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Load and Destruction Test of 160-Ft. Girder Designed for First Prestressed Concrete Bridge in U. S. A.

Belastungs- und Bruchversuche an einem Träger von 160' Länge für die erste vorgespannte Eisenbetonbrücke in den Vereinigten Staaten

Essais de charge et de rupture sur une poutre de 49 mètres destinée à la construction du premier pont en béton armé précontraint aux Etats-Unis

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1. Introduction

The first prestressed concrete bridge in the U.S.A. (the Walnut Lane Bridge), is now being constructed 40 feet above and over the Lincoln Drive in Fairmount Park in Philadelphia, Pennsylvania. A competitive conventional design consisted of a 150-ft. open spandrel arch bridge which would have cost some \$ 300000 more than the prestressed design and the construction of which would have lasted many months longer. The bridge is being built for the Department of Public Works of the City of Philadelphia of which Thomas Buckley is Director. The work is under the direction of the Bureau of Engineering, Surveys and Zoning. A. Zane Hoffman is Chief Engineer, Samuel A. Baxter, Assistant Chief Engineer, and E. R. Schofield, Principal Assistant Engineer, now retired.

The design of this bridge was made in accordance with the Belgian Magnel-Blaton-System of prestressing for which Preload Enterprises, Inc., in New York, is exclusive licensee for the U.S.A. Professor Gustave Magnel of the University of Ghent acted as Consulting Engineer to Preload Enterprises. General Contractor for the construction of the bridge is the Henry W. Horst Company of Philadelphia with The Preload Corporation of New York as Subcontractor for the construction and the placing of the prestressed girders. The strain gage measurements were made for The Preload Corporation by Mr. Arthur R. Anderson, Consulting Engineer.

2. General Description

The bridge consists of a central single span of 160 feet and two single approach spans of 74 feet each (Fig. 1). Expansion joints are arranged over the piers and at the west abutment with "Lubrite" sliding plate supports at one end of the girders and fixed steel plate hinges at the other.

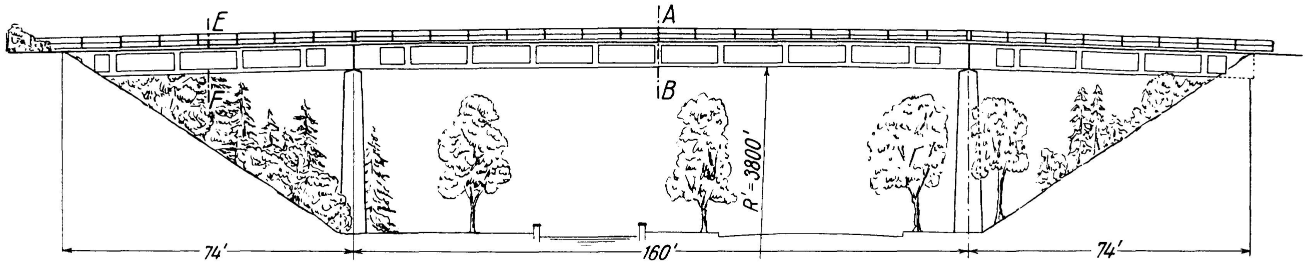


Fig. 1. General View

The cross section of the 13 main span girders is a 6′—7″ deep "I" shape (Fig. 2). This type of cross section lends itself to the economical application of "post stressing" methods as well as to a clear distinction between general concrete construction work (road slab, walkways, etc.) and the prestressed girders proper.

Economic and aesthetic considerations led to the adoption of a similar cross section also for the 7 girders of each approach span (Fig. 2).

A typical longitudinal section of the main span girder is shown in Fig. 2.

3. Construction Procedure

A short description of actual construction procedure will explain the consideration in the design of a series of loading stages, each of which deserves the adoption of certain design criteria in accordance with its temporary or permanent nature.

After the foundations, piers and abutments were completed by the General Contractor, the scaffolding to below girder soffit was erected. Two sets of formwork for alternate pouring of two girders every 28 days were erected at the extreme downstream side of the scaffolding (Fig. 3). After curing for approximately 14 days each girder was prestressed and moved sideways into its final position on specially prepared transverse sliding tracks under the girder ends. Subsequently the prestress anchor plates at both ends were covered, according to plans, with poured concrete, thereby reducing the gap between main span and approach span girders to 4 inches.

Transverse stiffener wings on either side of the main span girders were poured monolithically with the girder. For the approach spans, however, these

