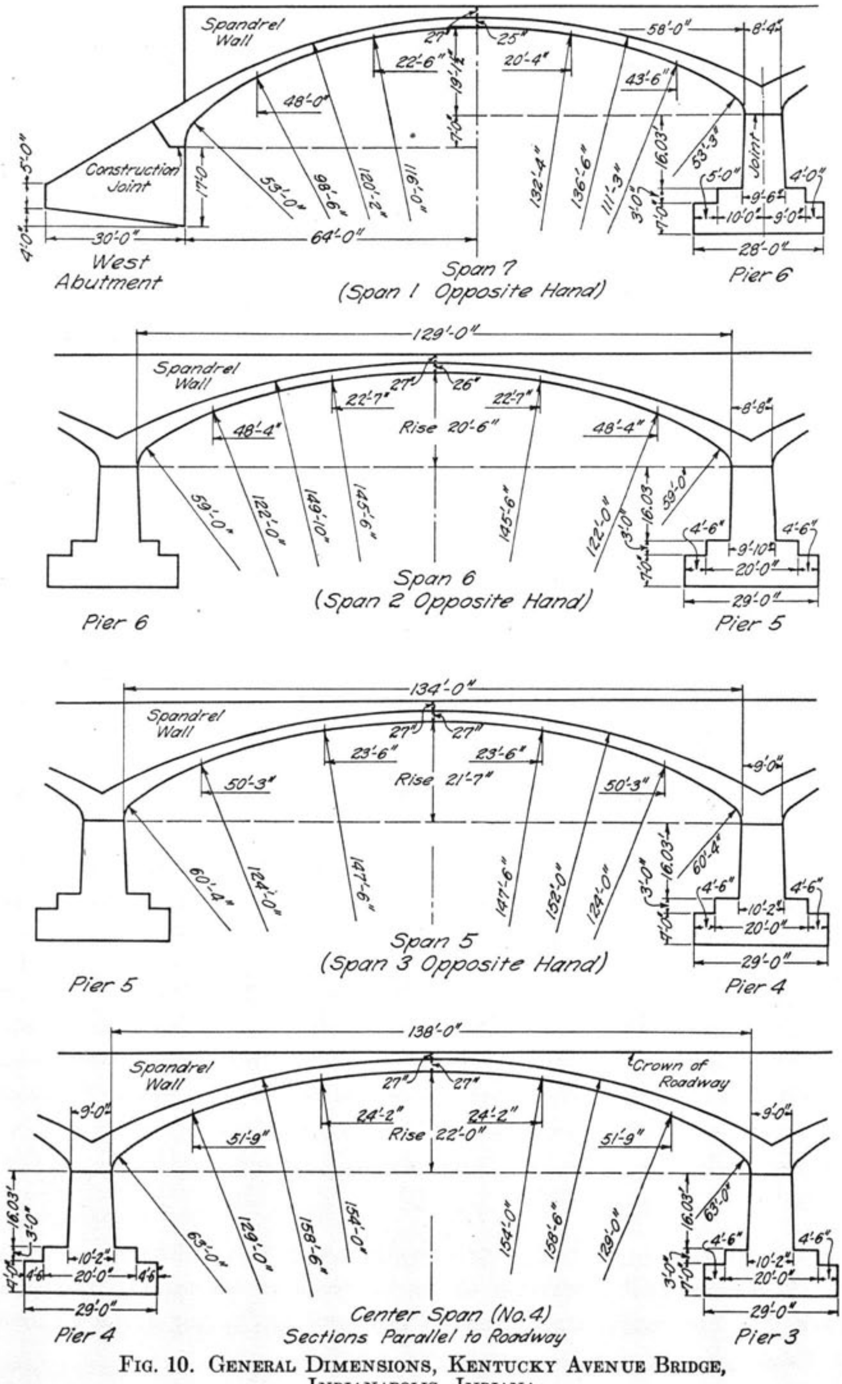

FIG. 10. GENERAL DIMENSIONS, KENTUCKY AVENUE BRIDGE, INDIANAPOLIS, INDIANA

