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The Yadkin River Bridge

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VOL. 9, NO. 10 — **DECEMBER, 1928** — **R. E. ROYALL, Editor**

TABLE OF CONTENTS

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LOADING TESTS ON A REINFORCED CONCRETE ARCH

REPORT ON TESTS MADE ON YADKIN RIVER BRIDGE IN NORTH CAROLINA

Reported by ALBIN L. GEMENY, Senior Structural Engineer, Bureau of Public Roads, and W. F. HUNTER, Designing Bridge Engineer, North Carolina Highway Commission

BEFORE describing the details of these particular tests it is desired to discuss briefly some of the assumptions which are made in arch-bridge design.

The hingeless, reinforced-concrete arch rib is a statically indeterminate structure which can be analyzed only by consideration of the elastic properties of the concrete and steel of which it is constructed. In applying the theory of elastic structures to the analysis of an arch rib, it is assumed that the modulus of elasticity of the concrete is constant for all parts of the arch and at all intensities of stress up to the working stress for which the arch is designed. It is further assumed that a plane section of the rib remains plane after the rib has been deformed.

In open-spandrel arch construction, the floor system is supported by the spandrel columns through which the loads are transmitted to the ribs. In current design practice it is assumed that the loads are distributed only to adjacent panel points and are applied as vertical forces at the points of the rib at which the columns are attached, although it is apparent that a continuous floor system distributes the loads to panels beyond those in which they are applied, thus rendering indeterminate the distribution of the load to the rib. The floor system may be continuous or it may be broken by expansion joints at one or more points. It may be rigidly attached to the tops of the spandrel columns or it may have movable bearings on the columns. In practically all cases the columns are integral with the rib.

In designing, the effect of the superstructure on the deformation of the rib is generally neglected even though it is obvious that this effect may be of considerable importance. The degree to which the rib deformation is modified by the superstructure depends upon the number of breaks in the continuity of the floor system, the method of connecting it to the columns, and upon the stiffness of the columns. In the case of an arch with the floor system continuous over the whole span and rigidly attached to the tops of the columns, we have, in fact, a fixed, spandrel braced arch in which the diagonals are omitted and their functions performed by the rigid joints at the ends of the columns. In the case of an arch with expansion joints at each panel point and with the floor system supported on expansion bearings, the condition would approach those assumed in designing. Usually the conditions lie somewhere between these two extreme cases. The free rib is three times indeterminate and the complete arch, in the present case, thirty-nine times indeterminate.

STATEMENT BY THE ADVISORY COMMITTEE [1]

THE NORTH CAROLINA STATE HIGHWAY COMMISSION built in 1922, as a Federal-aid project, a 3-span concrete arch bridge over the Yadkin River, also known as the Pee Dee River, between Albemarle and Mount Gilead. In 1926 the Carolina Power & Light Co. began the construction of a dam on a site about 9 miles downstream from the bridge. The water of the river, upon the closing of the dam, was to be backed up to such a height as to submerge the bridge and necessitate its replacement by a new bridge at a higher elevation. Between the time of completion of the new bridge and the closing of the dam, a period of several months, the old bridge was to be demolished so as to offer no obstruction to the flow of water in the river.

These circumstances presented a unique opportunity to test a modern, full-size, reinforced concrete arch bridge with moderately long spans. In recent years the popularity of the arch bridge has increased greatly because of its superior esthetic value and, in this country, millions of dollars are spent annually on this type of bridge alone. Consequently, there is a widespread tendency on the part of bridge engineers to embrace any idea which may lead to more economical or more satisfactory arch design without sacrificing safety. In departing from current practice, the judgment of the engineer is based more and more on data developed by the various research agencies of the world.

The North Carolina State Highway Commission, recognizing the opportunity to make this test, and desiring to make it as complete as the available time and money permitted, requested the cooperation of the Bureau of Public Roads.

The bureau acceded to the State's request, and the two agencies then jointly issued to various technical and scientific societies and colleges invitations to participate in the experiment by appointing one or more of their members to serve on an advisory committee. [1] The purpose of this committee was to formulate general plans for the test, and, by meeting from time to time, assist those in active charge in the solution of problems which would certainly be encountered during the period of the test, and assist in interpreting the results. The advisory committee first met in April, 1927, and formulated general plans; several meetings were held during the course of the test, and at a final meeting on November 8, 1928, this report was approved by the committee. Acknowledgments by the committee are given below. [2]

[1] Membership of the advisory committee formed as a result of invitations issued by the Bureau of Public Roads and the North Carolina Highway Commission was as follows: University of North Carolina represented by Dean G. M. Braune; North Carolina State College represented by Dean W. C. Riddick; American Association of State Highway Officials represented by Searcy B. Slack, bridge engineer of the Georgia State Highway Board; American Society of Civil Engineers represented by Prof. Clyde T. Morris of Ohio State University; American Railway Engineering Association represented by J. B. Hunley, engineer of structures of Cleveland, Cincinnati, Chicago & St. Louis Ry. Co.; American Concrete Institute represented by A. B. Cohen, consulting engineer, New York, N. Y.; Highway Research Board represented by A. T. Goldbeck, director of the bureau of engineering, National Crushed Stone Association; U. S. Bureau of Standards represented by D. E. Parsons, associate engineer; American Society for Testing Materials represented by F. E. Schmitt, editor, Engineering News-Record; U. S. Bureau of Public Roads represented by E. F. Kelley (chairman), chief, division of tests; O. L. Grover, principal bridge engineer; H. M. Westergaard, professor of theoretical and applied mechanics, University of Illinois; and L. W. Teller, senior engineer of tests; North Carolina State Highway Commission represented by L. R. Ames, State highway engineer; Wm. L. Craven, bridge engineer; M. M. Trumbull, assistant bridge engineer; and E. H. Kivett, engineer of tests.

[2] The instruments and scientific apparatus used in this test were furnished by the following organizations: The American Society of Civil Engineers and the Bureau of Standards furnished the electric telemeters. The Bureau of Standards furnished the Berry strain gauges and temperature coils. The Bureau of Public Roads furnished the radiusmeter, weighing cells, thermometers and deflection wires. The committee on concrete and reinforced concrete of the American Society of Civil Engineers furnished the clinometers.

The installation of instruments and making of field observations were under the direction of G. W. Davis of the Bureau of Public Roads, assisted by W. F. Hunter and W. M. Price of the North Carolina Highway Commission and Albin L. Gemeny and E. C. Sutherland of the Bureau of Public Roads. The electric telemeters were calibrated and installed by O. S. Peters of the Bureau of Standards. All computations were made by W. F. Hunter and Albin L. Gemeny. The preliminary model analysis was made by D. H. Overman of the Ohio State Highway Department under the direction of Prof. Clyde T. Morris of Ohio State University. The brass wire model analysis was made by G. W. Davis and E. C. Sutherland. The final model analysis was made by Prof. J. T. Thompson of Johns Hopkins University and Albin L. Gemeny, using a model constructed by the Bureau of Public Roads. The bridge maintenance department of the North Carolina Highway Commission made available one of its forces during the entire period of the test to do all construction work and operate the ferry. The foreman of this force was J. P. Beach under the general direction of C. B. Taylor. Success in the prosecution of the test was due in large measure to the enthusiastic cooperation of the North Carolina Highway Commission through Messrs. Craven, Trumbull, and Hunter of the bridge department.

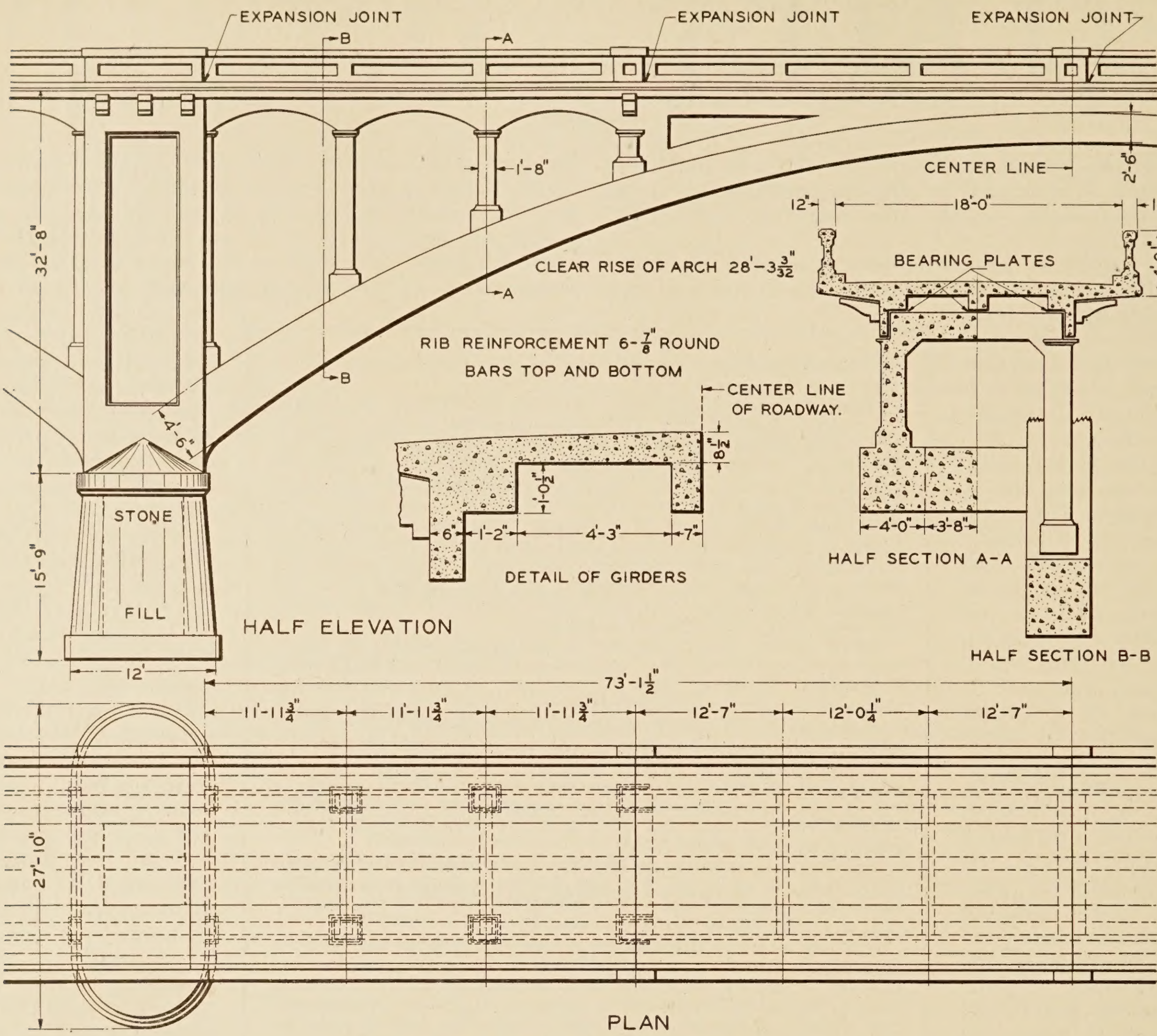


Fig. 1.—Details of Test Span

A method of analyzing indeterminate structures, developed comparatively recently by Professor Beggs, of Princeton University, consists in studying the elastic action of a model, each member of which is of the same relative stiffness as the corresponding member of the structure. By the application of the Maxwell theorem of reciprocal deflections, and Müller Breslau's principle that any influence line is a deflection diagram, the moment, thrust, and shear at any section may be found and the stresses computed.

OBJECTS OF THE TESTS OUTLINED

The North Carolina bridge tests were conducted for the following specific purposes:

(1) To compare the measured deformations of a full-size, reinforced concrete arch rib with the deformations as determined by the theory of elastic structures, when the rib carried loads producing stresses of moderate intensities, and was as free as practicable from the restraining action of the superstructure.

(2) To make the same comparisons when the rib carried loads which produced stresses of high intensities.

(3) To determine the effect of the superstructure on rib deformations by comparing deformations measured when the superstructure was intact and the measured and computed deformations of the rib free from restraint by the superstructure.[3]

(4) To compare the measured deformations of the rib, both with and without the restraining action of the superstructure, and the deformations as determined from an analysis made by the use of an elastic model.

TEST BRIDGE DESCRIBED

The test bridge consisted of three 2-rib open-spandrel arch spans of 146 feet 3 inches clear span and 28 feet 3 inches rise with seven 42-foot 6-inch deck-girder approach spans at each end. The floor system of the arches rested on sliding bearing plates at each panel point. These plates were found to be badly corroded and probably had ceased to function freely as sliding bearings. The intermediate arch piers below the springing line were of hollow construction, the hollow space being filled with field stones. The end arch piers had buttresses on the shore side to increase their resistance to overturning under the unbalanced thrust. The piers were founded on solid rock. Details of the test span are shown in Figure 1.

The bridge was built by contract, using cement and reinforcing steel furnished by the State highway commission. The coarse aggregate consisted of crushed field stones found in the vicinity of the bridge site. Inspection of the aggregate in cores taken from the

[3] Further references will be made simply to the "free rib."

bridge disclosed several varieties of stone, all of which were apparently hard, sound, and durable. The fine aggregate consisted of a mixture of 5 parts of sand to 1 of stone screenings. A well-known brand of Portland cement was used.

The concrete for the piers below the springing line was mixed in the proportions 1:2½:5 and, for the remainder of the bridge, in the proportions 1:2:4. Tests of 6 by 12 inch cylinders of the 1:2:4 concrete made at 28 days showed strengths of 2,140, 1,900, 1,655, and 1,258 pounds per square inch. Each of these strengths is for a single cylinder representing about 50 cubic yards of rib concrete and are arranged in the order of location of the batch from springing line to crown. Inspection records do not show clearly which of the arch spans is represented by these cylinders.

The reinforcing steel consisted principally of round, deformed bars of intermediate grade steel. Some of the minor reinforcing consisted of square, deformed bars. Tension tests on the steel showed an average yield point of about 48,000 pounds per square inch and an average ultimate strength of 75,300 pounds per square inch.

Cuts made in the concrete for installing instruments, taking test specimens and destroying the continuity of the superstructure showed dense, hard concrete apparently of good quality. The steel, where the covering of concrete was stripped off, showed clean surfaces, free from all signs of corrosion.

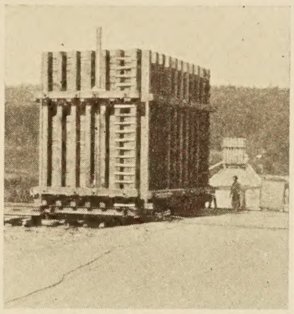
PRELIMINARY EXPERIMENTS MADE

Various possible methods of measuring vertical deflections of the rib and horizontal movements of piers were considered and it was decided to use suspended wires. The wires for measurement of pier movements were to be fixed at the far piers of the adjacent spans on the assumption that temperature movements of any point on the wire would be vertical, the position of such point horizontally remaining fixed. In order to test this assumption, a wire was stretched between two firmly planted posts at the Arlington Experiment Farm and observations made of the movements of a number of points fixed on the wire over a period of time during which there was a considerable change in temperature. It was observed that no appreciable horizontal movements of the points took place.

The deflection wires (described in detail on p. 192) were installed in June, 1927. At the same time thermometers were placed in holes drilled in the ribs at different distances from the surfaces of the concrete. The holes were filled with cup grease and closed with corks through which the stems of the thermometers passed. Temperature movements at the crown of both ribs of the center arch span were observed daily over a period of several months. The observations showed an average movement of one-fortieth of an inch for a change of 1° C. in average rib temperature.

TANKS FILLED WITH WATER USED FOR TEST LOADS

The test span was loaded with tanks of water, filled by pumping from the river. The tanks were 12 feet 6 inches wide by 20 feet long and 18 feet high, inside dimensions, and were built of timber with structural steel underframes. The length was such as to permit supporting the load at two adjacent panel points. Rollers were provided so that the empty tanks could be easily moved into any desired position on the bridge. After being placed in position, the tanks were jacked up and allowed to rest symmetrically on four bearing blocks located over the center of the columns at which the loads were to be applied, as shown in Figure 2, page 190. The tanks were leveled by the use of plumb bobs suspended at each end and then filled with water. The tanks were moved while empty to avoid overstraining the floor system.

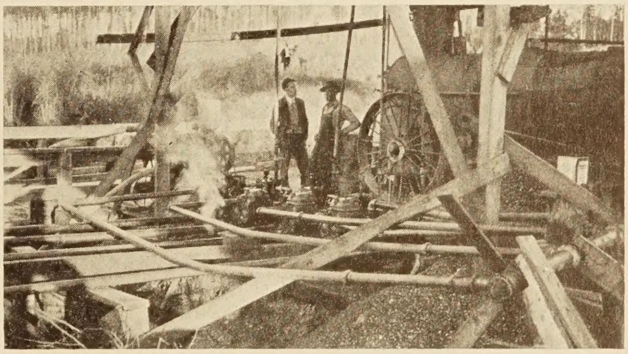


Loading Tank

Force to move the tanks was applied by a truck through a block and tackle arrangement anchored to the solid handrails on each side of the bridge. At ordinary temperatures the tank could be rolled over the rock asphalt surface of the bridge floor but at high temperatures it was necessary to use plank runways to prevent the rollers from sinking into the asphalt.

It was not possible to weigh the tanks by ordinary methods and a special weighing cell was used for the purpose. This device makes use of a small copper cylinder, specially heat-treated and of fixed size, which when compressed under load, is permanently deformed according to a fixed law.

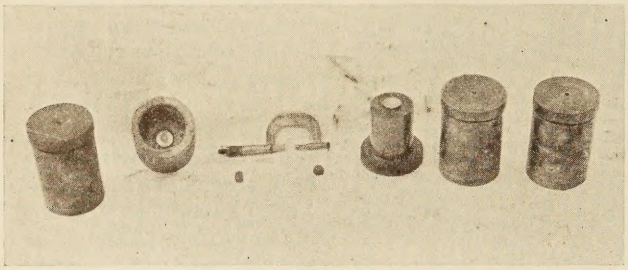
The complete weighing cell consists of a hollow steel cylinder into which a steel piston fits closely. On the inside of the cylinder head is a hardened steel face or anvil with a plane, smooth surface. On the entering end of the piston is a corresponding hardened steel face. The copper cylinder, one-half inch in diameter by one-half inch high, is placed on end in the steel cylinder on the smooth surface and the piston is allowed to rest on it. The load whose magnitude is desired is then applied to the piston and its entire weight is



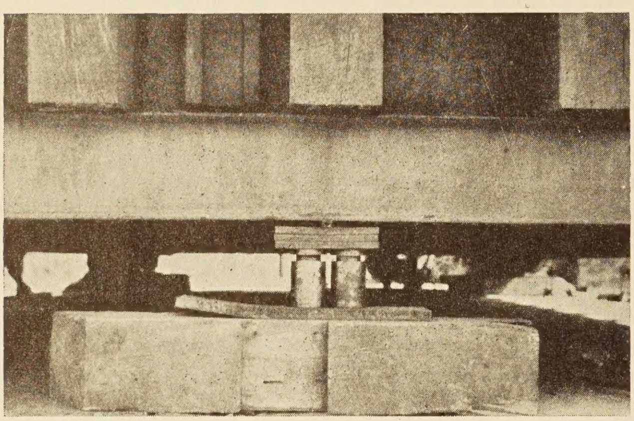
Pumping Plant for Filling Tanks

transmitted to the copper cylinder. The length of the copper cylinder is measured with a micrometer caliper before and after the load is applied. The weight corresponding to the deformation of the cylinder is taken from a calibration curve which has been previously determined in the laboratory for the particular size and quality of copper cylinder used.