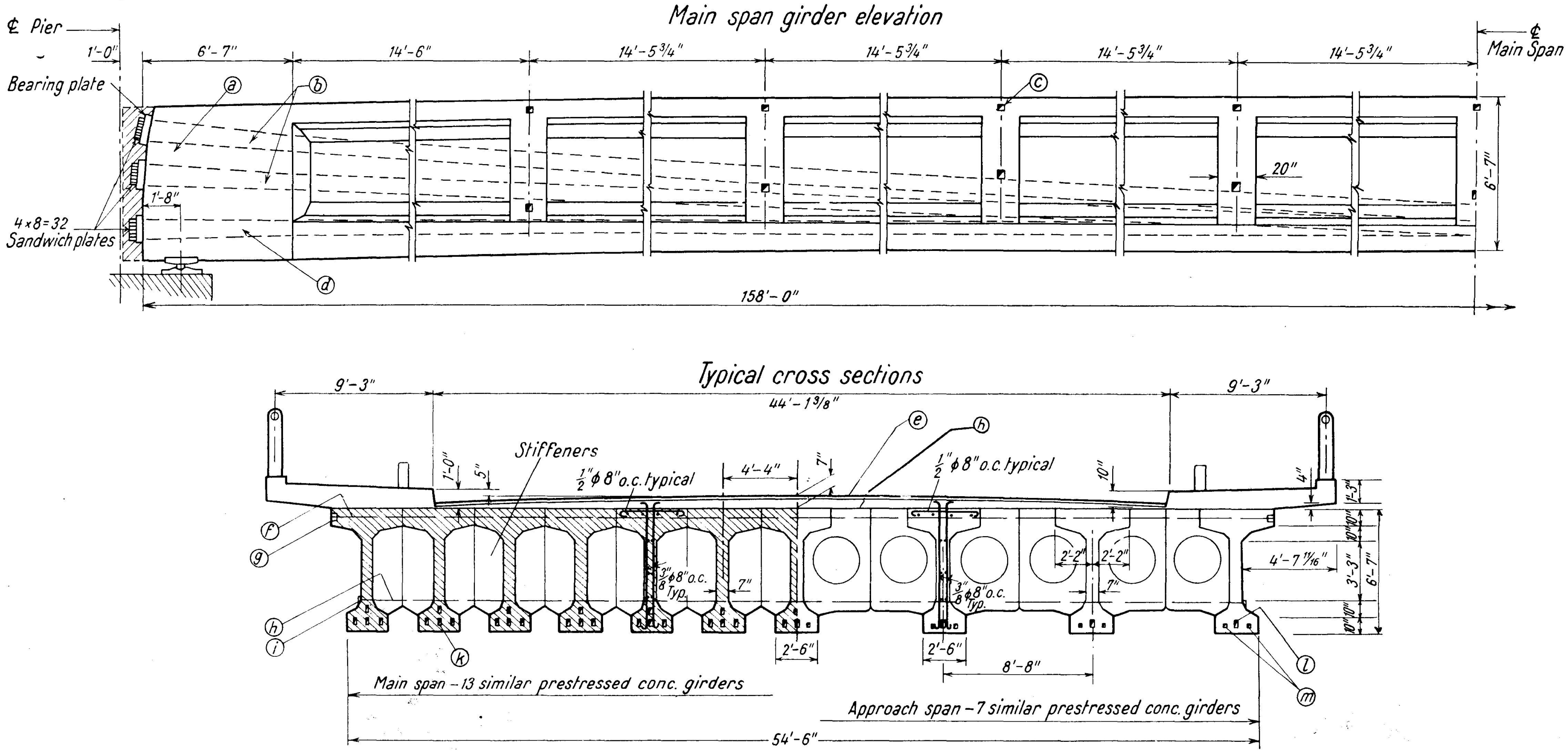


Fig. 2. Typical Cross Sections and Elevation

a = standard steel reinf. not shown.
b = 2-curved prestressing cables of 64 wires 0,276″ dia. each.
c = 8 Wires 0,276″ dia. at each stiffener.
d = 2-straight prestressing cables of 64 wires 0,276″ dia. each.
e = I″ asphalt plank paving.
f = one transverse prestressing cable of 8 wires 0,276″ dia. per stiffener.
g = one sandwich plate grouted in.
h = one transverse prestressing cable of 16 wires 0,276″ dia. per stiffener.
i = two sandwich plates grouted in.
k = four prestressing cables of 64—0,276″ dia. wires each for each main span girder.
l = one prestressing cable of 48—0,276″ dia. wires } for each approach span girder.
m = two prestressing cables of 24—0,276″ dia. wires each } for each approach span girder.
n = concrete paving base.