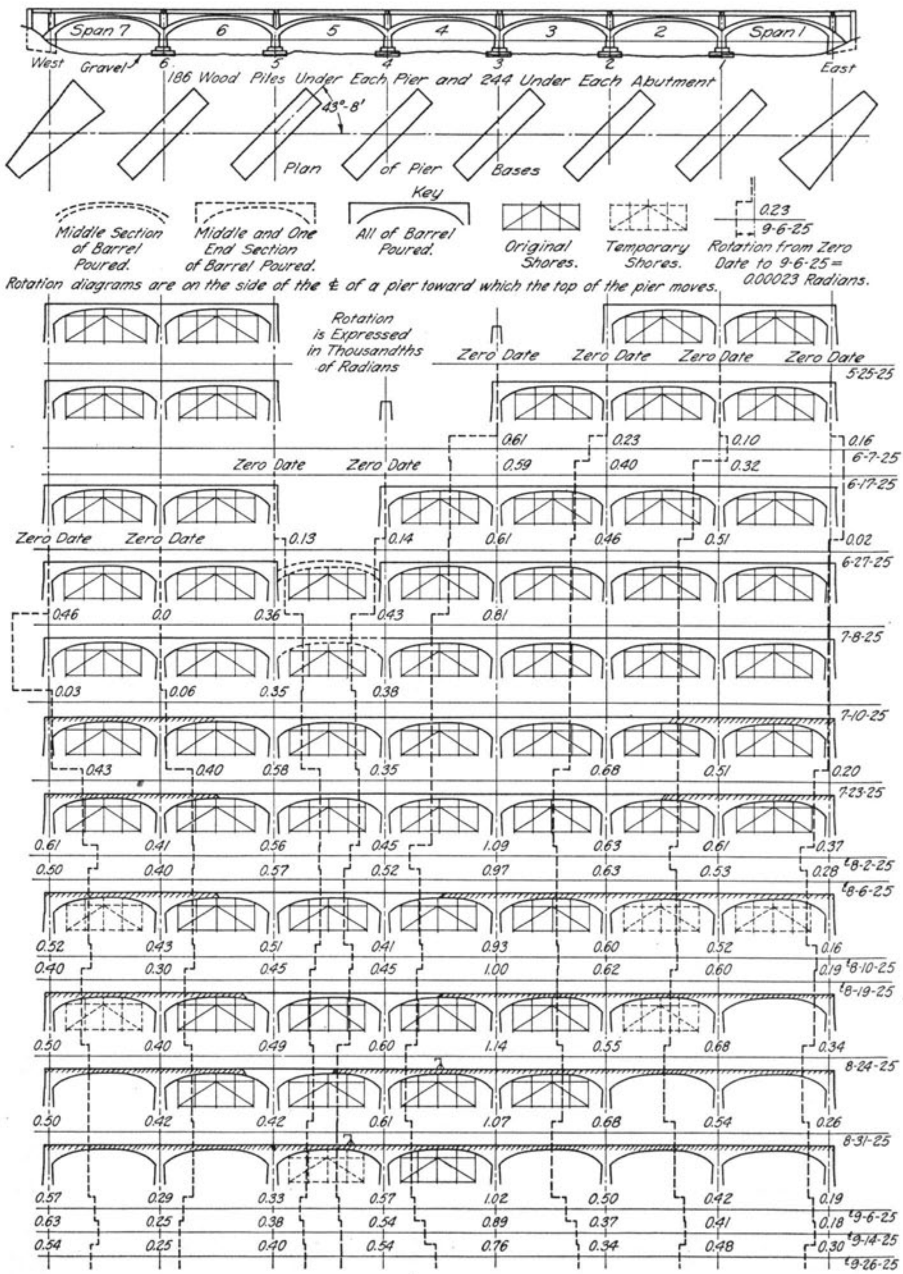

Fig. 11. Rotation of Piers During Construction, Kentucky Avenue Bridge, Indianapolis, Indiana