The empty tank was weighed by placing a cell under each corner of one end and two cells with an equalizer under the center of the opposite end. The total weight of the tank was the sum of the four weights indicated by the cells. The weight of the tank was also calculated from the unit weights of the timber determined by weighing specimens of the material used in constructing the tanks. The weights determined by the two methods checked within 1 per cent. The weight of each empty tank was found to be approximately 47,000 pounds and this figure was used in the computations. The water capacity of each tank was 4,500 cubic feet, 33,750 gallons, or 281,250 pounds. The increments of water load at each panel point were 22,750, 45,500, and 68,250 pounds.



Details of Weighing Cells



Two Weighing Cells in Place at One End of Tank

TESTS DIVIDED INTO THREE PHASES

The test was divided into three phases which, for convenience, are designated as series 1, 2, and 3.

In series 1 a single tank was placed so as to apply its load at two adjacent panel points and deformations observed over the entire rib from springing line to springing line with the superstructure intact. The series of loadings began at a pier and continued successively to the crown. The tank was also placed on one of the adjacent spans in a panel next to the crown, and deformation readings were taken on the span under observation.

In series 2, the same procedure was followed except that the deck and railings were cut at each panel point and supported on new, greased bearing plates so as to destroy, as far as practicable, the continuity of the floor system and its restraining action at the tops of the columns, and the curtain wall near the crown was broken out. In this series, the load at panel points 1 and 2 (fig. 2) was omitted because of the small deformations caused by the load in this position.

In series 3, two tanks were placed in the position to produce the maximum stress in the rib and the deformations were measured as in series 1 and 2, with the superstructure in the same condition as in series 2.

Four increments of load were applied at each position of loading: The empty tank, the tank filled with 91,000, 182,000, and 273,000 pounds of water.

In destroying the continuity of the superstructure for series 2 and 3 the slab, girders, and handrail over each cross beam were cut through with air drills and the steel severed with an oxyacetylene flame. The ends of the girders were then jacked up and new, well-greased bearing plates inserted at each girder bearing. The girder ends on the entire east half of the span were shattered by the cutting operation to such an extent as to make the bearings on the cross beam unsafe. To relieve these bearings of the tank loads, holes were cut through the deck at the panel points and timber bearing blocks placed directly on the cross beams. In two panels it was thought necessary to support the dead weight of the floor system on timbering built up from the ribs. This was done in such a way that the arch was not stiffened.

DEFORMATIONS MEASURED

In order to measure completely the deformation of the rib under live load, the following six measurements were made:

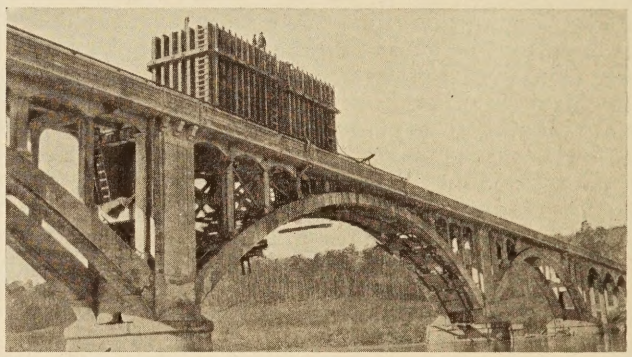
(1) Deformations of the concrete on the extrados and the intrados at nine sections of the rib spaced 18 feet 6 inches apart along the axis.

(2) Deformations of the reinforcing steel near the springing lines and at the crown.

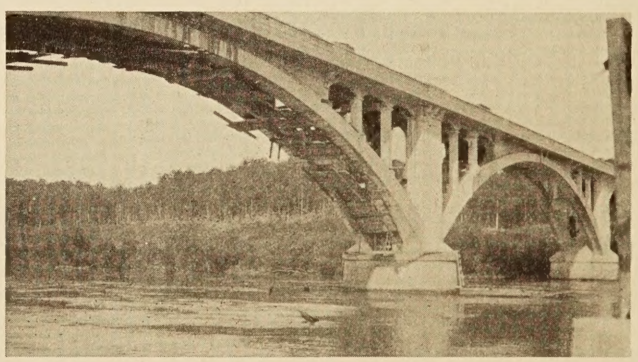
(3) Rotation of the arch axis at nine points spaced 18 feet 6 inches along the axis.

(4) Deflections of the rib at nine columns, no measurements being taken at the column next to each pier.

(5) The change in length of mid-ordinates of each of the 31 consecutive 5-foot arcs of the axis.

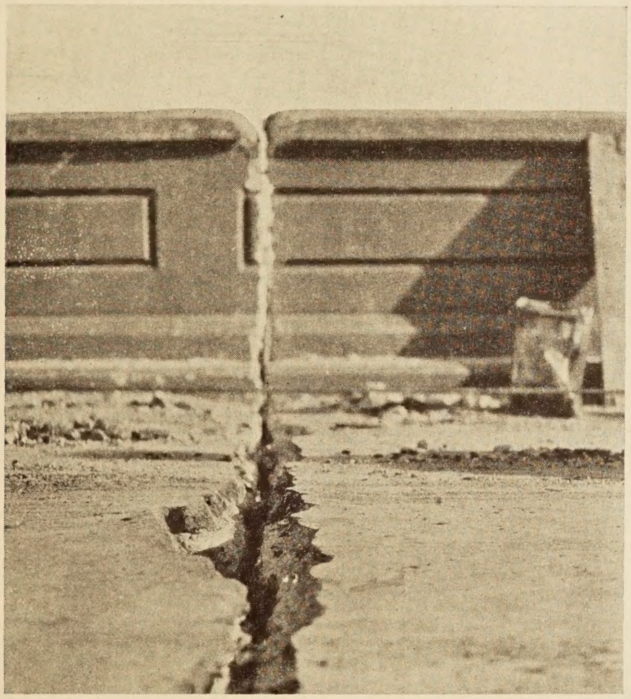


Two Tanks in Position for Series 3



Scaffold From Which Observations Were Made

Cutting Through the Floor System at a Panel Point to Destroy Continuity



Cut Through Handrail, Curb, and Floor

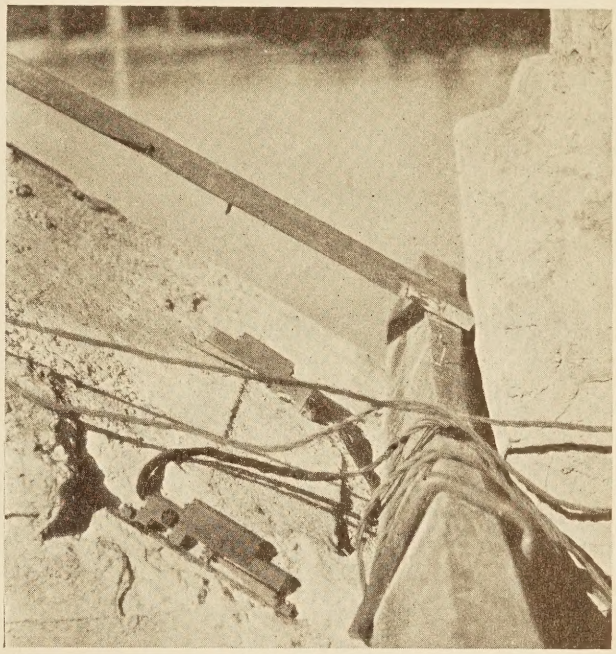
(6) Rotation and horizontal movements of the piers.

The locations of the instruments are shown in Figure 2. It was decided that deformations under load should be measured only on the north rib of the middle arch span. This span, including the piers, was symmetrical; the north rib was somewhat protected from the direct rays of the sun except in the early morning and was, therefore subject to more uniform temperature conditions than the south rib.

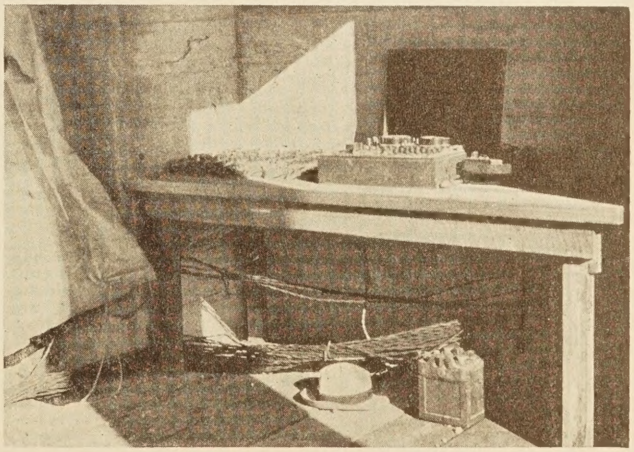
MEASURING INSTRUMENTS DESCRIBED

TELEMETERS USED TO MEASURE DEFORMATIONS OF CONCRETE AND STEEL

Deformations of the concrete at the extrados and intrados and deformations of the steel were measured by means of McCullom-Peters electric telemeters.[4] The electric telemeter consists of a stack of carbon disks held under pressure. A change of length of the stack is accompanied by a change of pressure and electrical resistance, the stack of disks acting as an elastic body. Suitable terminal pieces or mountings for the stack are supplied which can be attached to a structural member at two points spaced some distance apart. Changes in length between these points will change the length of the stack and also the electrical resistance which can readily be measured at any convenient point by running wires from the terminals of the carbon resistor to a suitable measuring instrument.



Telemeters Secured to Concrete and Reinforcing Steel On the Extrados Near the Springing Line



Central Station in Pier Where Telemeter Readings Were Taken

The 2-element type of telemeter with 8-inch gauge length was used in these tests. The terminals were fixed to the concrete by means of screws threading into steel plugs which were grouted in the concrete. Telemeters were placed on the steel by stripping off about a foot of the concrete cover, drilling into the steel and attaching the instrument.

[4] For a complete description of the McCullom-Peters electric telemeter see Technologic Paper No. 247 of the Bureau of Standards entitled "A New Electric Telemeter," by Burton McCullom, and O. S. Peters.

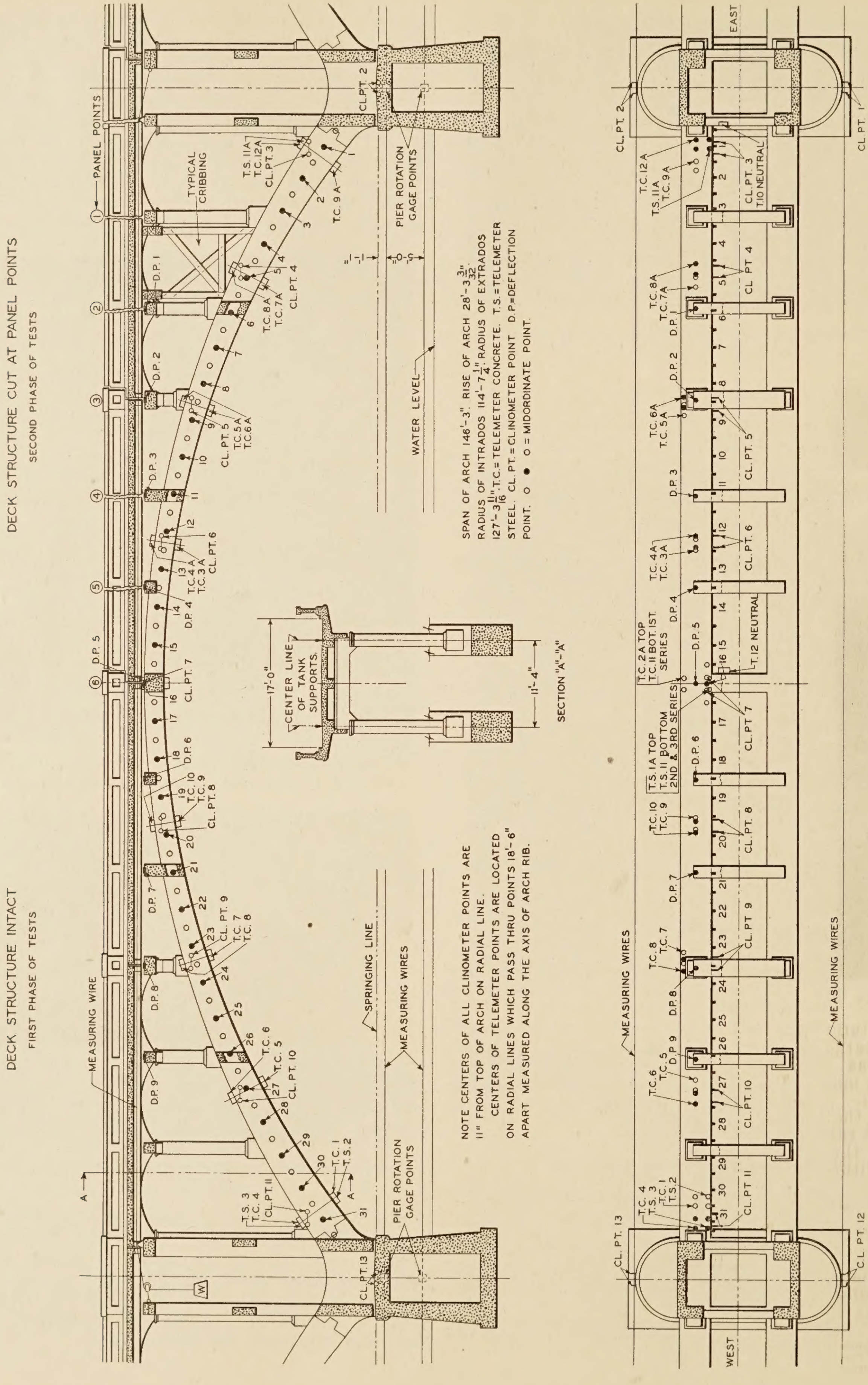


FIG. 2.—LAYOUT OF ARCH RIB SHOWING LOCATION OF INSTRUMENTS

Instruments were placed on the concrete at the intrados and extrados at each of nine normal sections of the rib which were spaced 18 feet 6 inches along the axis and also on the steel in the intrados and extrados near the two springing lines and at the crown. Wires were carried from each of the instruments to a Wheatstone bridge in the east pier of the center span of the arch where the resistances were read. The Wheatstone bridge was energized by a 6-volt battery and the current was kept constant by a variable resistance coil.

It was found necessary to make temperature corrections for the instruments. This was first attempted in series 1 by removing two of the instruments from the steel and placing them on members of the bridge which were not subjected to direct stress, one at the crown and one at the springing line. The readings on these neutral telemeters were used as corrections for the stress-measuring telemeters which were divided into two groups, each having temperature conditions apparently the same as those of one of the neutral telemeters. Recognizing the possibility of inaccuracy in this assumption, it was decided to use a more accurate method in series 2 and 3. A pair of resistance-coil thermometers was placed at each of the telemeters for measuring deformations of concrete, one embedded in the concrete near the instrument and the other fixed to the instrument itself. Wires were carried from the coils to a measuring instrument in the pier where the resistances were read each time a set of readings was taken on the telemeters. The difference in temperature between the instrument and the concrete, multiplied by the thermal coefficient of expansion of the metal of the instrument was applied as a correction to the unit deformation calculated from the direct reading on the telemeter.

The method of making this correction is illustrated by the following example. The telemeter considered is 12A, located on the extrados near the springing line. The empty tank was placed at panel points 2 and 3. It is desired to measure the stress produced by the weight of water in the full tank.

Resistance reading with empty tank = 13.1 ohms, in this case indicating tension.

Tension calibration factor for telemeter 12A = 0.0000222 inch per inch per ohm.

Telemeter reading = $0.0000222 \times 13.1 = 0.0002910$ inch per inch elongation.

The resistance coil thermometers indicated that the temperature of the telemeter was 3° C. lower than that of the concrete thereby increasing the tension reading due to deformation of the concrete.

Correction due to temperature = $0.0000110 \times 3 = 0.0000330$ inch per inch elongation.

Corrected reading = 0.0002580 inch per inch elongation.

This process is repeated with the tank full and the result is 0.0005018 inch per inch elongation.

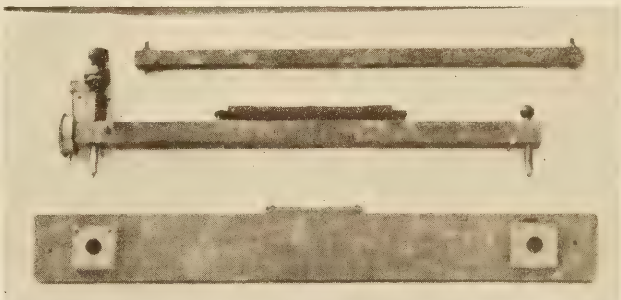
The difference between corrected readings for empty and full tank = $0.0005018 - 0.0002580 = 0.0002438$ inch per inch.

Stress produced by added load = $0.0002438 \times 4{,}000{,}000 = 975.2$ pounds per square inch.

In the above calculation the thermal coefficient of expansion of the metal of the instrument is 0.0000110 per degree centigrade and it is assumed to be approximately the same for the concrete. The same process was used for deformations of the reinforcing steel which was considered as taking the temperature of the surrounding concrete since only a small surface was exposed to the air.

When the resistance coils were installed, two were placed at the neutral telemeter at the crown in order to find the relation between the temperature corrections as determined by the neutral telemeters and by the resistance coils. It would have been desirable to have placed coils at the neutral telemeter at the springing line but none was left for this purpose. It was found that the difference in temperature between the neutral telemeter at the crown and the adjacent concrete changed by about the same amount as in the case of the instruments on the rib, although the actual temperatures varied from point to point of the rib. It was, therefore, decided that the reading on this one neutral telemeter would be a more accurate correction than that which had been determined by grouping the telemeters. All of the stress curves of series 1 are, accordingly, based on telemeter readings corrected in this manner.

When it was found necessary to correct the telemeter readings for temperature it was decided to attempt check measurements with a 20-inch Berry strain gauge. Because of the small strains to be measured and physical obstacles to careful manipulation of the gauge, the results were not satisfactory.



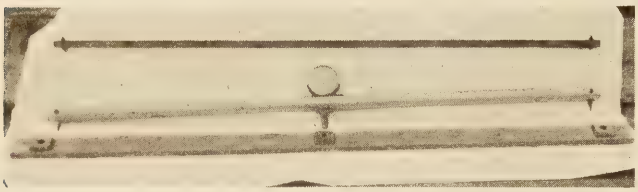
Clinometer for Measuring Rotation of Arch Axis

Taking a Rotation Measurement with a Clinometer

MEASUREMENT OF ROTATION OF ARCH AXIS

The rotations of the arch axis were measured by means of a clinometer or level bar. The clinometer consists of a square bar of steel supported horizontally on two pointed steel legs set vertically and spaced 20 inches apart. One of these legs is fixed while the other is adjustable vertically by a fine screw thread and hand nut. The amount of vertical adjustment necessary to level the bar is indicated by a suitable micrometer dial in contact with the adjustable leg. A sensitive level bubble is placed horizontally on the upper face of the steel bar.