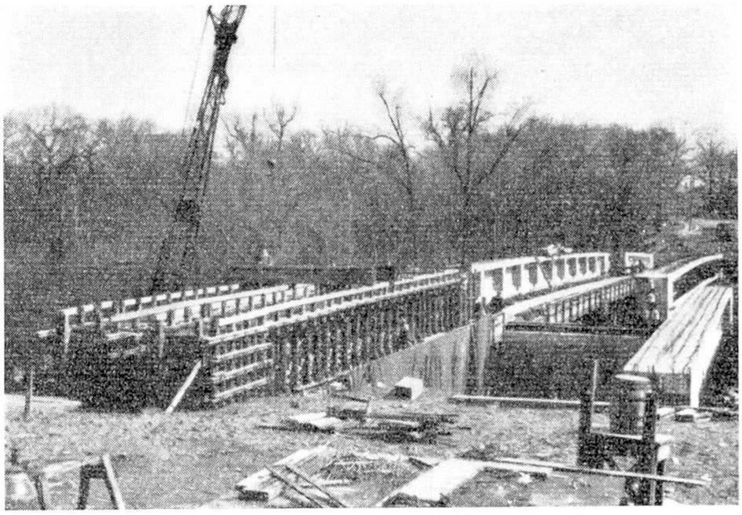


Fig. 3. Formwork, General View

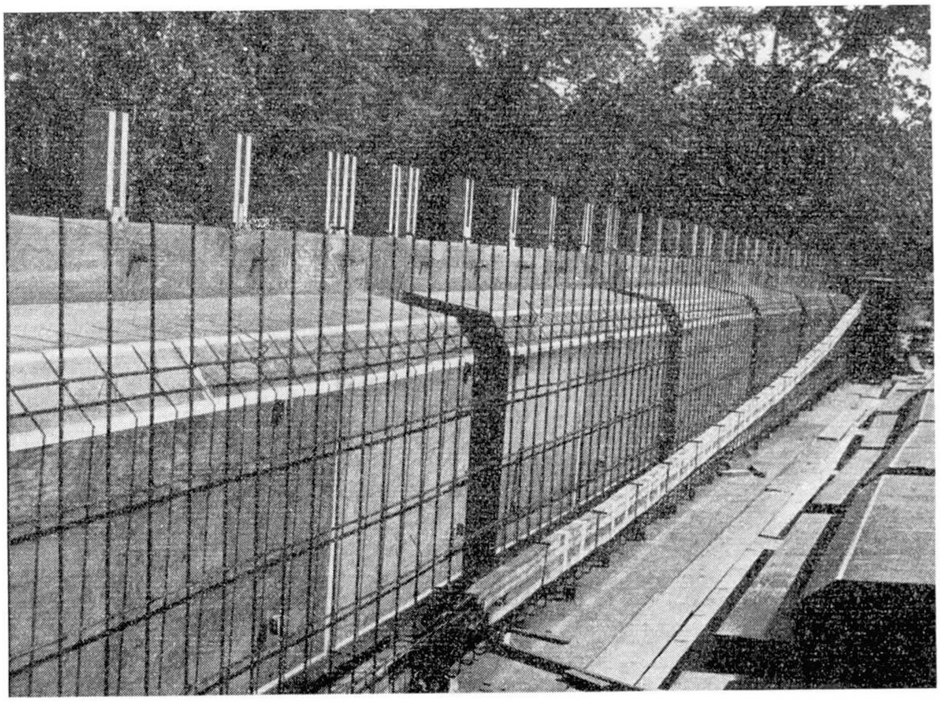


Fig. 4. Formwork Details, Reinforcing Steel and Rubber Cores

stiffeners were poured in situ after girders were in their final position. The 1″ wide joints between main span stiffener wings were filled with cement mortar before proceeding with the transverse cable prestressing operation at top and bottom of the stifferners.

A typical prestressed girder was fabricated as follows:

The downstream side of the girder formwork was erected for its whole length. Rubber cores for the formation of longitudinal cable holes (see Fig. 4) were then installed and secured in place in such a manner that they could be withdrawn after setting of the concrete. Standard vertical web reinforcing and special reinforcing in the end of the girders was placed and the upstream side of the formwork installed.

Fig. 5. Cable Assembly and Comb Spacers

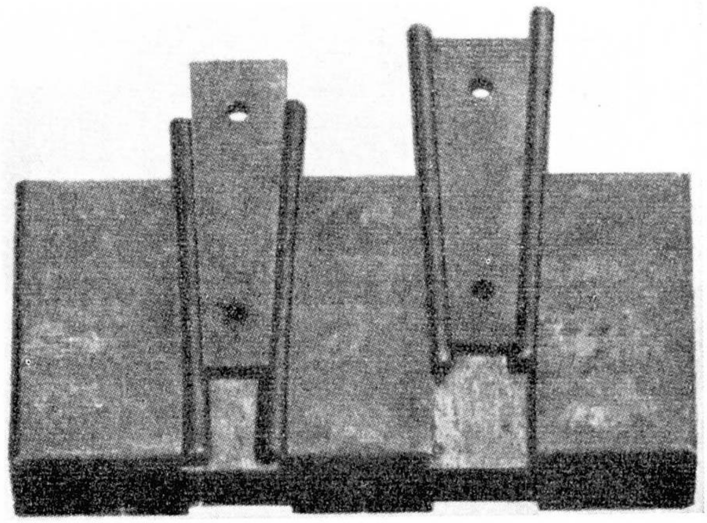


Fig. 6. Sandwich Plate

The girders were poured with an $8^1/_2$-bag mix of the following proportions per c.y.: Type I cement 799 lbs., fine aggregate 1010 lbs. and $^3/_4''$ gravel coarse aggregate 2020 lbs. $^3/_4$ lb. of Plastiment per bag of cement was added. The average slump of the mix was 2 inches with a water content of 4,0 gallons per sack of cement. Both internal and external vibration was used throughout. The average 28-day strength of the concrete was approximately 6500 psi.

Simultaneously with the pouring of the girders the assembly of the longitudinal prestressing cables was accomplished on special assembly benches with special "comb spacers" (Fig. 5). These cables consist of 24 to 64 parallel 0,276″ diameter high carbon steel wires. They were inserted in the longitudinal holes in the girders by a special jacking arrangement.