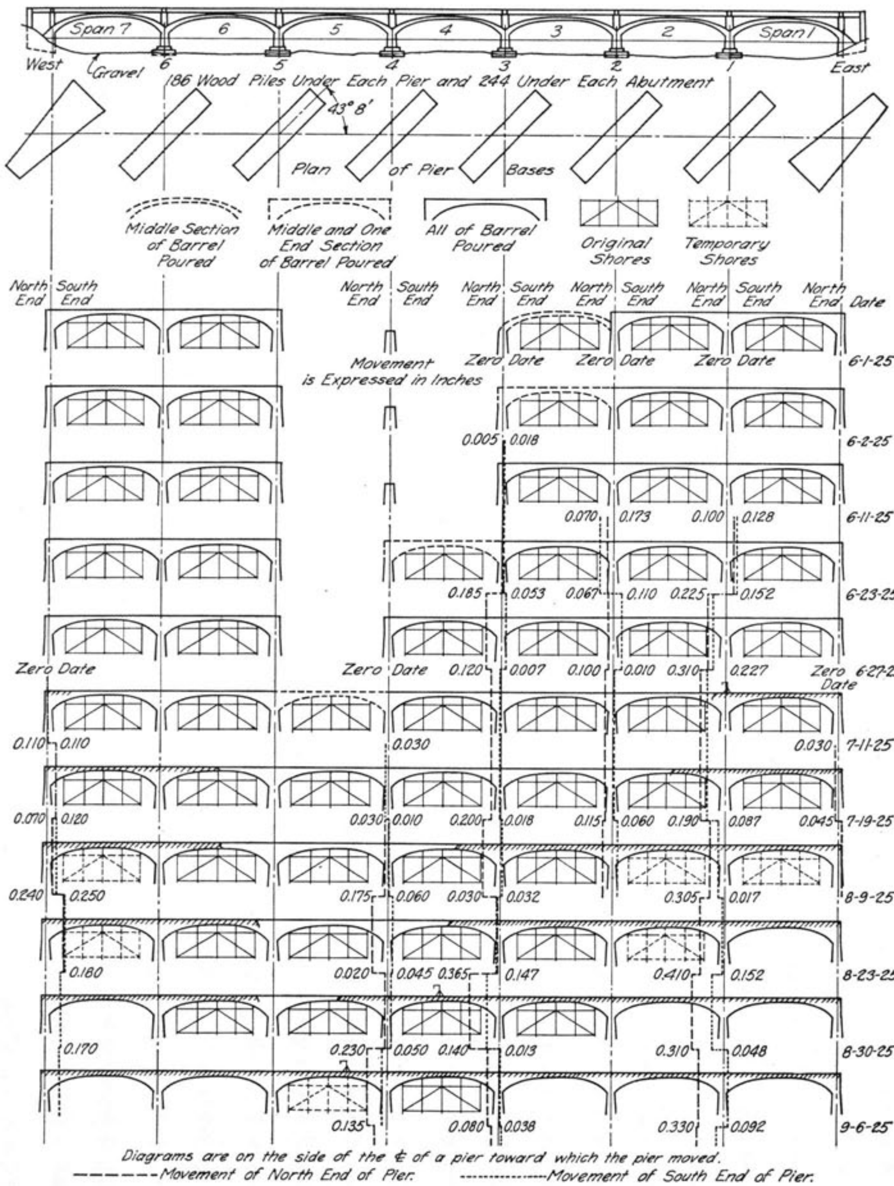

FIG. 12. HORIZONTAL MOVEMENT OF PIERS DURING CONSTRUCTION, KENTUCKY AVENUE BRIDGE, INDIANAPOLIS, INDIANA

TABLE 3

CHANGE IN ELEVATION OF TOPS OF PIERS, KENTUCKY AVENUE BRIDGE, INDIANAPOLIS, INDIANA

Date	Temperature, deg. F.	Change in Elevation of Top of Pier in Inches	
		North End	South End
Pier 1			
6-23-25	..	0.00	0.00
6-28-25	74	−0.10	+1.55*
7-12-25	96	+0.12	+0.14
7-19-25	88	+0.08	+0.16
8- 8-25	80	+0.11	+0.10
8-15-25	86	+0.07	+0.10
8-22-25	88	−0.05	+0.10
8-30-25	80	+0.04	+0.14
9- 6-25	95	+0.02	+0.08
9-19-25	92	+0.16	+0.17
9-26-25	..	+0.14	+0.19
Pier 2			
6-23-25	..	0.00	0.00
6-28-25	74	−0.22	+0.60
7-12-25	96	−0.01	+0.40
7-19-25	88	−0.10	+0.43
8- 8-25	80	−0.08	+0.29
8-15-25	86	−0.10	+0.23
8-22-25	88	−0.17	+0.12
8-30-25	80	−0.23	+0.06
9- 6-25	95	−0.16	+0.20
9-19-25	92	......	......
9-26-25	..	−0.10	+0.30
Pier 3			
6-23-25	..	0.00	0.00
6-28-25	74	−0.49	+0.64
7-12-25	96	−0.30	+0.42
7-19-25	88	......	+0.41
8- 8-25	80	−0.52	+0.32
8-15-25	86	−0.64	+0.19
8-22-25	88	−0.32	+0.19
8-30-25	80	−0.32	+0.08
9- 6-25	95	−0.62	+0.17
9-19-25	92	......	......
9-26-25	..	......	+0.25
Pier 4			
6-23-25	..	0.00	0.00
6-28-25	74	−0.86	−0.61
7-12-25	96	−0.56	−0.83
7-19-25	88	−0.47	......
8- 8-25	80	−0.48	−0.91
8-15-25	86	−0.44	−0.87
8-22-25	88	−0.59	−0.87
8-30-25	80	−0.48	−0.89
9- 6-25	95	−0.92	−1.03
9-19-25	92	−0.90	−0.98
9-26-25	..	−0.84	−1.07

*This is the value reported but apparently it is in error.

1, and was equal to 0.41 in. A change in span of 0.41 in., there being no rotation of the abutments, with $E = 1\ 000\ 000$ lb. per sq. in., would produce a unit stress of 79 lb. per sq. in.* at the springing, and 107 lb. per sq. in. at the crown.

The unbalanced horizontal thrust at the top of a pier supporting two adjacent skew barrel arches forms a couple that tends to rotate the pier about a vertical axis, the indicated rotation for the piers of the Kentucky Avenue bridge being anti-clockwise as seen in plan. For the observed data to indicate an anti-clockwise rotation, the broken line, Fig. 12, should be