Two steel plugs containing gauge holes were grouted in the inner side of the rib in such a manner that they were 20 inches apart and approximately in the same horizontal plane and the line between the gauge holes was bisected by a plane normal to the axis at the point at which it was desired to measure the rotation. To measure the rotation under any increment of load, the points of the clinometer were placed in the gauge holes of the plugs and the level bubble brought to the center of the bubble tube by use of the adjusting screw and the micrometer dial read. After the load was applied, another reading was taken. The difference in the two readings was the relative vertical movement of the two plugs in thousandths of an inch, and this change in relative elevation divided by the gauge length gave the rotation in radians.



The Radiusmeter

Measuring a Mid-Ordinate with the Radiusmeter

RIB DEFLECTIONS

Deflections of the rib were measured from a wire stretched between the tops of the piers, over the centers of the spandrel columns. This wire was fixed at one end and passed over a pulley at the other end and had attached a weight of about 100 pounds for the purpose of maintaining a constant tension and thus a constant sag in the wire. Small metal plates containing gauge holes were fixed in the tops of the columns directly under the wire. Deflections were measured with a scale graduated to fortieths of an inch and provided with a pointed metal shoe which was placed in the gauge holes in the plates. Measurements were made before loading and after each increment of load.

MEASUREMENT OF MID-ORDINATES OF CHORDS

Changes in length of the mid-ordinates of 5-foot chord lengths of the axis were measured with a radius meter. The radius meter consists of a steel tube to which is attached two hardened steel points set at right angles to the axis of the tube and 5 feet apart. One of these points is rigidly fixed to the tube and the other has a flexible joint permitting a slight variation in the distance between the points. Midway between these points and on the side of the tube opposite to that to which the points are attached a micrometer dial, reading to one ten-thousandth of an inch, is mounted. The dial stem is provided with an extension passing through a clearance hole in the tube. This tube is encased in a larger tube, to which it is attached only at the support points, so that, in manipulating the instrument, no force can be applied to the inner tube which would deflect it and affect the readings on the dial. The readings in series 1 were taken with the stem extension free to bend a small amount laterally. It was found that this lateral movement of the stem extension gave appreciable readings on the dial, and in series 2, a close-fitting sleeve bearing was provided for the extension so as to prevent lateral movement.

Round steel plugs containing gauge holes were grouted into the inner side of the rib on the axis so as to mark off consecutive 5-foot chords. Midway between each pair of these plugs a square plug was grouted into the rib so that its upper surface and the line between the gauge holes were in parallel planes normal to the face of the rib. In measuring mid-ordinate changes, the two steel points of the instrument were set in the gauge holes of a pair of plugs and the stem extension of the dial was allowed to rest on the square plug. On the back of the dial case was an arm which rested against the side of the rib and maintained the dial in a plane parallel to and at a constant distance from the face of the rib. The difference between readings before and after the application of the load gave the change in mid-ordinate.

Clinometer and Radiusmeter Gauge Plugs in Inner Face of Rib

MEASUREMENT OF PIER MOVEMENT

Provision was made for measuring pier movements in two ways. Clinometer plugs were placed on top of each end of each pier and on a line parallel to the axis of the bridge. Readings on these points gave rotations of the top of the pier but not displacements parallel to the bridge axis. Provision for measuring these displacements was made by stretching two wires on each side of the bridge fixed to points on the far piers of the spans adjacent to that being loaded. The wires were stretched so that they just cleared the ends of the piers and each pair of wires was spaced 5 feet apart vertically. Metal gauge plugs were fixed in the piers opposite a copper reference mark fixed to each wire. Horizontal movement of the points on the piers with reference to the fixed points on the wires were measured with a scale. Differences of movements of the upper and lower points on the piers gave an additional measure of the pier rotation.

MODULUS OF ELASTICITY OF THE CONCRETE DETERMINED

In order to determine the modulus of elasticity of the concrete for use in the preliminary computations, one specimen was cut from a curtain wall in each pier of the span under observation. Two 6 by 12 inch cylinders were drilled from each of these specimens and tested. Compression tests gave an average value of 4,500,000 pounds per square inch for the modulus of elasticity. These specimens were taken from an unimportant part of the structure which was not under stress and it was thought advisable to make further determinations of this value.

A more representative value of the modulus of elasticity was obtained after the completion of the field observations. Specimen blocks were taken from the rib at the sections where the telemeters had been attached. Nine of the specimens were taken with the intention of drilling two test cylinders from each block. Because of defects in the specimens only seven suitable test cylinders were obtained. These cylinders were 6 inches in diameter with an average length of 10 inches.

The stress-strain curves obtained with a mirror extensometer of the Martens type [5] are shown in Figure 3. The curves show an average value of the proportional limit of 1,111 pounds per square inch, an average ultimate strength of 4,293 pounds per square inch and an average modulus of elasticity of 3,930,000 pounds per square inch.

MATHEMATICAL ANALYSIS MADE OF THE FREE RIB

Field measurements were taken of the dimensions of the arch rib. The rib was analyzed by dividing the span into 20 equal horizontal divisions and applying the theory of elastic structures by the method of summations. It was assumed that the superstructure produced no effect on the deformation of the rib other than the effect as dead load. Since field observations showed no measurable movements of the piers, a condition of perfect fixity of the rib at the springing lines was assumed.

A value of 4,000,000 pounds per square inch was used for the modulus of elasticity of concrete and 30,000,000 pounds per square inch for that of steel which resulted in a value of 7.5 for n in the beam and column formulas. The coefficient of expansion for concrete was taken as 0.00001 per degree centigrade.

TABLE 1.—*Calculated horizontal thrusts, vertical shears, and moments at the springing line, and calculated moments at the telemeter points for unit loads of 1 pound at the columns* [1]

Load at column	H_o	V_o	M_o	M_a	M_b	M_c	M_d	M_e
	Pounds	*Pounds*	*Ft.-lbs.*	*Ft.-lbs.*	*Ft.-lbs.*	*Ft.-lbs.*	*Ft.-lbs.*	*Ft.-lbs.*
1	0.082	0.990	−9.35	−6.04	+1.31	+0.57	+0.09	−0.25
2	.349	.953	−12.86	−10.32	+1.59	+2.79	+.48	−.84
3	.720	.885	−11.11	−9.80	−2.57	+6.92	+1.85	−1.28
4	1.132	.779	−5.38	−5.50	−3.88	+1.55	+4.90	−.73
5	1.434	.650	+1.60	+.20	−3.05	−1.82	+4.01	+1.56
6	1.550	.500	+7.75	+5.46	−1.29	−3.41	−.75	+6.42
5′	1.434	.350	+10.96	+8.51	+.45	−3.48	−3.03	+1.56
4′	1.132	.221	+10.80	+8.61	+1.37	−2.69	−3.35	−.73
3′	.720	.115	+7.90	+6.30	+1.40	−1.60	−2.50	−1.28
2′	.349	.047	+4.10	+3.40	+.85	−.71	−1.28	−.84
1′	.082	.010	+1.01	+.83	+.25	−.17	−.32	−.25

[1] Letters *a*, *b*, *c*, *d*, and *e* designate the points along the axis at which the telemeters were attached to the rib, the point *a* being near the springing line and the others in order toward the crown where the point *e* is located.

[5] For a complete description of the mirror extensometer see Handbook of Testing materials by Adolph Martens, Pt. I, vol. 1, pp. 67 to 76.

A tabulation of the ordinates of the influence lines thus computed is shown in Table 1. From these influence lines moment diagrams were drawn and stresses, rotations, deflections, and mid-ordinate changes were computed.

MODEL ANALYSES MADE

A preliminary analysis was made with Beggs deformeter gauges using a paper model. This analysis was not complete owing to the difficulty of placing the gauges on the rib in panels near the crown, where the superstructure interfered with the placing of the plugs used to produce displacements in the model.

Next an attempt was made to analyze the arch by the use of a brass wire model, applying the principles used in the Beggs method. The thrust, shear, and moment at the end were accurately determined by displacing the brass plates used to represent the piers. However, when an effort was made to produce displacements near the piers and crown with the piers fixed, difficulty was experienced in getting sufficiently large deflections to be accurately measured by the means used because of the stiffness of the model.[6]

The results of the above analyses were finally discarded and an analysis was made with the Beggs gauges and a model cut from sheet celluloid 0.08 inch thick. This model included the two spans adjacent to the span under observation. It was first analyzed with

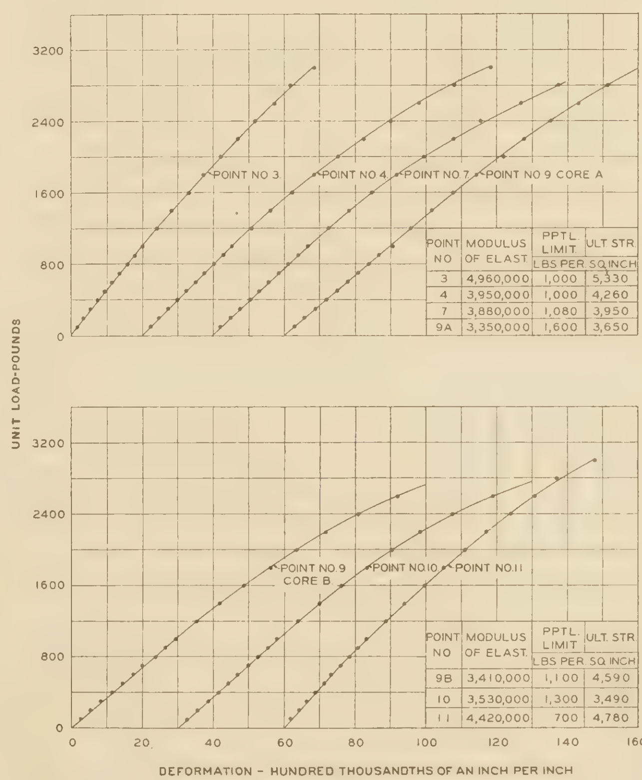


FIG. 3.—STRESS-STRAIN CURVES FOR 6-INCH CORES DRILLED FROM NORTH ARCH RIB AT CLINOMETER POINTS. CORES TESTED IN DRY CONDITION. DAMP CORES (NOT SHOWN) LOADED UP TO 700 POUNDS PER SQUARE INCH SHOWED MODULI APPROXIMATELY 5 PER CENT LESS THAN WHEN DRY

[6] A description of this method applied to end conditions for statically indeterminate frames may be found in an article entitled "Brass Wire Models Used to Solve Indeterminate Structures" by A. Bull, Engineering News-Record, Dec. 8, 1927.

the columns integral with the floor system but with cuts in the deck at sections corresponding to the expansion joints in the actual structure. Influence lines for thrust, shear, and moment were determined in each panel at sections of the model corresponding as nearly as practicable to the sections of the rib at which the telemeters were placed.

Then the model was modified in an effort to simulate the condition of fixity which obtained at the tops of the columns in the test bridge. The floor system was cut loose from the columns and connected again by welding flexible webs of celluloid across the cuts. The size of these connecting pieces was arbitrarily chosen as there was no means of determining the flexibility which would produce the same degree of fixity as that existing in the bridge. The analysis was repeated with the model in this condition. The entire superstructure was next removed and the analysis made on the free rib. For convenient reference, the analyses with the model in the three foregoing conditions are designated as A, B, and C, respectively. A complete description of this analysis, with detailed results, will appear in an early issue of Public Roads.

From the influence lines thus determined, the moment diagrams for all conditions of loading were drawn. In cases A and B, moments were calculated at each side of each column and at the springing line. Rotations were determined from these moment diagrams.

Stresses were computed on the extrados and intrados at each telemeter point.

COMPUTED AND MEASURED DEFORMATIONS COMPARED

The loading of the bridge was begun in September, 1927, and observations were taken day and night until December, 1927, with only two interruptions; one when the ferry broke down and the loading tanks had to be moved from the bridge to allow traffic to pass, and the other during the time required to cut the deck in preparation for series 2 and 3 loadings. Night as well as day observations were made so as to take advantage of the more uniform atmospheric conditions at night.

The data were compiled and compared as soon as they were observed. In this way any obviously inconsistent data were discovered and those due to instrumental imperfections were eliminated as far as practicable.

The computed and measured results are compared by means of charts showing the deformations of the rib for the conditions of continuity of superstructure and positions of load as described above. This comparison is made for deformations as measured by fiber stresses, rotations of axis, deflections, and changes in lengths of mid-ordinates of 5-foot arcs of the axis. In addition, the measured and theoretical stresses and rotations are compared with values computed from the results of the analysis of the structure with the Beggs deformeter gauges.

A full tank load on an adjacent span produced no measurable deformations in the span under observation and no measurable pier movement occurred during the test. Therefore, all computed results are based on fixity of the rib at the pier. All deformation curves are plotted with abscissas measured along the axis of the rib.

The deformations are shown for loads exclusive of the weight of the tank. In this manner, the effects of temperature changes are reduced to a minimum in

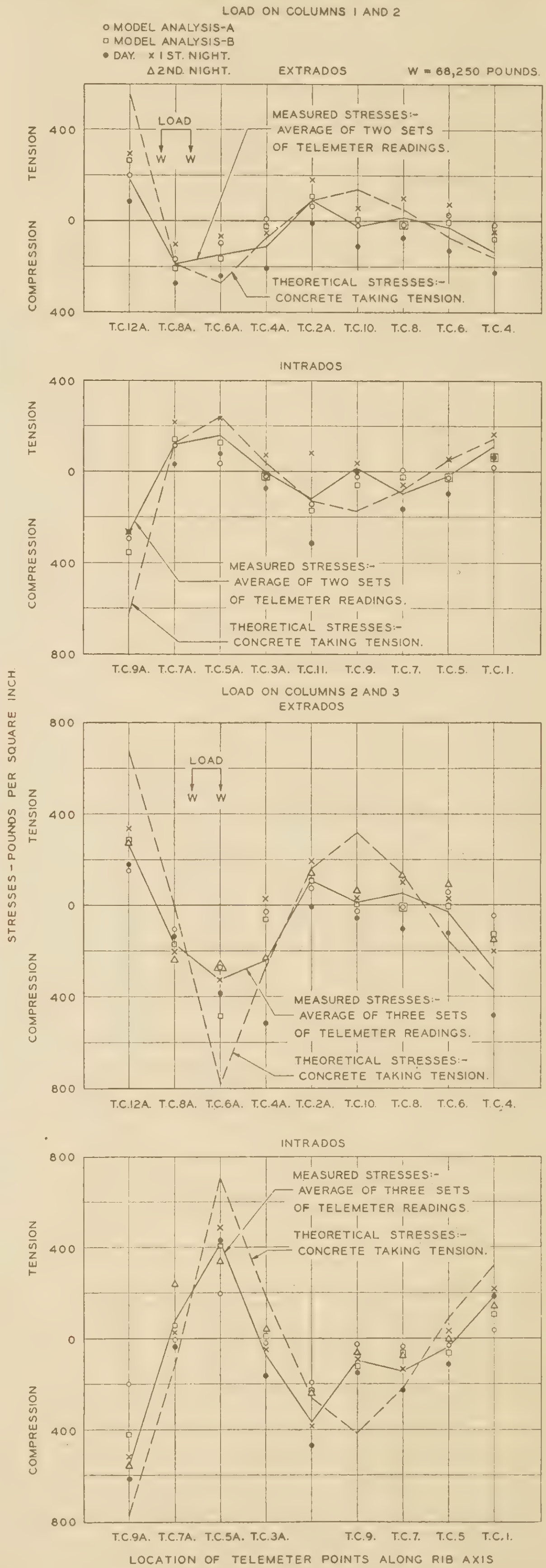


Fig. 4.—Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1)

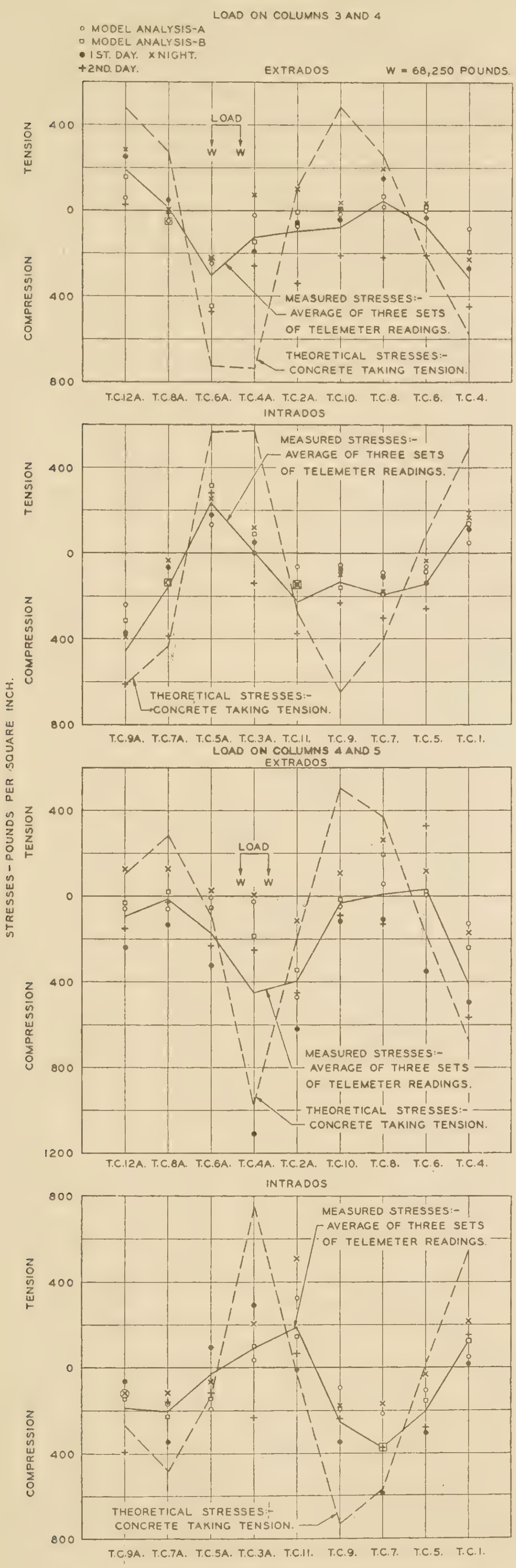


Fig. 5.—Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1)

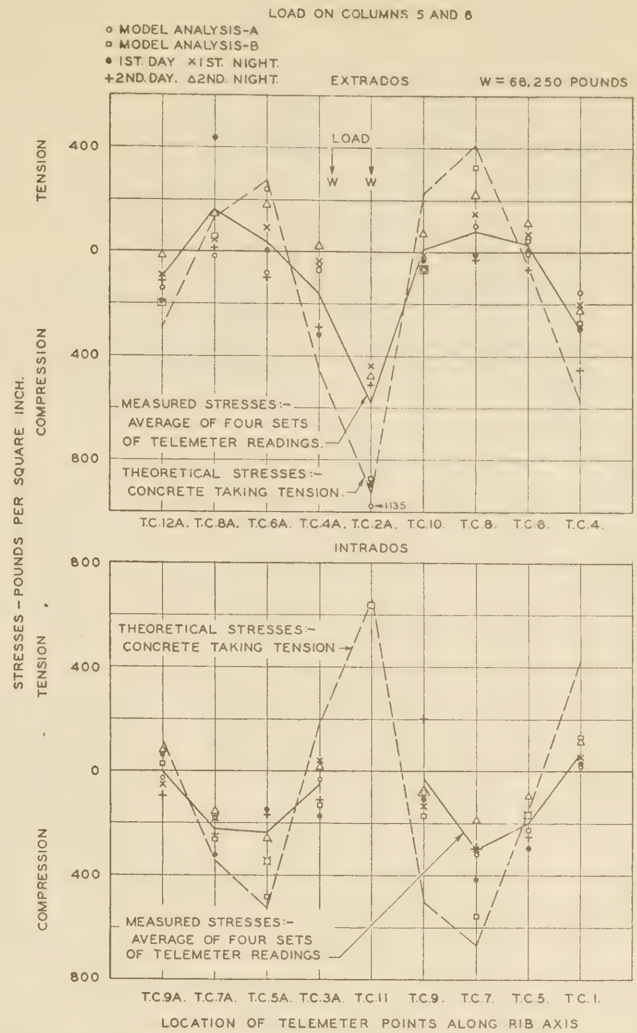


Fig. 6.—Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1)

making comparisons. The tank was placed in position and a set of readings taken on all instruments. Then loads of 91,000, 182,000, and 273,000 pounds of water were pumped into the tank, readings being taken at the completion of each increment. Thus, the total elapsed time between the first and last readings for any position of the load was about four hours.