Actual prestressing operations were undertaken when the concrete had reached a cylinder crushing strength of approximately $^3/_4$ of its 28-day strength. 2 wires are stretched at a time to a stress and elongation slightly higher than the specified initial value to allow for slight slippage in the so-called "sandwich plate" anchor wedge (Fig. 6) and due to successive elastic shortening of the girder. With a specially designed jack this operation takes but 1 to 2 minutes per wire pair or about $1^1/_2$ hours per 48 wire cable. The total force per cable is approximately 240 tons. It is distributed and transferred to the concrete by cast steel anchor plates $16'' \times 12^1/_2''$.

The last operation before moving the girder into position is the pressure grouting of the longitudinal cable holes in order to establish bond and protect the wires. Experience shows that girders with bonded prestressed wire have a higher cracking limit and a better crack distribution than those with unbonded cables.

4. Design and Load Requirement

The working live load was specified as a standard H-20-44 load pattern (American Association of State Highway Officials Standard) including an addition for impact of 17%. This load pattern consists of a uniformly distributed load of 640 lbs. per foot of traffic lane and a moving concentrated load of 18000 lbs. for moment and 26000 lbs. for shear.

According to the construction procedure described above each girder is subject to the following loading stages:

a) Before prestressing the dead load of the girder rests on formwork and scaffolding. After completion of prestressing the girder deflects upward and acts as a simple beam with the dead load transferred to end supports.
b) After installation of the girder in its final position further dead load is added (road slab, sidewalks) which acts as permanent live load as far as girder stresses are concerned.
c) In its completed condition the bridge is ready to be loaded with real live load and impact as per H-20-44.

While an important portion of the shrinkage has occurred before the prestressing operation, it will nevertheless slowly continue to shorten the girder. Also the known phenomena of "plastic flow" in concrete as well as the so-called "creep" in the steel wires tend to reduce the initial wire stress. However, after a certain time a state of permanent equilibrium is reached which is called the "ultimate" stress condition. In this condition all wire stresses are less than their initial value while compressive concrete stresses in the top fibre have been increased to an amount which should now be below the specified allowable value.

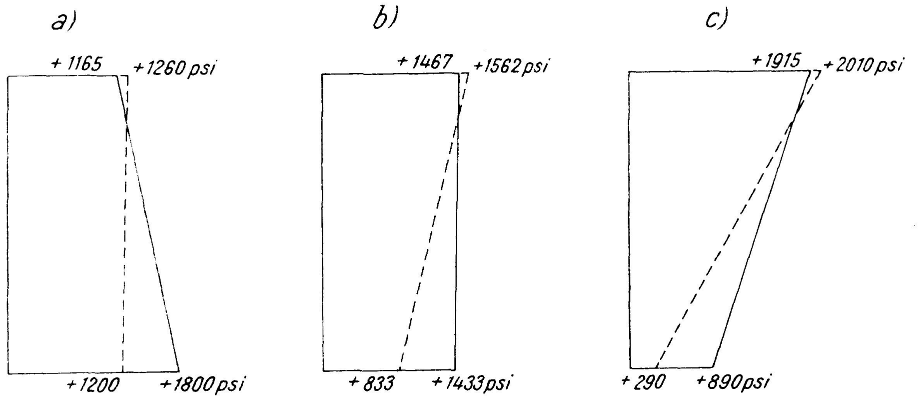


Fig. 7. Typical Stress Diagrams for Midspan Section
Full lines: „Initial" condition   Dotted lines: „Ultimate" condition
Positive stress indicates compression

Typical calculated stress diagrams at midspan for the 3 loading stages a) to c) inclusive are shown in Fig. 7 in full lines for "initial" condition and in dotted lines for the "ultimate" condition as described above.

The owner's Specifications required a minimum concrete cylinder crushing strength at 28 days of 5400 psi. Actual average 28-day cylinder strength obtained up to March 6, 1950 was 6500 psi.

The prestressing wire strength requirements prescribed by the owner were as follows:

Minimum yield strength
(0,2% offset) 160 000 psi
Minimum ultimate
strength 210 000 psi

The wire actually used for construction exceeded these minimum requirements considerably as can be noted from the stress strain diagram in Fig. 10. The actual 0,2% yield strength is in the neighborhood of 210000 psi and the ultimate strength