to the left of the dotted line. The diagrams are consistent with this requirement in all cases except that for pier 2 on June 11, 1925. The magnitude of the rotation, however, is very small, the greatest rotation occurring for pier 1 on August 30, 1925. On this date the north end of the pier had moved west 0.31 in. and the south end east 0.05 in., making a difference of 0.36 in. in a radius of approximately 1000 in. The corresponding angle is 0.00036 radian.

The vertical movement of the piers reported is given in Table 3.

III. Conclusions

7. *Summary of Conclusions.*—The movement of the piers, although small, produced an appreciable strain in the concrete. The strain often occurred when the concrete was green and did not occur instantaneously, but over a considerable period of time so that, although the modulus of elasticity is not known, it was probably small, and may not have exceeded the 1 000 000 lb. per. sq. in. used in the text in converting stress into strain. Even with a value of E equal to 1 000 000 the stress corresponding to the unit strain* due to the pier movements is appreciable.

The movements reported were so small that they could not be detected without the use of precision instruments, yet the stress resulting from these movements is considerable. This is merely another way of saying that in arch construction caution should be exercised to prevent the piers from moving due to unbalanced thrusts during construction processes. It would not appear too great a precaution to require that the piers be kept under observation with precision instruments during construction in order that any motion of the piers might be detected and steps taken to prevent its continuation.

*See note bottom of page 8.

On the whole, the results of the observations made on these bridges are reassuring. They indicate that pier movements during construction will not be excessive if proper precautions are taken. The stability of piers on piles driven in gravel is of particular interest.

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