The temperature changes during this period were generally very slight, whereas, if the comparisons had been made from zero readings, a period of as much as a week would sometimes have elapsed between readings in the same set, with the probablity of considerable temperature changes. Only results for the maximum panel-point loads of 68,250 pounds are shown in the cases of stresses and mid-ordinate change because of the difficulty of accurately measuring the deformations resulting from the smaller increments of load. Deformations for all three increments of load are shown on the rotation and deflection curves.

STRESSES IN THE CONCRETE AT THE EXTRADOS AND INTRADOS OF THE RIB

The stresses on the intrados and extrados are plotted separately and at normal sections of the rib 18 feet 6 inches apart along the axis of the rib for each position of the load and condition of superstructure.

Figures 4 to 6 are for series 1, in which the superstructure was intact. The stress values through

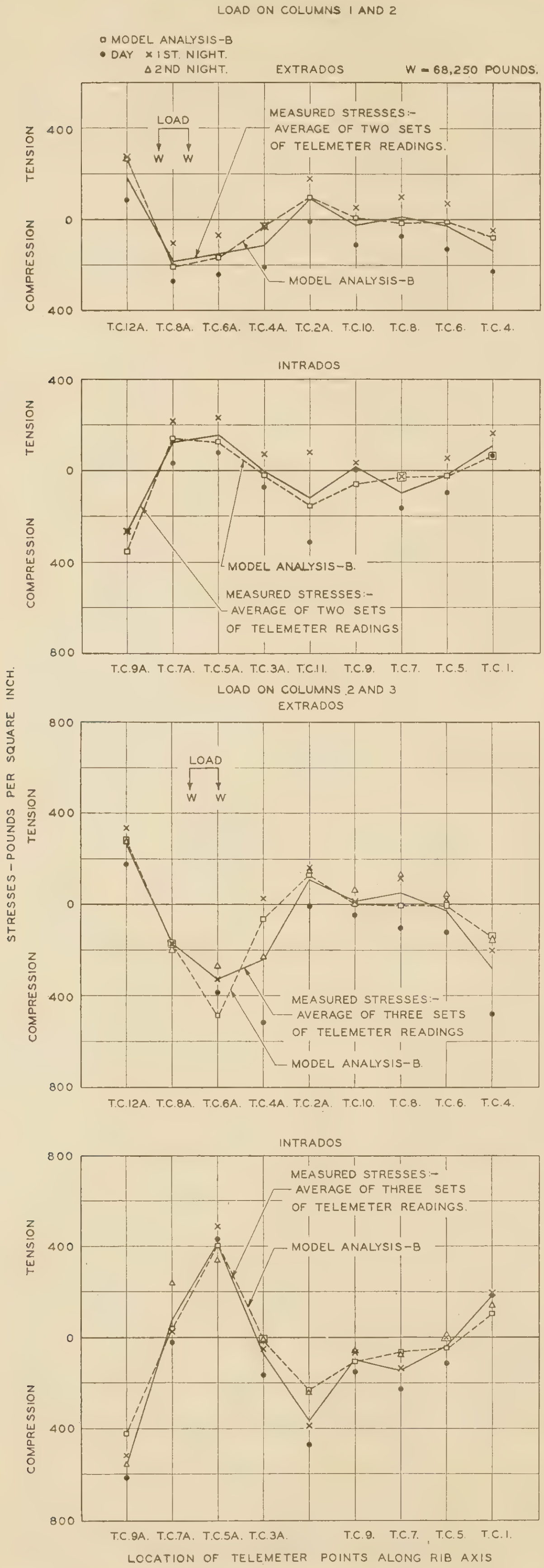


Fig. 8.—Comparison of Measured Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1) and Model Analysis B

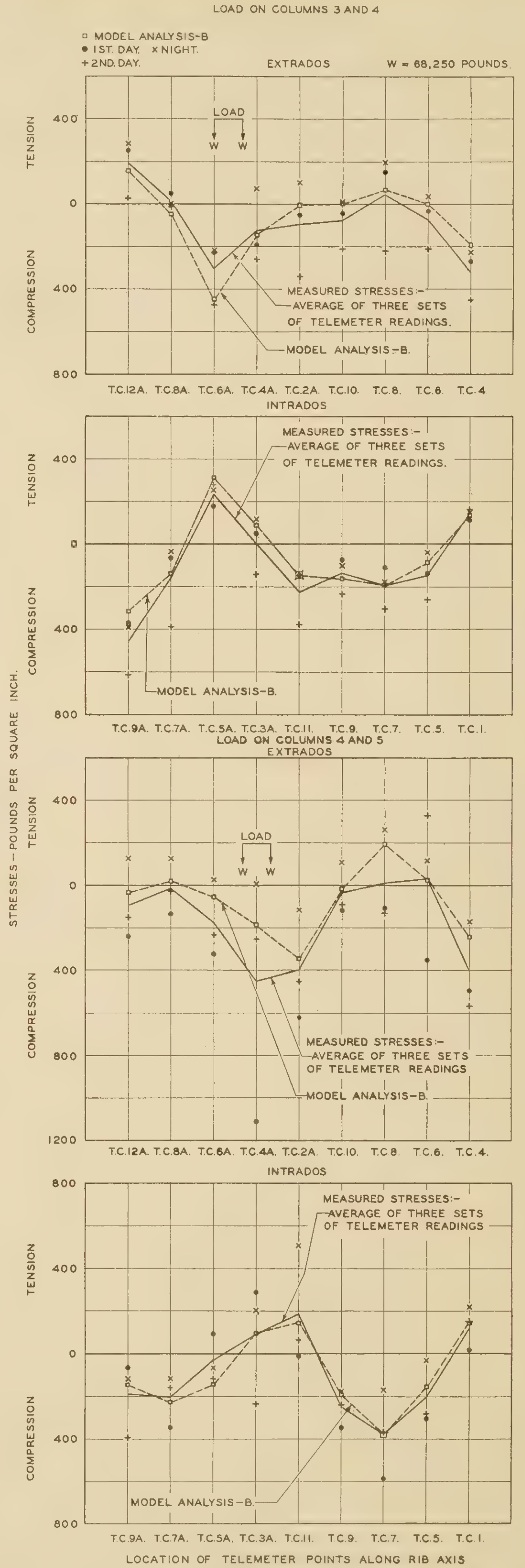


Fig. 9.—Comparison of Measured Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1) and Model Analysis B

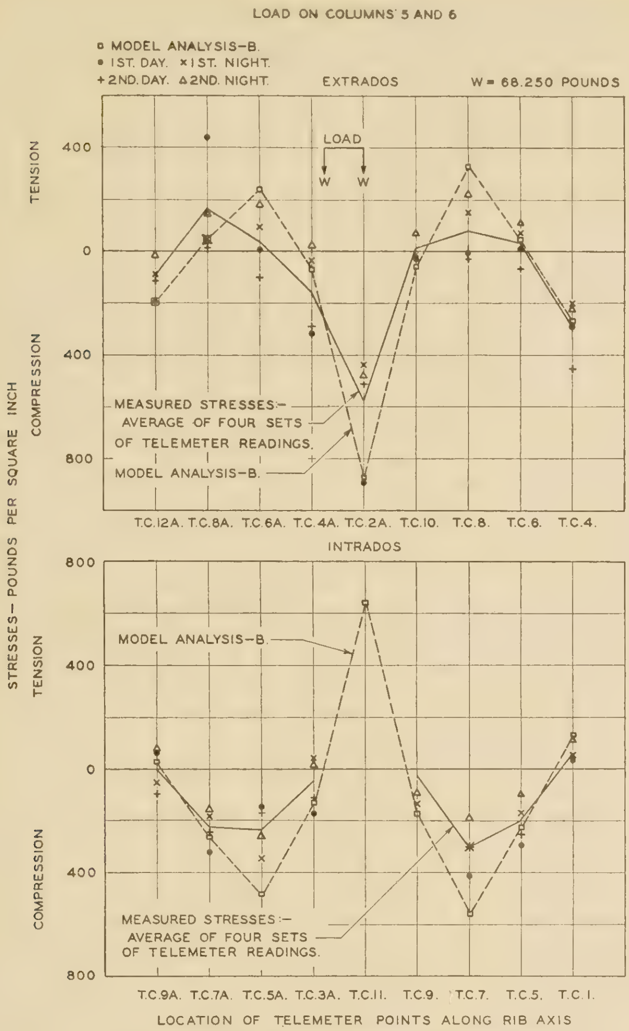


Fig. 9.—Comparison of Measured Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1) and Model Analysis B

which the full lines are drawn are the averages for the several sets of readings. Each of the values from which the average was derived is shown by a distinctive symbol. The computed values through which the broken lines are drawn are based on the assumptions that the rib is free from restraint by the superstructure and that the concrete takes tension, the steel taking only its proportionate share. As will be shown later in discussing series 2, the compressive stresses calculated on the latter assumption are closer to the measured stresses than those based on the assumption that the concrete takes no tension, even when the tension is high enough to produce a visible crack on the tension side of the rib.

The straight lines drawn between the stress values are not intended to represent the manner in which the stress changes between the plotted points but are intended solely to aid in following the points through the chart.

Stresses derived from the model analysis, for the two conditions of column fixity, A, and B, are also shown on the charts. Figures 7, 8, and 9 show a comparison of the average measured stresses for series 1 and the stresses derived from model analysis B. These charts afford an indication of the effect of the superstructure on the stresses produced in the rib, as well an an indication of the accuracy with which the stresses may be deduced from an analysis by the use of a model.

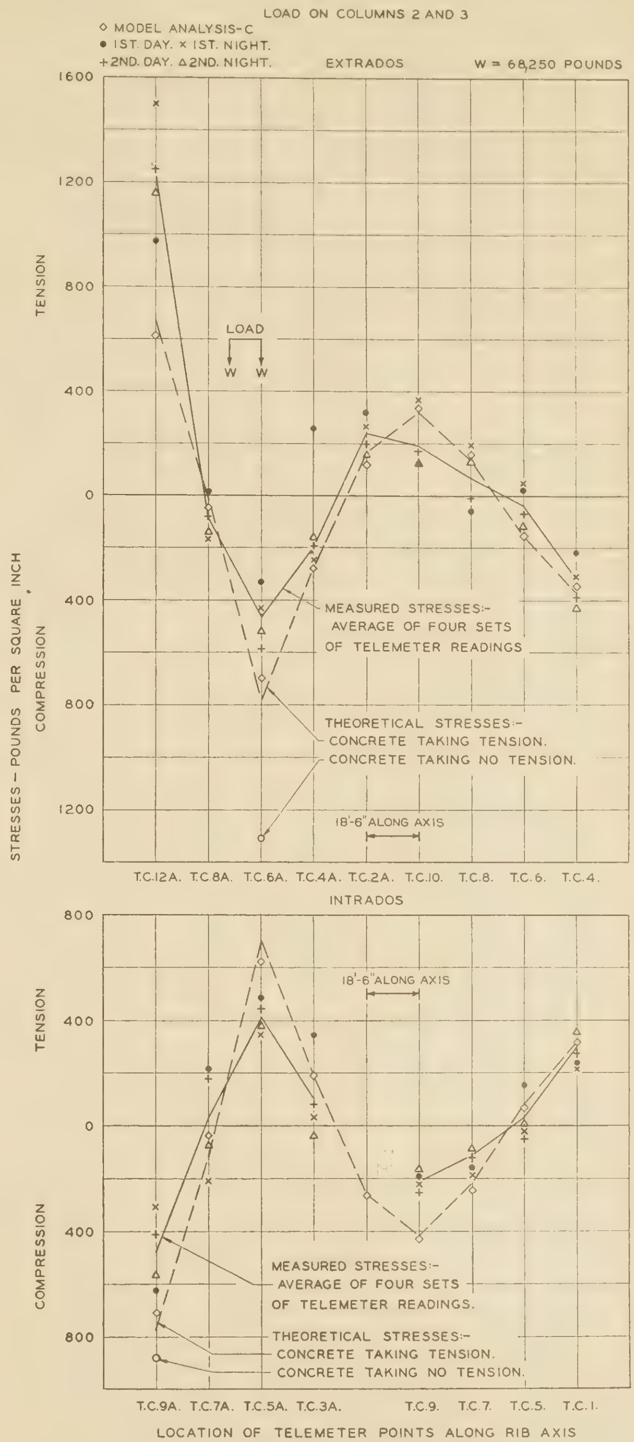


Fig. 10.—Stresses in Arch Rib Under 1-Tank Loading with Deck Cut (series 2)

These charts indicate that, in general, the rib stresses with structure intact are less in maximum value than the computed stresses for the unrestrained rib and that the stresses calculated from the model analysis with the two assumed conditions of fixity of the tops of the columns are less than the measured stresses. There is a closer agreement between measured stresses and those obtained by the model analysis than between the measured stresses and those computed for the unrestrained rib.

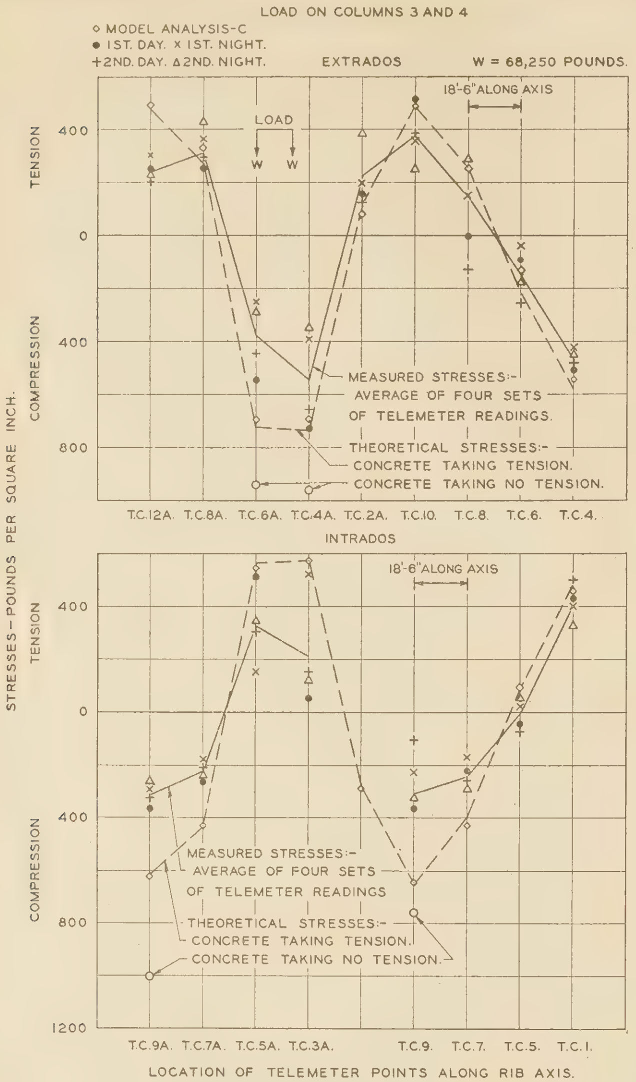


FIG. 11.—STRESSES IN ARCH RIB UNDER 1-TANK LOADING WITH DECK CUT (SERIES 2)

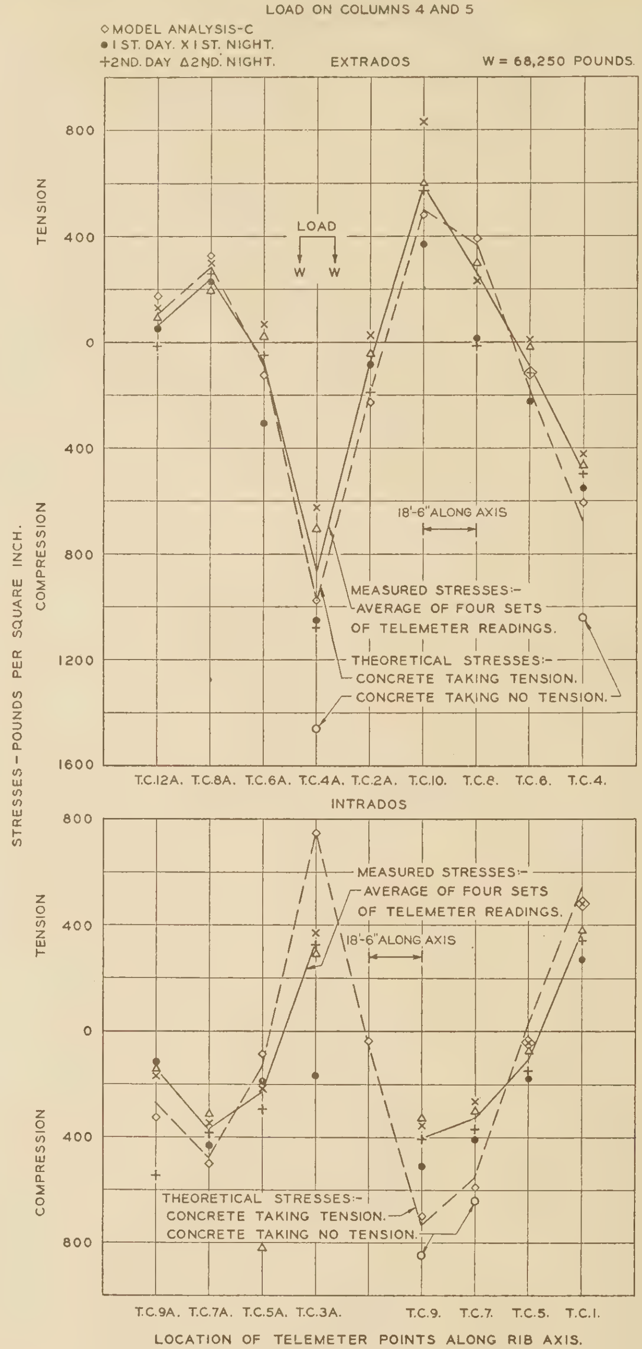


FIG. 12.—STRESSES IN ARCH RIB UNDER 1-TANK LOADING WITH DECK CUT (SERIES 2)

It may be observed that, for model condition B (see p. 194), the stresses, in general, check the measured stresses closely, while those for model condition A are somewhat lower, particularly for maximum stresses. It must be said, however, that this check is more or less accidental since the size of the connection between the floor and columns in model B was arbitrarily selected. The stresses for conditions A and B differ most in the regions of maximum stress. It will be noticed that, at some points of low stress, the model results for condition A check the measured results more closely than for condition B. The rotation curves (to be discussed later), will show these relations more clearly.