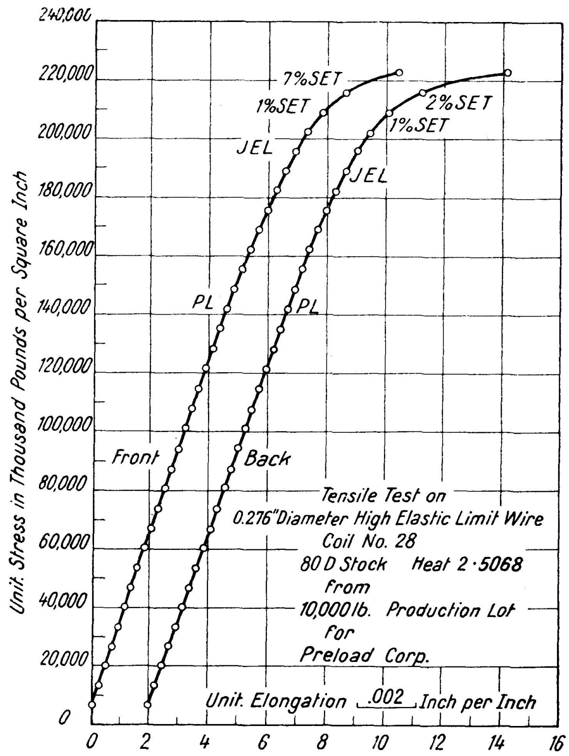


Fig. 8
Stress Strain Curve of Roebling 0,276" Wire

is approximately 245000 psi. The fact that the design was made for a lower quality wire than the one actually furnished by the contractor is the reason the test girder failed due to crushing of the concrete rather than due to breaking of the wire.

The owner's Engineers adopted conventional working stress limitations in accordance with the minimum material strength requirements specified above.

These stress limitations were as follows:

Prestressing Wire:	
Maximum initial stress	126 000 psi
Prestressing Wire:	
Maximum working stress after losses due to shrinkage and plastic flow	110 000 psi
Concrete:	
Maximum intial compressive stress (Dead load only)	2 150 psi
Maximum compressive working stress after losses due to shrinkage and plastic flow (for total load)	1 900 psi
Maximum principal tensile stress	215 psi
Initial tensile stress in top fibre (for dead load only); no tension allowed	0 psi
Minimum residual compressive stress in bottom fibre under total load after all losses due to shrinkage and plastic flow	50 psi
Maximum initial local compressive stress under end anchor plates	3 000 psi

The girder ends have the important function of safely distributing and transferring concentrated forces from supports and from prestressing anchors to the regular girder section. However, the scope of this paper does not permit a detailed demonstration of these design calculations.

5. Loading and Destruction Test of 160-Ft. Prestressed Girder

Valuable technical information was gained by the testing to destruction of a typical 160-ft. girder. This test was a requirement of the Specifications but also served to demonstrate the conservative margin of safety and the behaviour of the girder under working and failure loads. The tests were conducted at the bridge site in the presence of some 500 Engineers from all parts of the United States.

For several days before the final test, concrete strains, deflections and end slopes were measured and recorded under various loads in accordance with a detailed testing program as described later on.

a) *Instrumentation*

A total of 22 external electric strain gages Type SR 4 (Baldwin) were attached to the girder and waterproofed (Fig. 9) at locations as shown in Fig. 10. All strain gages were connected by separate leads to two Anderson Model 301 Strainmeters housed in a special instrument shelter. This shelter also contained a concrete test cylinder with a compensator gage for the elimination of temperature influences. (Fig. 11). This arrangement permitted the recording of concrete strains in 10^{-6} units or micro inches per inch.