When the load was placed on columns 5 and 6 in series 1, a crack occurred on the intrados of the rib between the points of attachment of telemeter No. 11. In series 2 and 3 this telemeter was removed from the rib and no stresses are shown at this point on the charts.

Figures 10 to 13 show stress values for series 2 in which the continuity of the deck had been destroyed by cutting through the floor system and inserting new bearing plates. The stresses in this series are plotted in the same manner as in series 1, and, in addition, where tensile stresses exist, the compressive stresses on the opposite side of the rib are calculated on the assumption that the concrete takes no tension. These latter values are indicated by open circles.

Agreement between the measured and computed results is much closer than in series 1, becoming closer as the load moves toward the crown and the deflection becomes greater. It is also noted that the agreement between the measured and computed results is much closer when it is assumed that the concrete takes tension. The measured stresses were nearly always less than the computed stresses. The results from the model analysis of the free rib check the computed results very closely.

Figure 14 shows the stresses for series 3 in which the deck was cut and two tank loads placed so as to produce maximum stress at the springing line. The agreement between measured and theoretical stresses is again

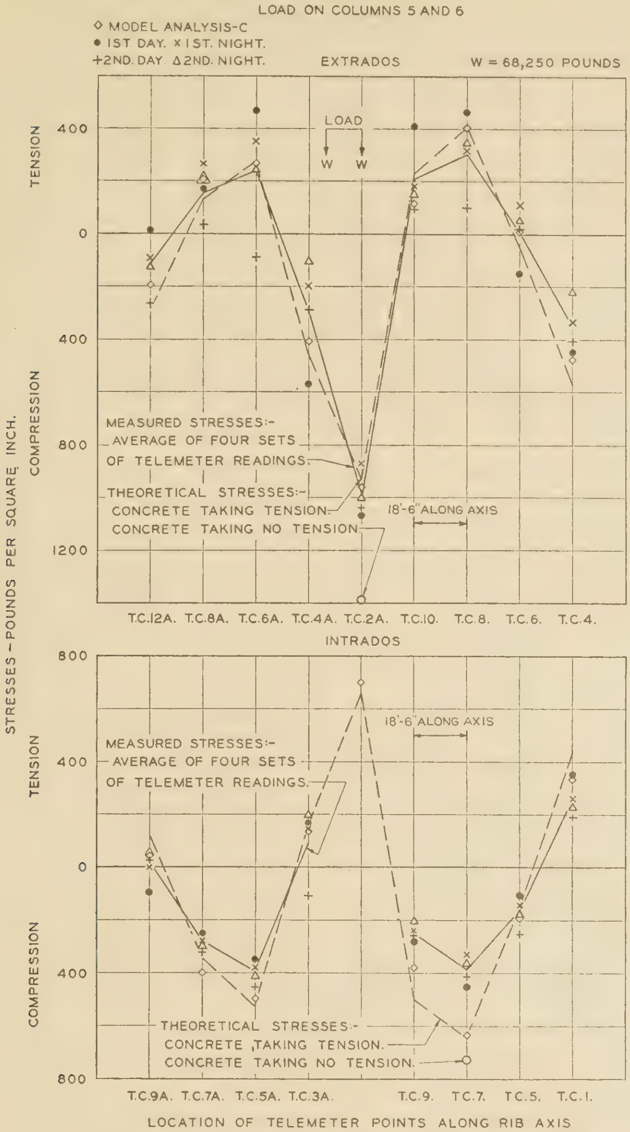


Fig. 13.—Stresses in Arch Rib Under 1-Tank Loading with Deck Cut (series 2)

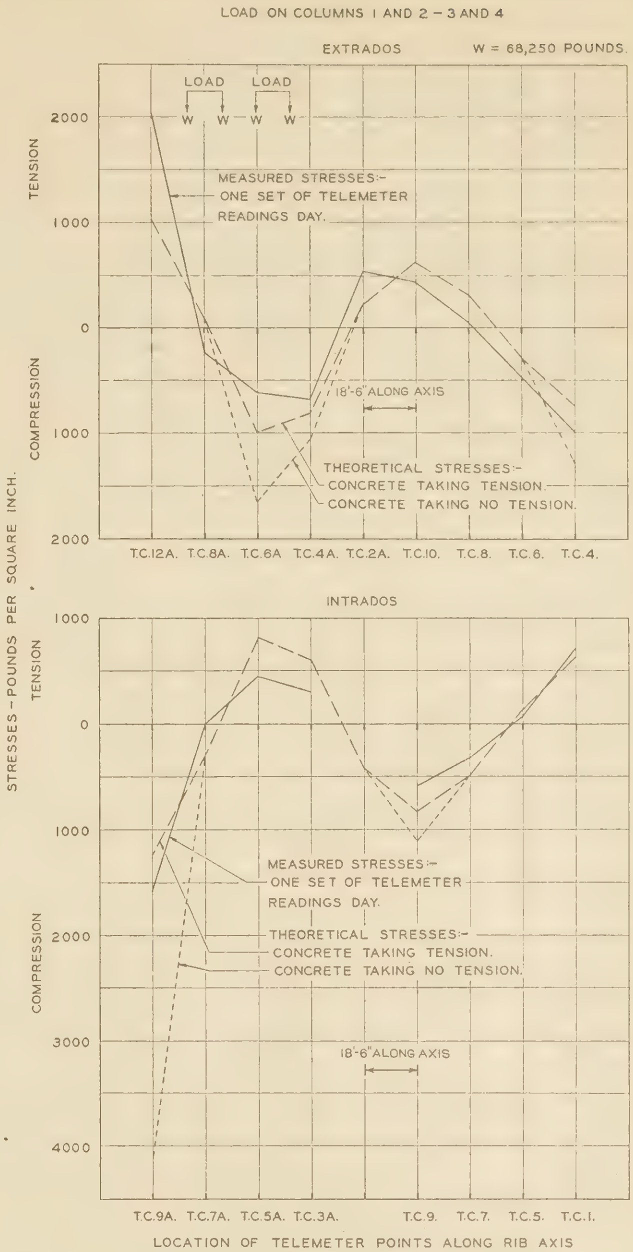


Fig. 14.—Stresses in Arch Rib Under 2-Tank Loading with Deck Cut (series 3)

close. In this series a visible crack occurred at the springing line, extending over four-fifths of the depth of the rib. Nevertheless, there is still a much closer agreement between the computed and measured compressive stresses when the former are calculated on the assumption that the concrete takes tension.

Loads of 68,250 pounds at columns 1, 2, 3, and 4 produced a measured compressive stress of about 1,600 pounds per square inch in the concrete of the intrados at the springing line. When this measured stress is combined with the computed stress due to the weight of the tank, dead load, shrinkage, and temperature, the stress is increased to a value of about 3,000 pounds per square inch. Such high stresses over a short length of the rib apparently do not affect the general relation of computed to measured stress, although, as will be seen later, the rotations and deflections are measurably increased in series 3.

Figures 15 to 18 present a comparison of measured stresses of series 1 and 2, the difference between the two showing the effect of the superstructure. It is seen that the stresses for series 1 are practically always less than series 2 at the maximum values. The difference increases as the load moves from the springing line to crown and is greatest when the loads are near the crown. It was observed in series 2 that the measured stresses were nearly always somewhat less than the computed.

Figures 19 and 20 are intended to show variations in stress at the respective telemeter points as the load moves from pier to pier. Stresses are shown as observed and as calculated theoretically. The curves for measured values are mean curves, the diagram showing each point from which the mean is derived. In calculating the averages several obviously erratic values were discarded but all values are plotted on the diagrams. The points at which the stresses are plotted are located at the centers of gravity of the loads or midway between the columns on which the loads were placed. The curves are plotted on the assumption that the rib is symmetrical and the materials are homogeneous and that, therefore, a load placed at any

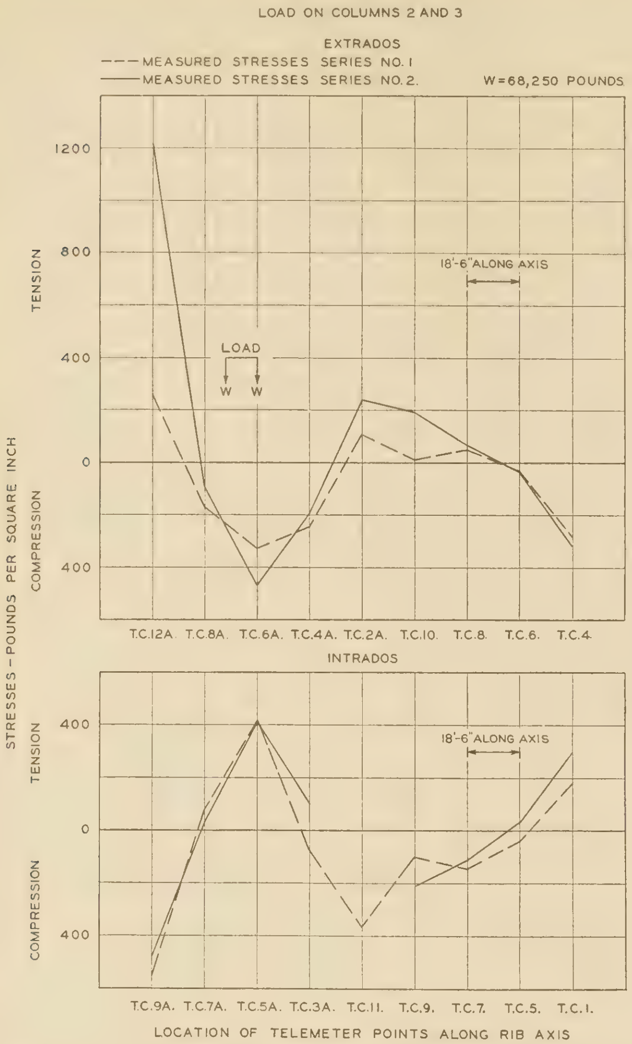


Fig. 15.—Comparison of Stresses in Arch Rib Under 1-Tank Loading with Deck Intact, (series 1) and with Deck Cut (series 2)

Table 2.—*Observed and computed stresses in steel in series 2* [1]

[Load: 68,250 pounds per column. E_s=30,000,000 pounds per square inch]

Columns loaded	Telemeters	Telemeter results				Computed stresses, concrete taking—	
		First day	First night	Second day	Second night	No tension	Tension
2 and 3	Ex. 11A	T10,700	T 9,220	T 8,180	T 6,280	T35,850	T4,760
Do	1A	T 680	C 5,700	T 1,620	T 420	T 1,220	T 162
Do	3	C 1,370	C 1,650	C 1,520	C 1,700	C 2,730	C 364
Do	In. 11	C 2,770	C 5,780	C 3,350	C 3,570	C 1,940	C 259
Do	2	T 2,310	T 1,590	T 1,340	T 540	T 2,640	T 352
3 and 4	Ex. 11A	T 2,340	T 3,590	T 1,610	T 2,650	T13,000	T1,764
Do	1A	T 500	T 630	C 650	T 2,140	T 880	T 117
Do	3	C 2,430	C 1,290	C 2,120	C 3,900	C 5,540	C 739
Do	In. 11	C 4,640	C 2,300	C 2,010	C 2,450	C 2,120	C 283
Do	2	T 2,000	T 2,480	T 1,560	T 2,380	T 8,770	T1,182
4 and 5	Ex. 11A	C 500	T 1,270	C 220	T 3,820	T 850	T 113
Do	1A	C 240	C 70	C 670	C 860	C 1,480	C 197
Do	3	C 3,620	C 3,140	C 2,450	C 1,140	C 7,030	C 937
Do	In. 11	C 3,760	C 670	C 1,400	C 780	C 230	C 30
Do	2	T 1,280	T 2,910	T 1,530	T 2,160	T12,900	T1,720
5 and 6	Ex. 11A	C 2,370	C 750	C 1,840	C 490	C 2,150	C 287
Do	1A	C 2,550	C 4,510	C 3,490	C 3,380	C 8,510	C1,135
Do	3	C 5,000	C 2,380	C 2,200	C 1,720	C 4,690	C 626
Do	In. 11	T12,410	T10,230	T11,930	T11,380	T14,600	T1,950
Do	2	T 940	T 1,460	T 1,320	T 1,720	T 4,830	T 645

[1] Tension and compression are indicated by the letters T and C, respectively.

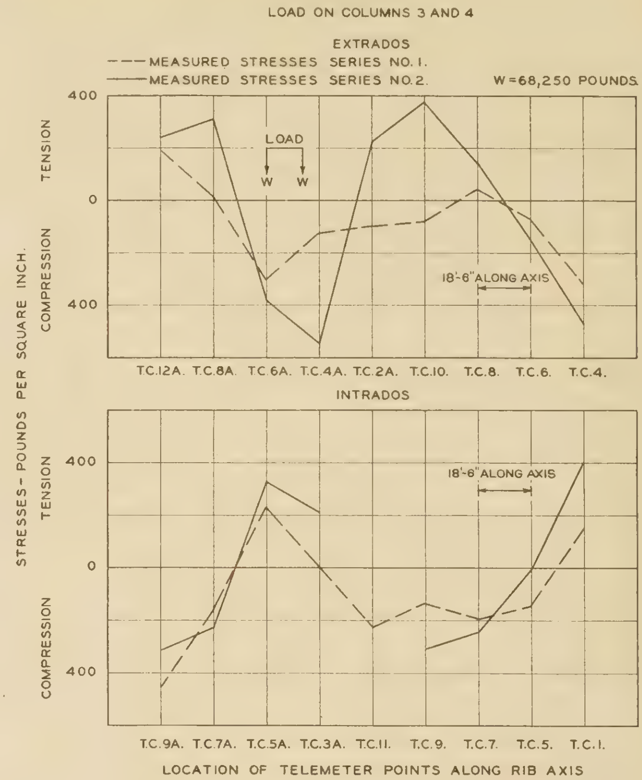


Fig. 16.—Comparison of Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1) and with Deck Cut (series 2)

point, say, at column 3, produces a stress at any point, say, telemeter point a' on the opposite side of the crown, equal to the stress at a produced by an equal load at 3′, where the position of each of the pairs, 3, 3′, and a, a', are symmetrical with respect to the crown.

STRESSES IN STEEL DISCUSSED

Table 2 shows observed and computed stresses in the reinforcing steel at the crown and springing lines. It is noted that the measured stresses are, in general, less than the stresses computed on the assumption that the concrete takes no tension and greater than those computed on the assumption that the concrete takes its proportionate part of the tension. In the case of the high tensile stresses, the measured stresses are considerably less than the computed stresses based on the assumption of no tensile strength for the concrete.

ROTATION OF THE ARCH AXIS DISCUSSED

In the curves showing rotation, the slope of any curve is proportional to the bending moment divided by the moment of inertia of the section. Therefore, at any section along the rib, the slopes of any series of rotation curves are proportional to the bending moments. This relation between bending moment and slope of the rotation curves should be kept in mind in making the following comparisons. The smooth curves drawn through the measured rotation values are not intended to represent the exact manner in which rotations change between sections at which measurements were made.

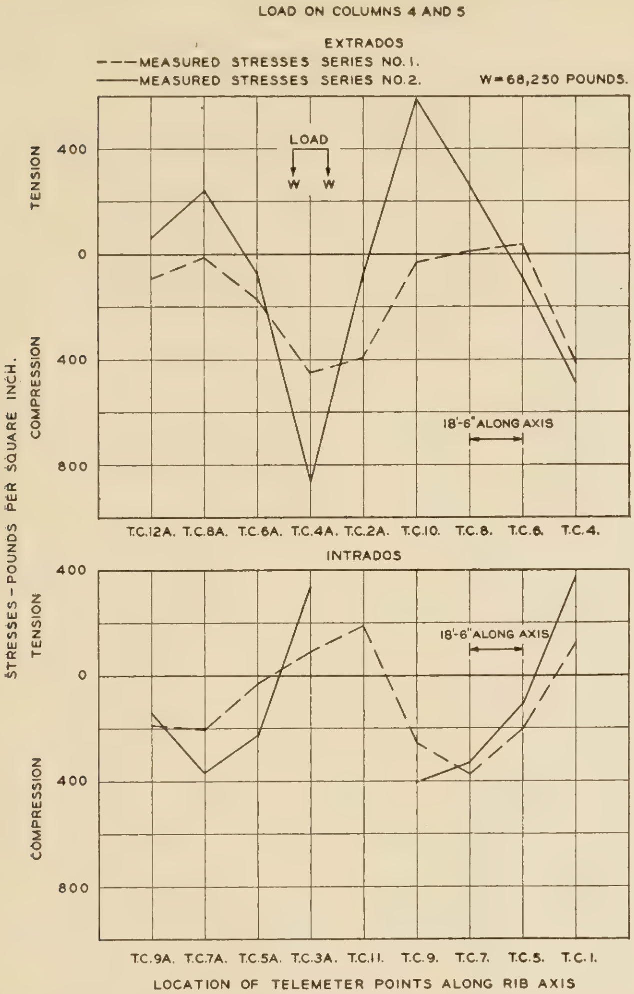


Fig. 17.—Comparison of Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1) and with Deck Cut (series 2)

Figures 21, 22, and 23, show rotations in series 1 for five positions of load. The measured values are shown for three increments of load, 22,750 pounds, 45,500 pounds, and 68,250 pounds per column. The computed curve is shown for a column load of 68,250 pounds only.

It will be noted that the measured rotations for this series are of the same general nature as the computed rotations but are much reduced in value at the points of maximum values. The influence of the superstructure on the moment is clearly seen. The general influence of the floor system may be seen in the reduction of the maximum values of the rotations and, therefore, the reductions in slopes of the curve between these high and low points. The local effect of the curtain walls between panel points 3 and 3′ may be seen by the flattening of some of the curves after passing panel points 3 and 3′ toward the crown. These are the points at which the curtain wall ends and has its maximum height. The curtain wall tapers off to a height of 0.85 foot at the crown where its effect is negligible.