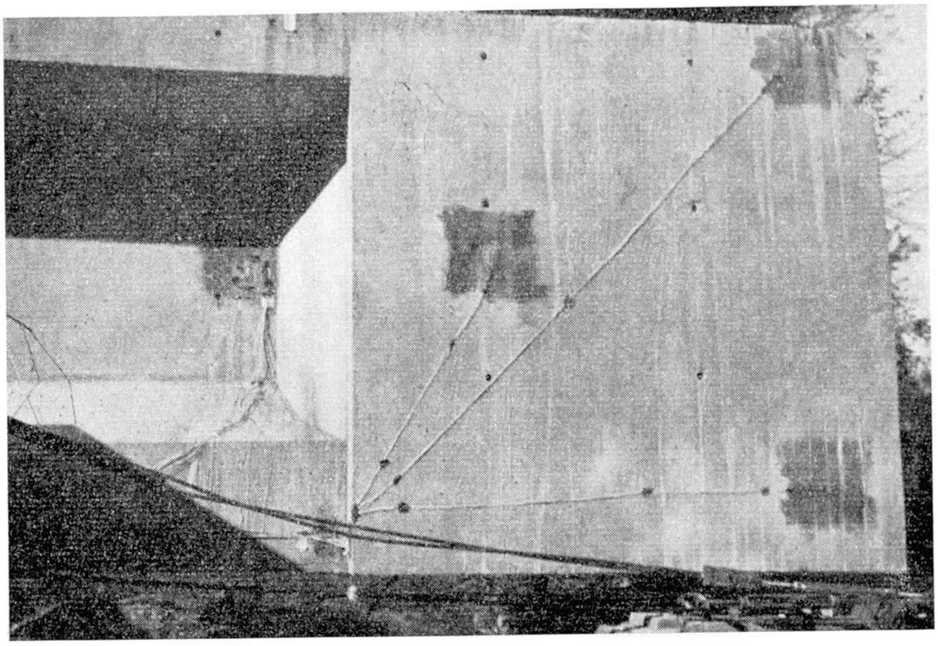


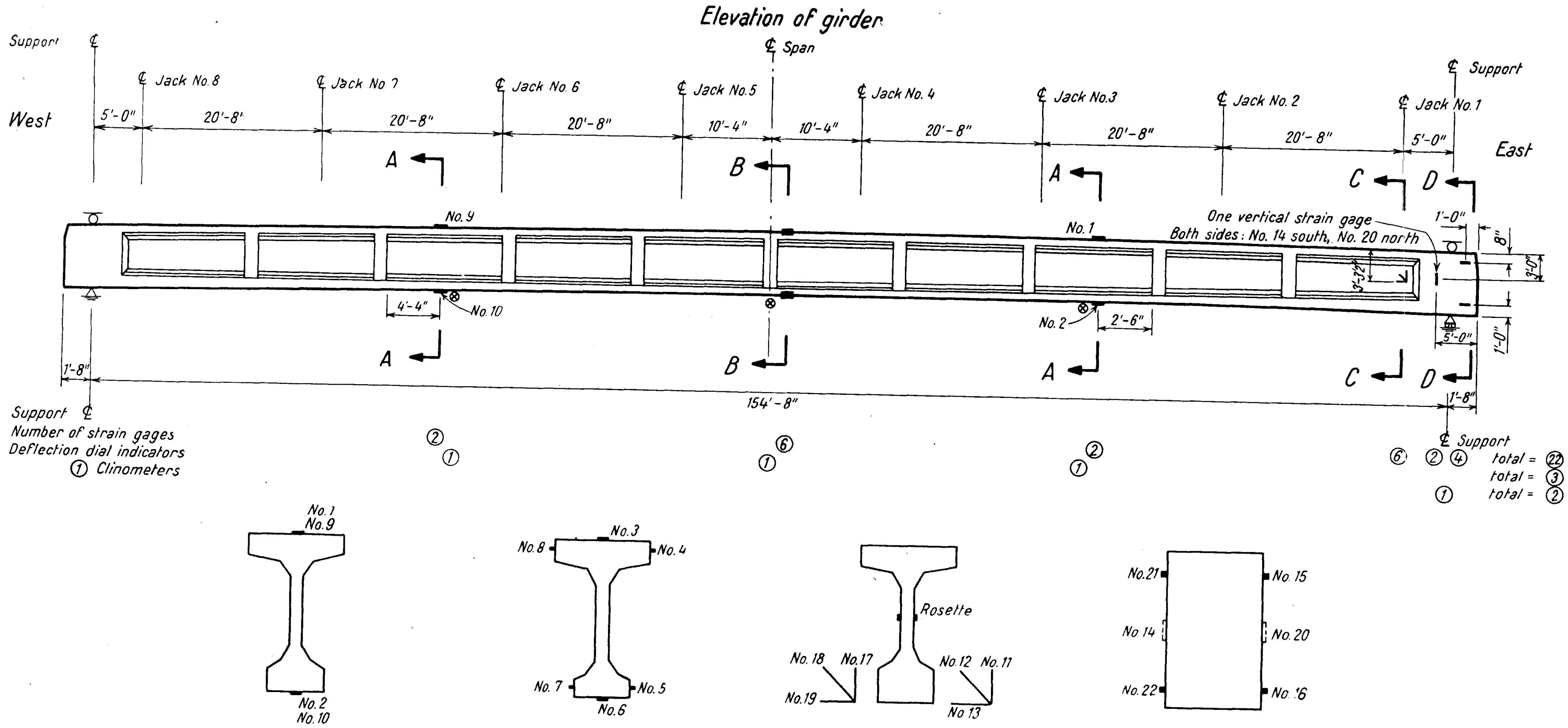
Fig. 9. Strain Gages and Leads at End of Test Girder
East end of girder, looking north, showing strain gage installation on concrete

Vertical deflections were measured with dial indicators at midspan and at both quarterpoints during the load tests and in addition at a point 8 feet from the East end of the girder during the prestressing operation.

For the determination of the angular turn of the ends of the test girder clinometers known as Gunners Quadrants were used at each end. These instruments made possible the recording of angular turns to the nearest one-half minute after application of each load increment.

b) *Load Application*

The test girder was poured on September 21, 1949 and prestressed during the week of October 7 to October 13, 1949. The total of 256 wires were stretched in pairs to an elongation of $8^7/_8$ inches which is sufficient to result in an initial stress in their final anchored condition of approximately 134000 psi. In the



Section A-A	*Section B-B*	*Section C-C*	*Section D-D*
$2 \times 2 \times 1 = 4$ Strain Gages	$2 \times 2 = 4$ Strain Gages	$2 \times 3 = 6$ Strain Gages	$2 \times 2 = 4$ Strain Gages (Horiz'l.)
	$2 \times 1 = \frac{2}{6}$ Strain Gages		$2 \times 1 = \frac{2}{6}$ Strain Gages (Vert'l.)

Notes: Marks ▬ Denote Gages for Measuring Horizontal Strains
,, ❘ Denote Gages for Measuring Vertical Strains
,, ↘ Denote Rosettes for Measuring Strains in 3 Directions
,, ⊗ Denote Deflection Dial Indicator
,, ○ Denote Clinometers (Gunners Quadrants)

Fig. 10. Test Girder Details and Strain Gage Locations

intermediate and lower cables a stress loss of approximately 8000 psi occurs due to successive elastic shortening of the girder in the process of prestressing. The total final prestressing force was in the neighborhood of 960 tons.