Figures 24 and 25 show the rotations for series 2 with loads on each pair of columns from 2 to 6 and for the same increments of loads as in series 1. The computed and measured values of the rotations for this series show a remarkable agreement. The maxi-

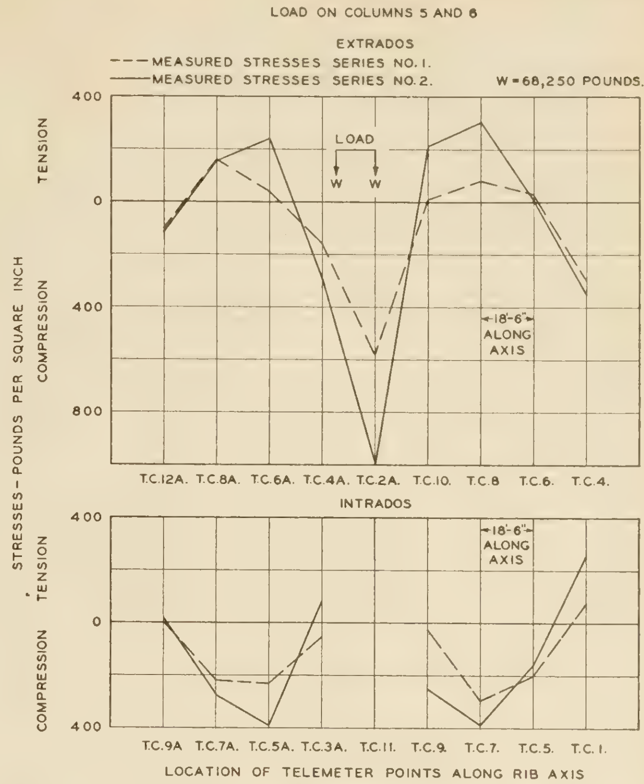


Fig. 18.—Comparison of Stresses in Arch Rib Under 1-Tank Loading with Deck Intact (series 1) and with Deck Cut (series 2)

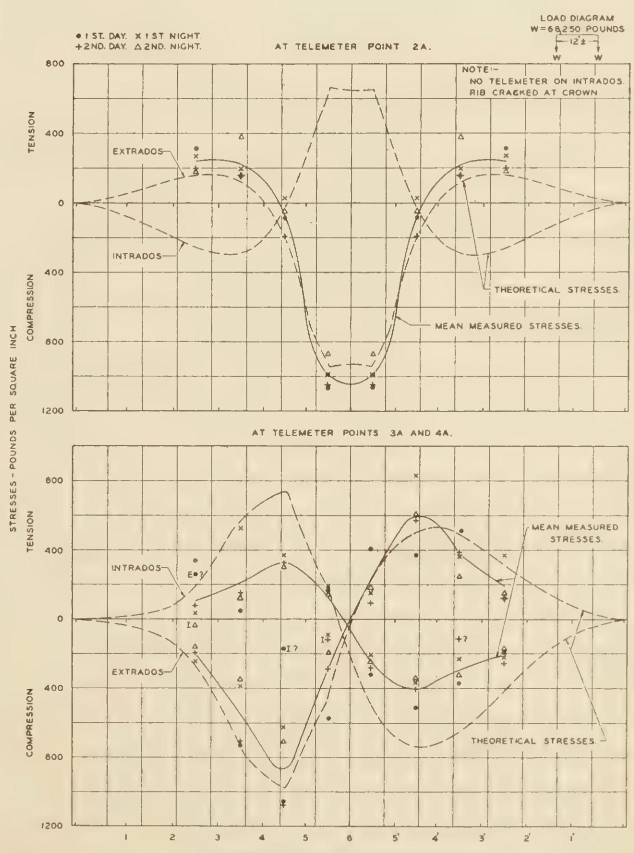


Fig. 19.—Variation in Stress in the Concrete at Telemeter Points for a 1-Tank Load Moving Across the Span with Deck Cut (series 2)

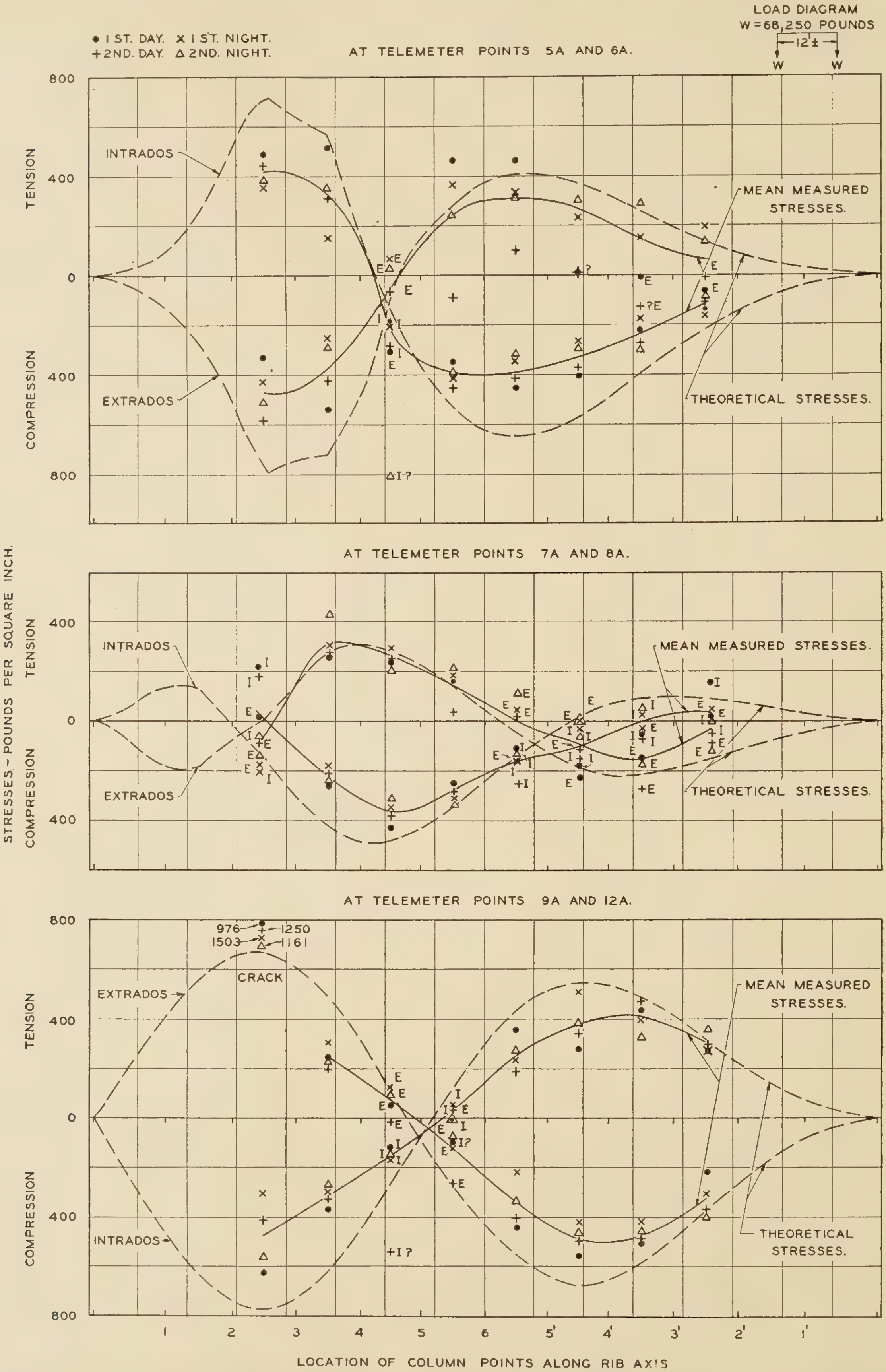


FIG. 20.—VARIATION IN STRESS IN THE CONCRETE AT TELEMETER POINTS FOR A 1-TANK LOAD MOVING ACROSS THE SPAN WITH DECK CUT (SERIES 2)

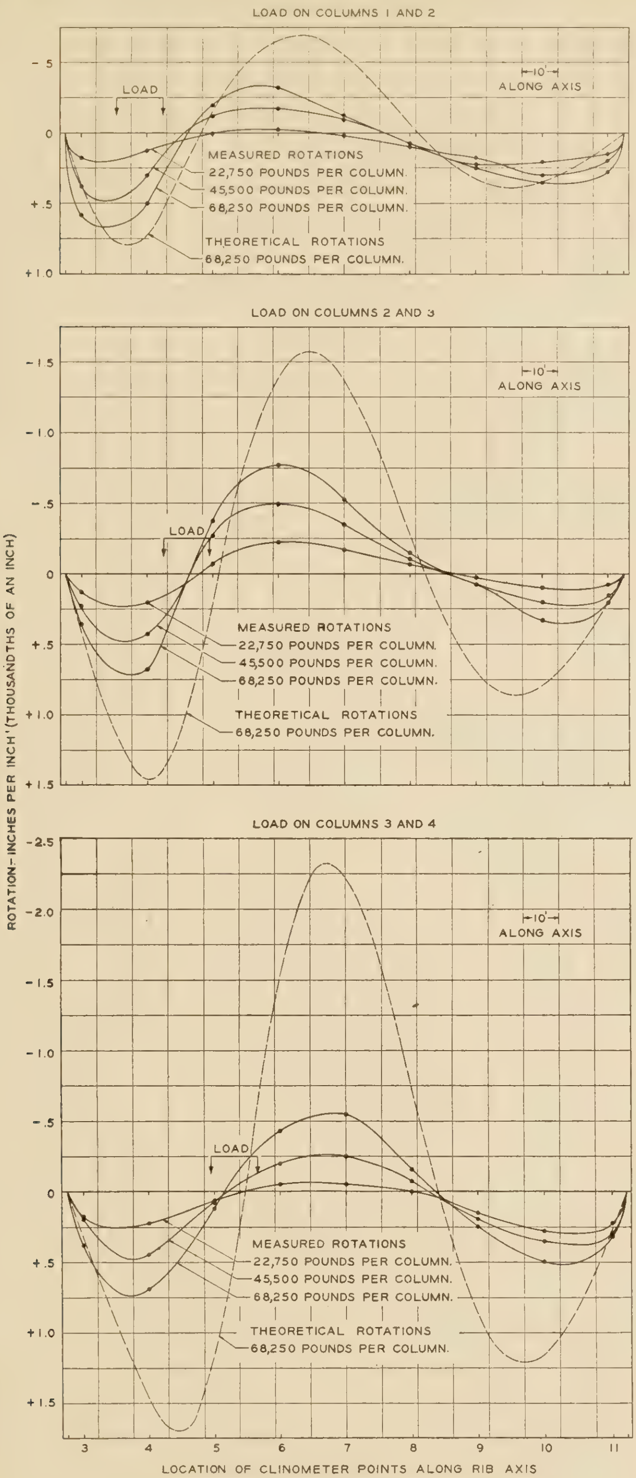


Fig. 21.—Rotations of Arch Rib Under 1-Tank Loading with Deck Intact (series 1)

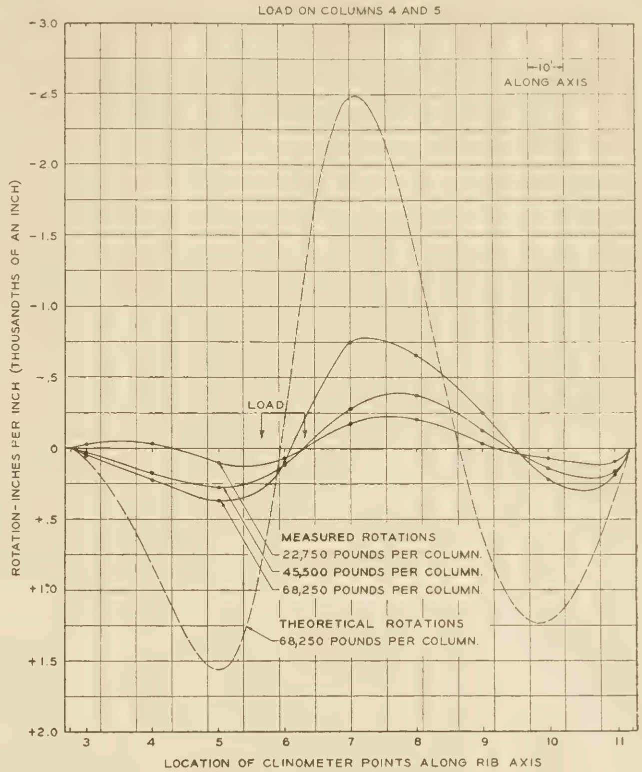


Fig. 22.—Rotations of Arch Rib Under 1-Tank Loading with Deck Intact (series 1)

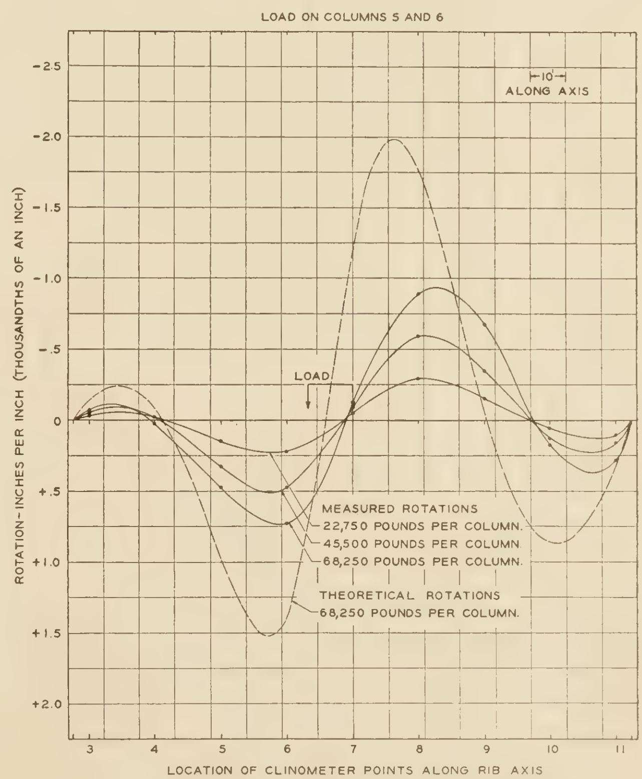


Fig. 23.—Rotations of Arch Rib Under 1-Tank Loading with Deck Intact (series 1)

mum slopes of the computed and measured curves are practically the same over a considerable length of the rib. The greatest differences of slopes of the two curves occur near the points of maximum rotations or zero bending moment. At these points it may be expected that the greatest discrepancies between computed and measured stresses would occur in so far as the stress is dependent on the bending moment.

Figure 26 shows rotations for series 3 with loads over columns 1, 2, 3, and 4. The maximum measured rotations in this case show greater values than maximum computed rotations although the maximum slope and, therefore, maximum moments agree very closely. It appears that even though the maximum rotations for this case are somewhat increased by the cracking at

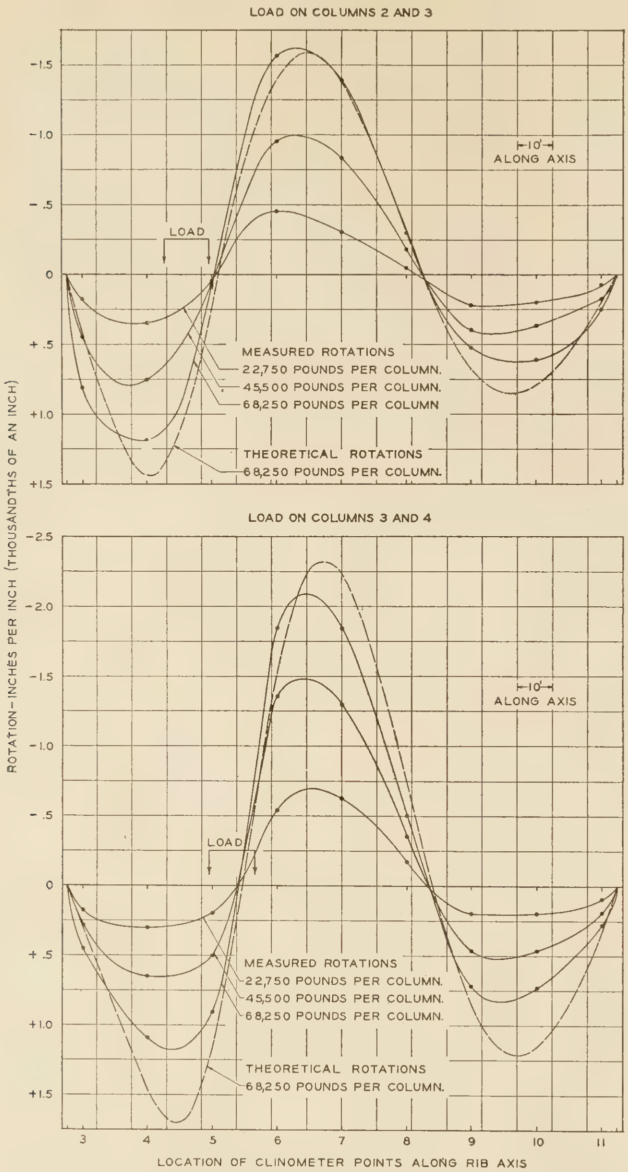


Fig. 24.—Rotations of Arch Rib Under 1-Tank Loading with Deck Cut (series 2)

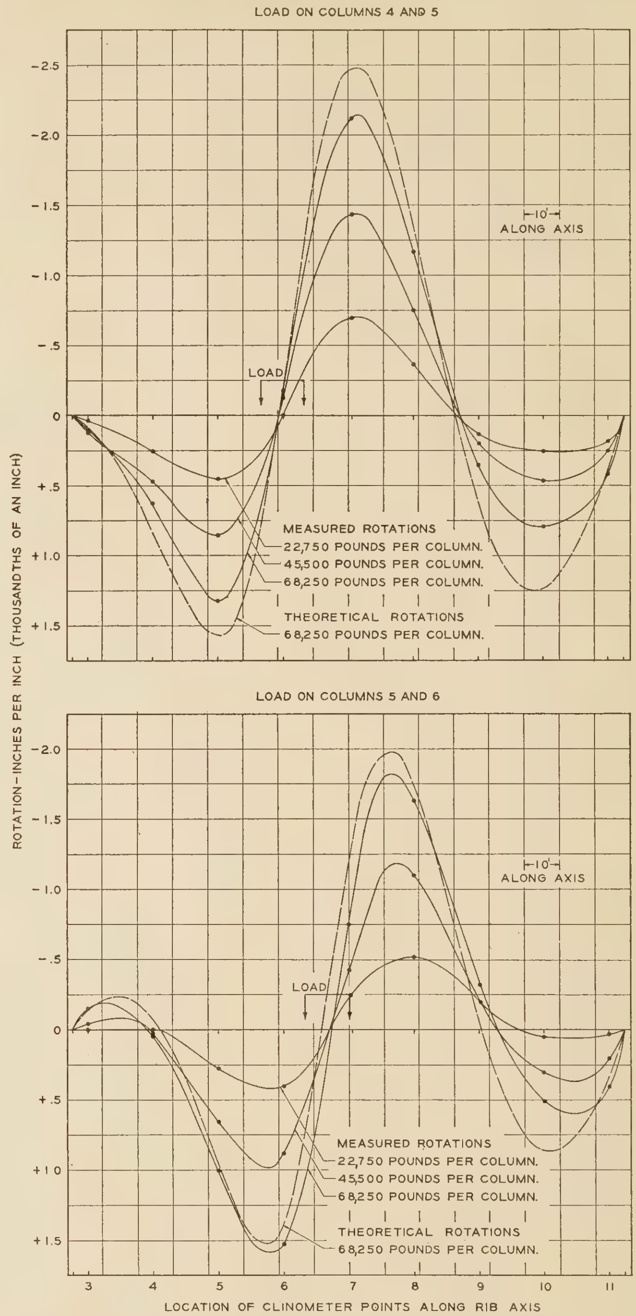


Fig. 25.—Rotations of Arch Rib Under 1-Tank Loading with Deck Cut (series 2)

the springing line because of high local stresses, the arch still continued to deform elastically.

ROTATIONS FROM MODEL ANALYSIS COMPARED WITH MEASURED AND THEORETICAL ROTATIONS

Moment diagrams for each condition of loading and each condition of rib restraint were made from the influence lines derived from the model analysis. Rotations were calculated from these moment diagrams, the rotation due to rib shortening caused by thrust being neglected. Curves in Figures 27 and 28 show comparisons between measured rotations for series 1 (curve E), the computed rotations (curve D) and those calculated from the model analysis for the three conditions of rib restraint (curves A, B, and C). These curves show the relation between the model analysis, for the two conditions of the superstructure, and the measured rotations. For the free rib, the agreement between computed results and the model analysis is very close.

It is felt that the rotations are the most significant of the measurements taken in the field because the instrumental work was subject to few inaccuracies and because they represent a summation of deformation and are not, therefore, as sensitive to sudden local changes in moment as in the case of stresses.

DEFLECTION OF THE RIB

Figures 29 and 30 show measured deflections for series 1 at all columns except those next to the piers and also the computed deflections. The stiffening effect of the superstructure is shown by the greatly reduced deflections. The effect of the curtain wall is indicated by the flattening of some of the curves where the curtain wall occurs.

The curves shown in Figures 31 and 32 give comparisons of measured and computed values of deflection for series 2. With the curtain wall removed and the effect

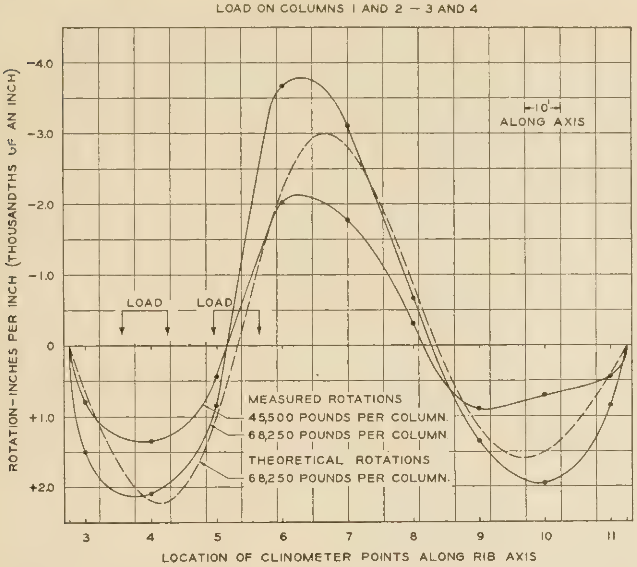


Fig. 26.—Rotations of Arch Rib Under 2-Tank Loading with Deck Cut (series 3)

of the deck reduced to a minimum, the two curves agree very closely.

Deflections for series 3 are shown in Figure 33. Here, as in the rotations for series 3, the maximum measured results are somewhat greater than the theoretical, but the curves are of almost identically the same form.

CHANGES IN LENGTH OF MIDORDINATES OF THE ARCH AXIS

Figures 34 and 35 show measured and computed values for the changes in midordinates of successive 5-foot arcs of the axis for series 1 and 2. The ordinates of these curves are, of course, proportional to the bending moment divided by the moment of inertia of the section.

TEMPERATURE DEFORMATIONS

Figure 36 shows measured rotations and deflections due to a temperature change of 1° C. compared with the computed values. A coefficient of expansion of 0.00001 per degree centigrade was used in these computations. Readings were taken with the load conditions constant at different rib temperatures and these readings were divided by the change in average rib temperature. All the values for a 1° change were averaged to obtain the plotted values.

The curves for series 1 and 2 check each other very closely, indicating that the superstructure in this particular case, has little effect on the action of the rib due to temperature changes. The curves for series 1 and 2 check the computed curve very well.

CONCLUSIONS

The following conclusions as to the action of a reinforced concrete arch appear to be justified by the data presented above:

(1) The action of an arch rib, when free from the restraining effect of the superstructure and supported by practically immovable piers, conforms closely to the action as determined by the generally accepted theory of elastic structures, even at high unit stresses over short lengths of the rib.

(2) The observed compressive stress in the concrete at any section of the rib checks the computed stress more closely when it is assumed that the concrete takes tension than when it is assumed that concrete is not active in resisting tensile stress, even where the high tensile stress causes cracks in the concrete.

(3) The observed tension in the steel was, in general, less than that computed by assuming the concrete to take no tension.

(4) The superstructure in an open-spandrel rib arch greatly reduces the deformation of the rib; the amount

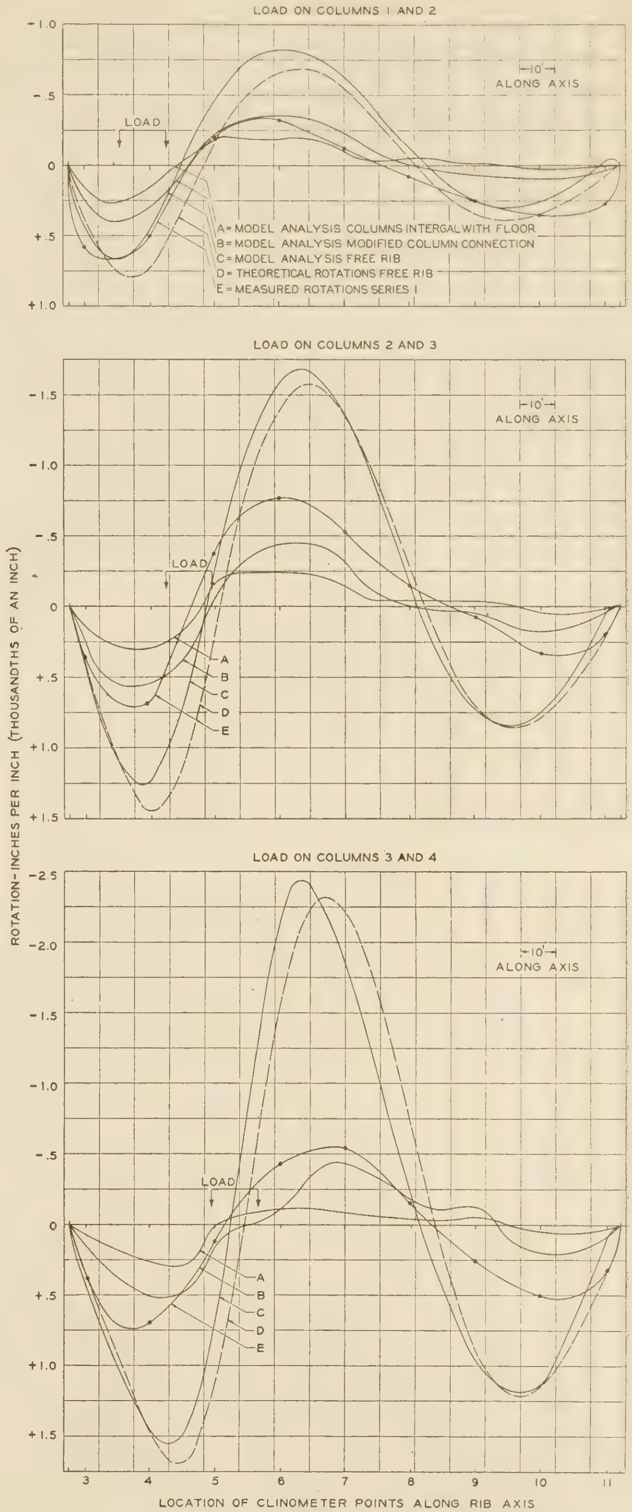


Fig. 27.—Rotations or Arch Rib Under 1-Tank Loading from Model Analysis Compared with Measured Rotations (series 1 and 2)

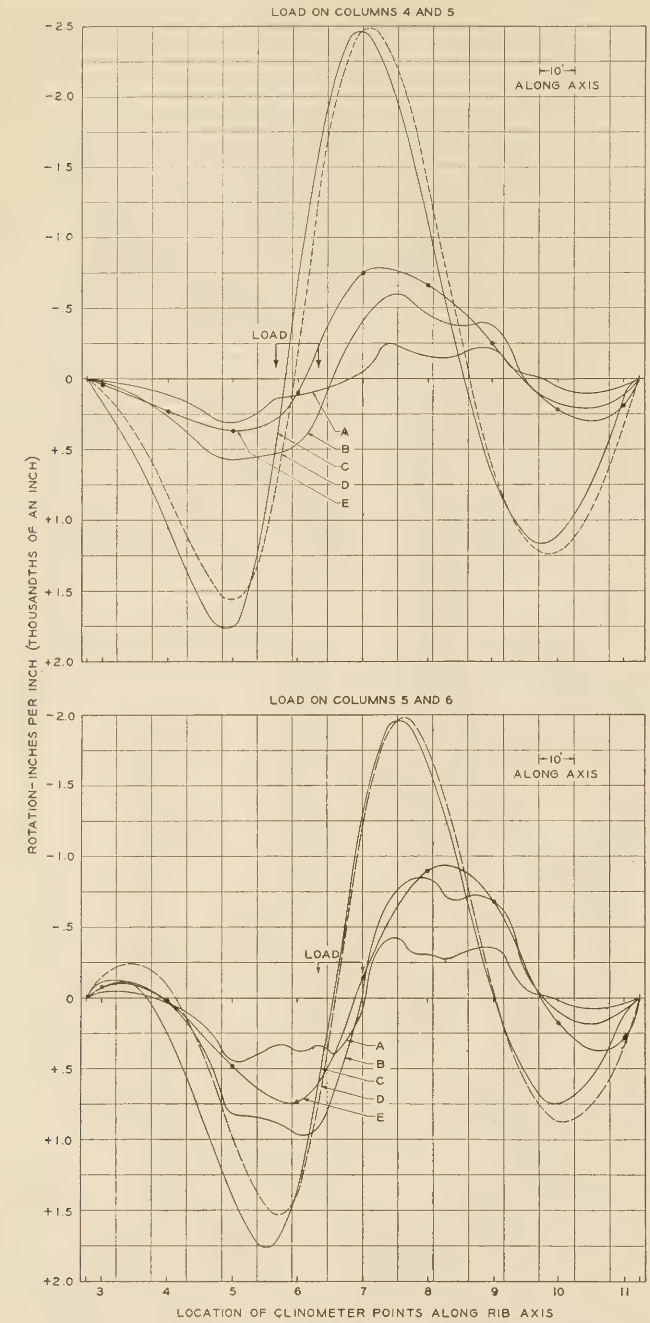


Fig. 28.—Rotations of Arch Rib Under 1-Tank Loading from Model Analysis Compared with Measured Rotations (series 1 and 2)

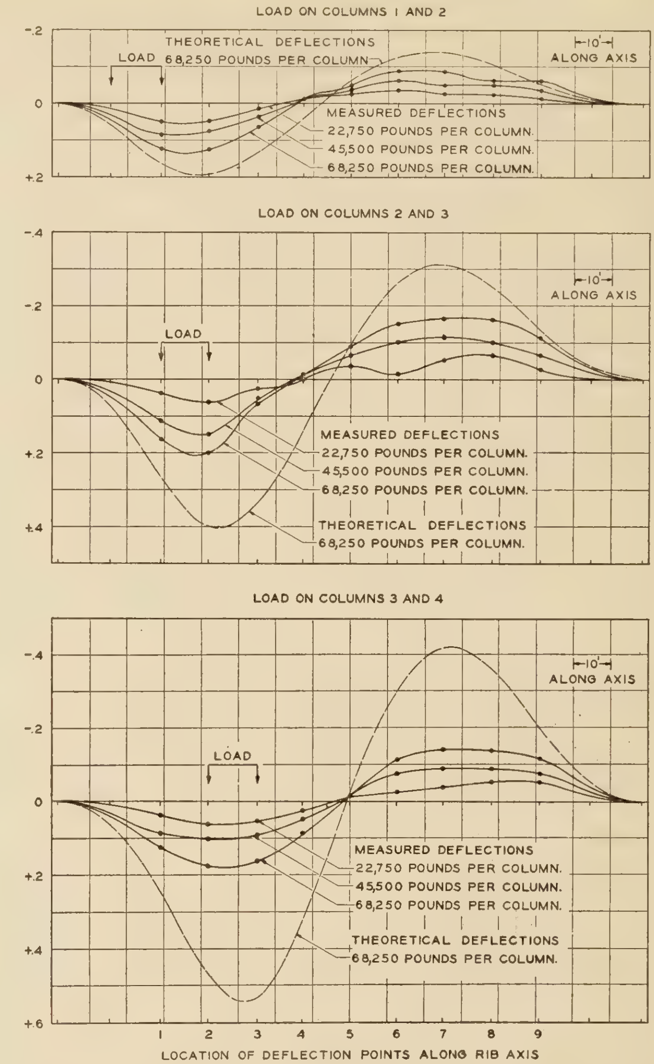


Fig. 29.—Deflections of Arch Rib Under 1-Tank Loading with Deck Intact (series 1) Compared with Theoretical Deflections

of reduction depending upon the degree of continuity of the floor system, the manner in which the floor system is attached to the tops of the spandrel columns, and the stiffness of the columns.

The use of expansion joints of the sliding type in the floor system of this bridge apparently did not prevent the coaction of the superstructure with the arch ribs

(5) A quantitative determination of the effect of the superstructure on the arch may be made by the use of the Beggs deformeter gauges on an elastic model of celluloid or some other suitable material, if all members are connected in a definite way. While it seems possible to get a reliable model analysis when the structure has frictional joints such as the floor bearings of the Yadkin River Bridge, there is no practical means of making a model which represents such joints with certainty.

(6) Temperature deformations of the rib appear to be independent of the superstructure for this particular arch.

APPLICATION OF CONCLUSIONS TO ARCH DESIGN DISCUSSED

These tests were concerned only with deformations produced by live loads applied over comparatively short periods of time. The conclusions are not, therefore, necessarily true of dead-load deformations due to continuous application of the load over long periods of time because of the possibility that the elastic properties of the concrete may be changed by a continuously applied stress.

It may be inferred from the first conclusion that, even though the dead-load stresses are kept down to a low value, the combined dead and live load stresses may be safely allowed to reach a much higher value, depending upon the quality of concrete which may be secured in the work.

Conclusions 4 and 5 indicate that, if full advantage is to be taken of the stiffening effect of the superstructure, a type of structure must be used which can be accurately represented by a model. In order to do this,