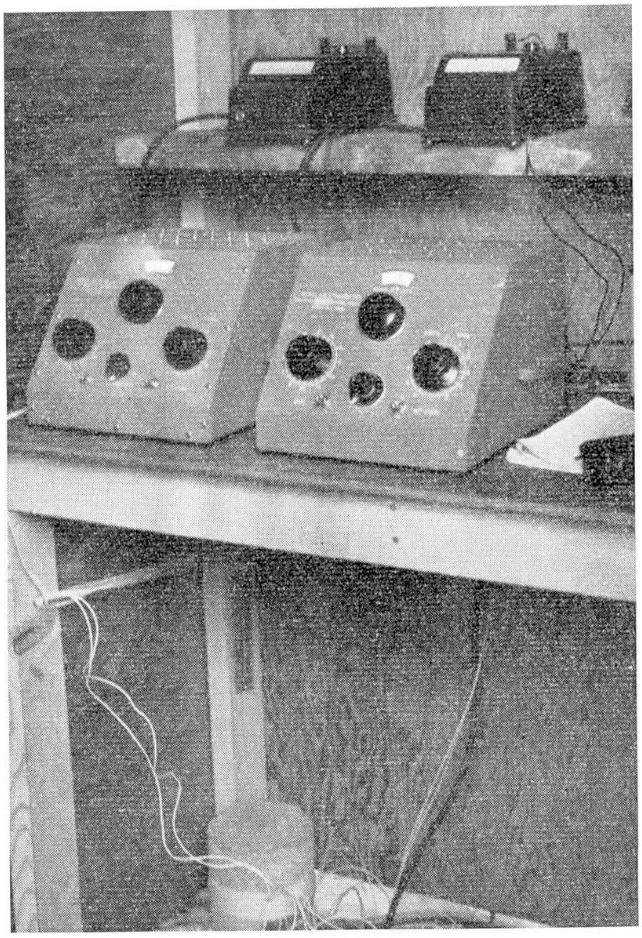


Fig. 11. Strainmeters and Compensator Gage
Two Model 301 Strainmeters used with the electric strain gages. Compensator gage is located on the test cylinder, bottom of picture

The test load was applied to the girder by means of 8 hydraulic jacks spaced as shown on Fig. 10 and 12 and operating against a steel frame and the weight of steel ingots suspended on hangers at each loading frame. The jacks were spaced in such a way as to produce the same bending moment as a uniformly distributed live load. A total of approximately 400 tons of steel ingots was available. The force exerted by each jack was determined by a load cell on top of each jack consisting of a short 2″ diameter steel bar with a pair of strain gages mounted on opposite sides of the bar. These load cells were calibrated to permit the jacks to be pumped up to definite and accurate testing loads producing known bending moments in the girder together with the dead load of the girder and the weight of the steel frames.

Unfortunately the use of the load cells above the jacks had to be abandoned at a load corresponding to 1400 lbs. per foot. At this stage the tilting of the load frames became excessive and for the remainder of the test jacking loads were determined from the hydraulic pressure gages in the jacks.

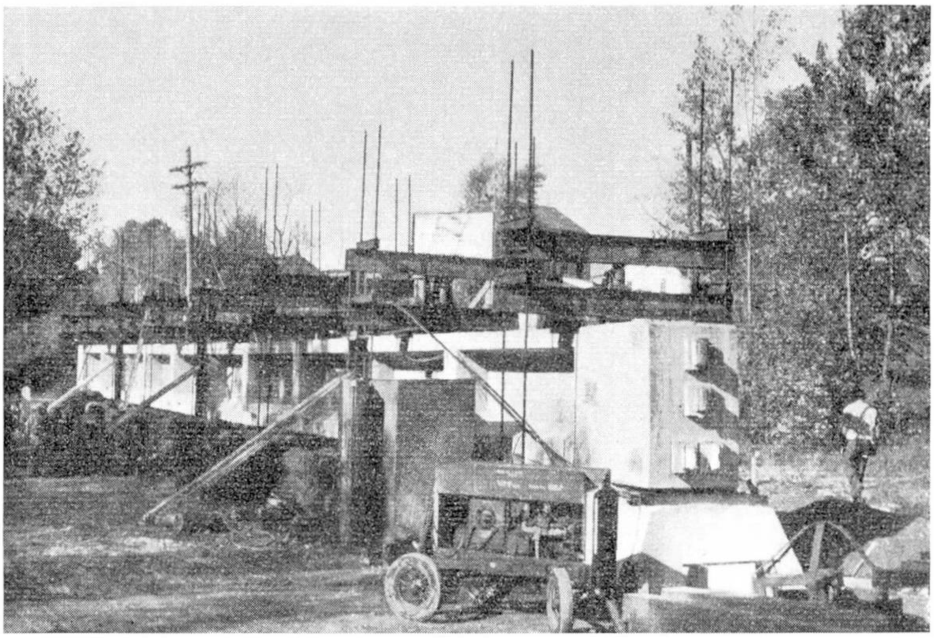


Fig. 12. View of Test Girder and Loading Arrangement
Looking northwest, after loading up to 3000 lbs. per foot. Instrument shelter in the foreground