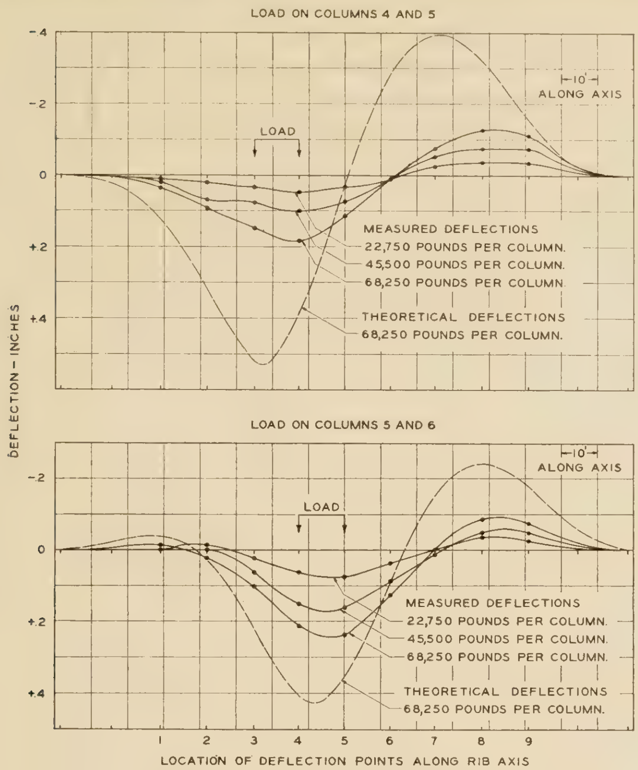


FIG. 30.—DEFLECTIONS OF ARCH RIB UNDER 1-TANK LOADING WITH DECK INTACT (SERIES 1) COMPARED WITH THEORETICAL DEFLECTIONS

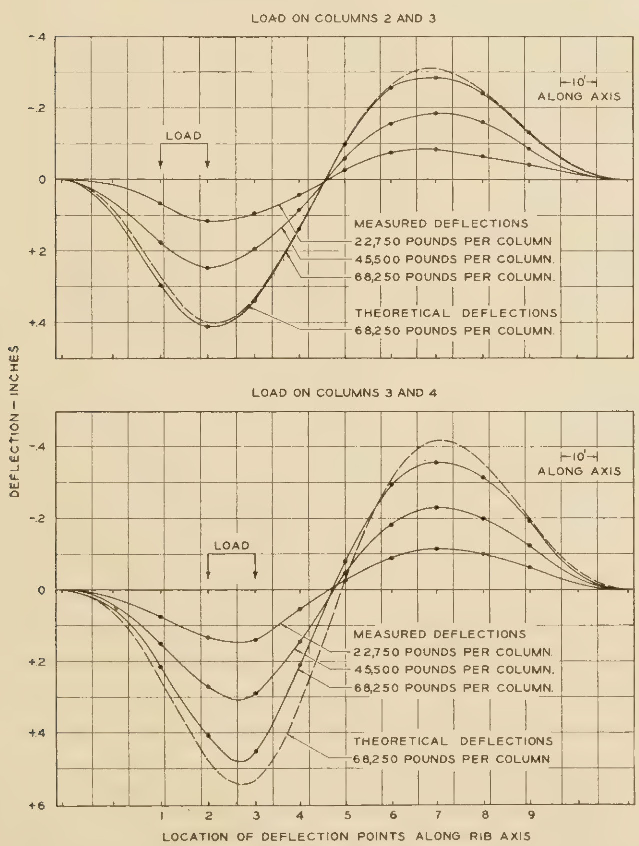


FIG. 31.—DEFLECTIONS OF ARCH RIB UNDER 1-TANK LOADING WITH DECK CUT (SERIES 2) COMPARED WITH THEORETICAL DEFLECTIONS

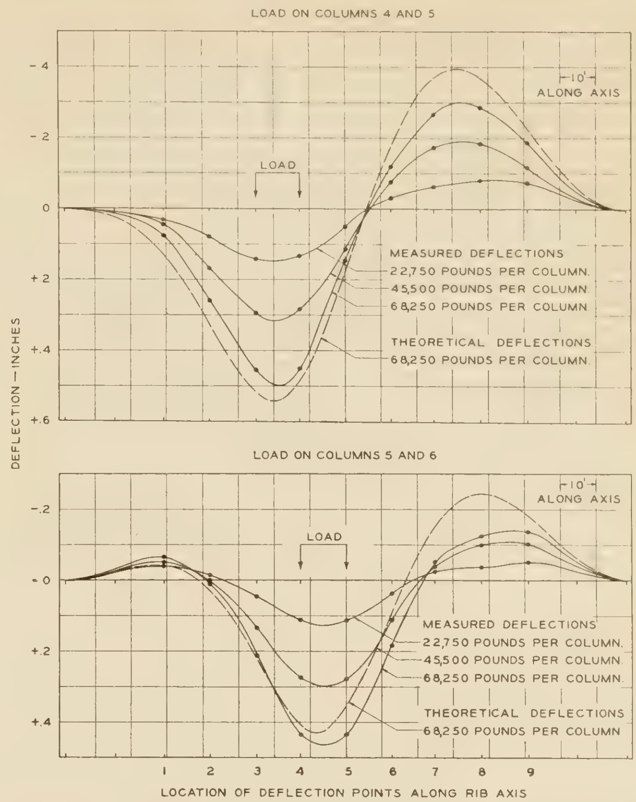


FIG. 32.—DEFLECTIONS OF ARCH RIB UNDER 1-TANK LOADING WITH DECK CUT (SERIES 2) COMPARED WITH THEORETICAL DEFLECTIONS

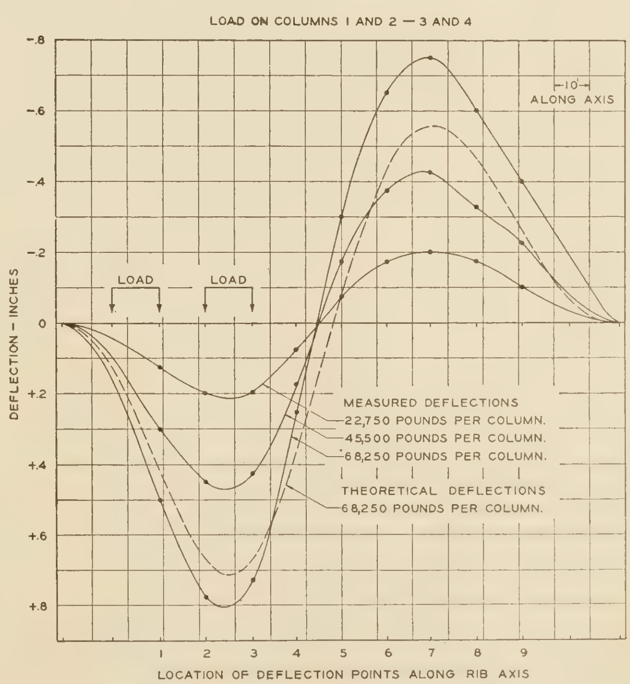


FIG. 33.—DEFLECTIONS OF ARCH RIB UNDER 2-TANK LOADING WITH DECK CUT (SERIES 3) COMPARED WITH THEORETICAL DEFLECTIONS

sliding joints should be eliminated from the structure as far as practicable and definite connections made between all members. It should be pointed out, however, that, if full advantage is taken of the reduction of rib stresses due to the stiffening effect of the superstructure, a

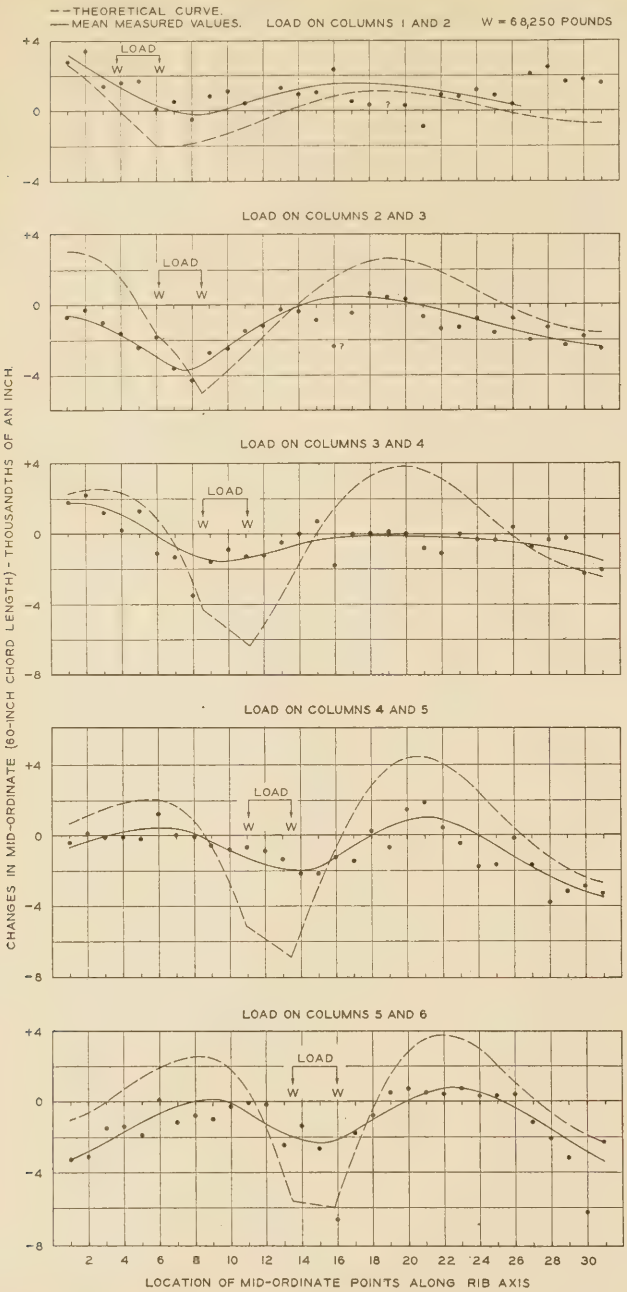


Fig. 34.—Measured and Computed Mid-Ordinate Changes Under 1-Tank Loading with Deck Intact (series 1)

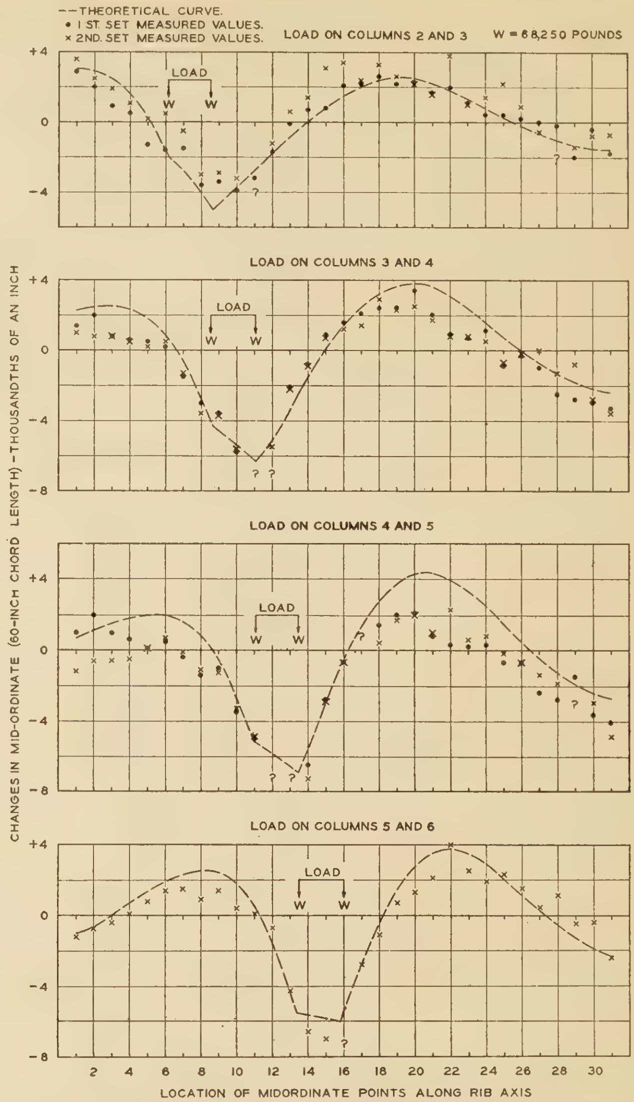


Fig. 35.—Measured and Computed Mid-Ordinate Changes Under 1-Tank Loading with Deck Cut (series 2)

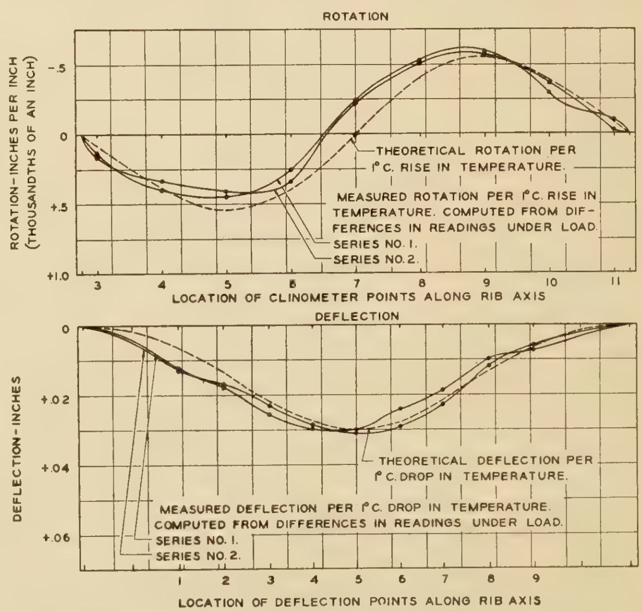


Fig. 36.—Temperature Curves—The Measured Curves Represent the Averages of All Differences in Readings Under the Same Loading Divided by the Difference in Average Temperature of the Arch Rib

complete investigation should be made of the accompanying increase in the stresses in the spandrel columns.

DEFORMATION MEASURING INSTRUMENTS DISCUSSED

Of all the instruments used in this test, the clinometer is probably the most dependable because of its simplicity of construction and its comparative immunity to temperature effects. However, it is felt that, when proper provisions for temperature corrections are made, the electric telemeter gives satisfactory results when relatively large deformations occur but it is difficult to make proper provision for temperature corrections under field conditions.

It seems that the radiusmeter is subject to some inaccuracies in measuring very small quantities such as were measured in these tests.

Deflections measured from a piano wire under high tensile stress proved to be very satisfactory.

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Report of the Chief of the Bureau of Public Roads, 1925.
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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

NOVEMBER 30, 1928

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$				$	$				$	
Alabama	1,831.0	4,584,448.50	2,287,818.07	308.0	31.3	339.3	166,068.00	83,033.99		21.1	21.1	1,333,701.63	Alabama
Arizona	884.1	1,414,493.94	1,198,546.43	66.9	2.4	69.3	89,976.13	65,088.70	6.1	5.6	11.7	2,679,482.82	Arizona
Arkansas	1,748.7	3,746,897.39	1,658,744.63	102.4	6.2	108.6	84,989.12	42,494.55	10.9	.3	11.2	1,734,235.64	Arkansas
California	1,528.7	7,055,304.08	3,191,753.01	170.2	6.7	176.9	1,648,363.89	931,700.03	57.8		57.8	1,471,672.82	California
Colorado	987.7	5,358,767.04	2,725,212.00	191.4	31.2	222.6	662,647.88	364,639.65	35.7	.9	36.6	1,738,068.32	Colorado
Connecticut	218.4	1,985,798.70	444,766.53	23.4		23.4	667,713.84	279,338.73	3.6		3.6	360,414.44	Connecticut
Delaware	203.2	499,461.60	178,885.54	12.9	2.1	15.0	230,055.00	100,211.50	2.7		2.7	95,265.72	Delaware
Florida	429.6	2,318,953.51	1,000,227.20	68.5	5.4	73.9	673,317.04	336,658.50	42.3		42.3	1,146,622.52	Florida
Georgia	2,422.3	6,316,349.85	2,846,469.39	261.4	65.9	327.3	336,794.44	151,746.62	18.2	5.9	24.1	14,619.27	Georgia
Idaho	1,042.8	2,331,048.79	1,386,474.66	161.1	12.6	173.7	61,000.00	35,000.00	6.3		6.3	67,181.31	Idaho
Illinois	1,766.7	20,601,984.92	9,386,774.04	644.5		644.5	1,480,814.25	628,151.94	41.5		41.5	22,458.19	Illinois
Indiana	1,173.0	7,711,554.04	3,655,212.77	227.1	3.5	230.6	1,599,601.92	706,853.04	51.4		51.4	14,637.76	Indiana
Iowa	2,925.5	4,814,060.85	1,977,925.47	70.6	125.5	196.1	595,767.19	231,211.52	3.7	35.0	38.7	190,086.34	Iowa
Kansas	2,324.6	5,506,588.62	2,165,474.96	337.2	12.0	349.2	1,740,647.40	701,586.75	81.4	10.9	92.3	98,318.38	Kansas
Kentucky	1,224.6	5,344,536.82	2,461,674.11	227.0		227.0	1,099,366.03	408,123.32	72.4		72.4	25,201.43	Kentucky
Louisiana	1,279.9	4,780,221.55	2,382,092.27	194.5		194.5	103,055.30	25,000.00	.1		.1	182,706.36	Louisiana
Maine	442.1	2,181,064.52	758,238.55	51.8		51.8	929,217.40	366,207.32	30.8		30.8	824,136.41	Maine
Maryland	557.6	1,742,202.42	799,750.00	70.4	7.2	77.6						30,596.23	Maryland
Massachusetts	532.7	4,140,586.99	1,348,323.61	81.5		81.5	300,402.68	114,240.00	7.6		7.6	1,436,596.38	Massachusetts
Michigan	1,426.1	11,462,789.54	4,837,765.49	299.2		299.2	895,652.35	344,965.50	23.1		23.1	143,804.45	Michigan
Minnesota	3,970.0	3,464,135.80	1,083,947.49	172.7	75.5	248.2	1,106,143.26	15,000.00	22.9		22.9	34,193.96	Minnesota
Mississippi	1,592.4	5,133,593.12	2,335,153.53	211.4	31.6	243.0	459,205.47	229,015.79	42.4	.9	43.3	171,696.40	Mississippi
Missouri	2,265.8	5,038,528.78	2,179,136.59	156.6	44.8	201.4	1,094,576.62	509,404.70	39.6	9.9	49.5	455,147.99	Missouri
Montana	1,455.2	3,850,397.05	2,346,215.45	293.7	16.3	310.0	837,704.43	445,320.49	87.5	1.3	88.8	3,942,504.17	Montana
Nebraska	3,422.3	4,161,204.87	2,059,263.17	391.1	105.1	496.2	187,314.71	93,657.33	11.1	19.0	30.1	1,805,882.54	Nebraska
Nevada	1,004.9	1,293,822.52	1,117,209.34	133.6	66.4	200.0	181,620.00	161,341.93	34.5	1.6	36.1	155,584.42	Nevada
New Hampshire	322.3	644,874.62	249,184.96	16.9		16.9						33,298.16	New Hampshire
New Jersey	436.8	5,628,563.60	973,962.35	66.9		66.9	108,429.58	33,975.00	2.3		2.3	8,195.94	New Jersey
New Mexico	1,777.7	3,487,326.44	2,265,852.55	210.2	.5	210.7	288,676.61	182,432.03	23.2	5.1	28.3	144,428.48	New Mexico
New York	1,945.8	31,318,700.00	7,125,605.00	470.6		470.6	5,098,031.43	995,505.00	66.4		66.4	3,356,032.88	New York
North Carolina	1,607.7	2,543,289.74	1,238,264.71	93.9	33.2	127.1	732,486.55	362,353.17	35.9	7.2	43.1	343,455.26	North Carolina
North Dakota	3,473.3	3,228,686.79	1,409,746.21	485.8	224.3	710.1	984,291.23	355,842.13	128.8	106.0	234.8	65,108.86	North Dakota
Ohio	1,887.8	13,344,421.46	4,920,098.69	300.9	.2	301.1	2,899,987.46	842,298.73	44.5	13.0	57.5	1,301,317.76	Ohio
Oklahoma	1,684.1	3,338,701.62	1,533,214.65	138.5	14.3	152.8	1,138,547.62	489,148.90	55.7	16.5	72.2	137,252.27	Oklahoma
Oregon	1,121.5	1,311,749.94	752,299.94	42.8		42.8	146,074.50	90,062.99	.4		.4	1,161,077.81	Oregon
Pennsylvania	1,969.1	13,269,047.94	3,719,210.46	228.3		228.3	3,302,142.73	907,374.99	53.4	14.1	67.5	381,055.27	Pennsylvania
Rhode Island	159.6	568,459.26	131,835.00	8.8		8.8	112,609.50	43,974.55	1.5		1.5	537,574.23	Rhode Island
South Carolina	1,692.1	7,498,857.18	1,630,114.71	173.3	98.6	271.9	286,235.20	51,000.00	1.4		1.4	15,660.90	South Carolina
South Dakota	3,170.7	2,529,390.42	1,376,677.82	461.0	63.8	524.8	327,108.90	179,909.84	31.0	12.3	43.3	70,935.29	South Dakota
Tennessee	1,048.3	5,733,814.47	2,512,004.20	149.2	39.7	188.9	1,606,405.82	588,133.07	3.0	54.6	57.6	490,645.20	Tennessee
Texas	6,075.2	9,390,415.21	3,788,002.15	149.8	181.1	330.9	6,897,428.76	2,809,852.24	401.2	114.7	515.9	1,921,067.86	Texas
Utah	881.2	1,412,956.81	936,862.66	67.6	4.0	71.6	296,847.21	215,380.61	21.3		21.3	50,558.98	Utah
Vermont	227.2	1,083,457.34	326,408.03	22.2		22.2						44,310.95	Vermont
Virginia	1,275.0	4,445,014.64	1,424,533.49	117.2	21.6	138.8	934,538.70	166,209.94	29.9		29.9	1,542.70	Virginia
Washington	808.6	4,240,151.42	1,405,531.52	85.3	23.1	108.4	187,933.29	100,800.00	9.8		9.8	447,748.83	Washington
West Virginia	666.3	1,789,404.45	785,844.09	67.6	2.5	70.1	644,522.68	297,279.71	15.6	9.9	25.5	302,087.24	West Virginia
Wisconsin	2,153.8	7,424,144.13	2,772,017.47	204.9	29.7	234.6	603,638.80	213,808.00	24.4		24.4	969,349.56	Wisconsin
Wyoming	1,569.2	1,874,676.63	1,123,089.20	201.3	10.1	211.4						10,047.90	Wyoming
Hawaii	39.4	64,548.41	32,274.20	.1		.1	111,383.58	25,227.00	1.7		1.7	1,066,498.59	Hawaii
TOTALS	74,652.6	253,009,448.33	100,246,688.36	8,692.2	1,398.4	10,090.6	43,939,334.50	16,320,559.80	1,685.0	465.8	2,150.8	33,034,066.82	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.