c) *Calibration*

After the concrete strains due to prestressing were recorded the wiring was disconnected from the gages to permit removal of the cribbing. The gages on the concrete were then reconnected with one of the two strainmeters whereas the other strainmeter recorded the readings from the load cells over the jacks. The calibration of the load cells was made on the basis of 1 micro inch per inch strain being equal to 30 psi stress or a load of 94 lbs. on the load cell bar. The zero setting for the load cell gages was made when only the weight of the upper crossbeam of the loading frame rested on the load cell. Subsequently, by use of the units on the hanger bars, sufficient load was transferred into the hangers to obtain a reading of 32 micro inches per inch or a load of 3,016 lbs. on each load cell. According to calculations this corresponds to a uniform load of 250 lbs. per foot. At this load all strain gages on the concrete were set to zero and subsequent readings were for load increments over and above 250 lbs. per foot.

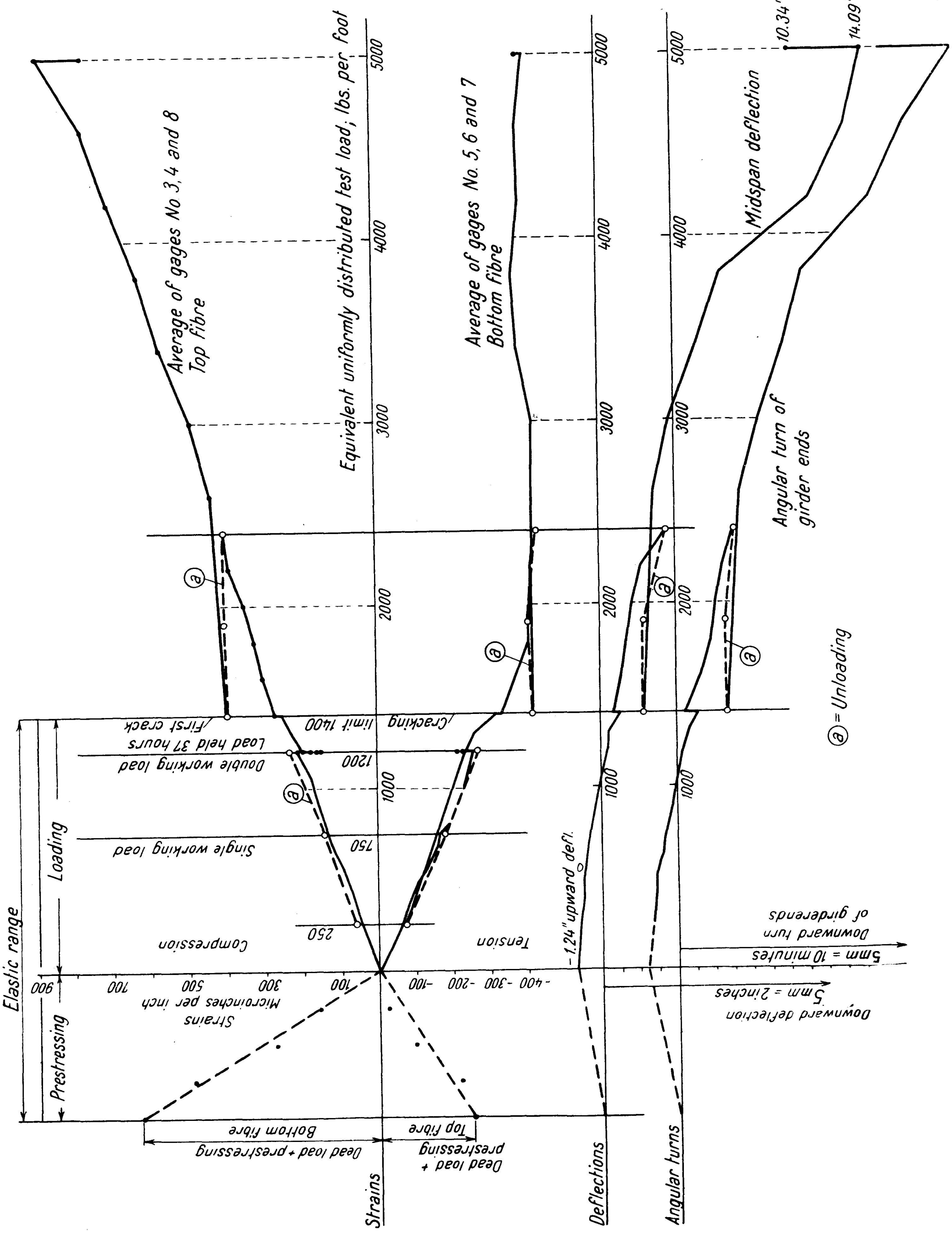


Fig. 13. Strain and Deflection Curves for Test Loads

d) *Testing Program*

In order to study the behavior of the girder under various loading conditions the following loading program was carried out and instrument readings were taken before and after all load increments:

Load girder with simulated uniform load in suitable increments up to equivalent of one half of working load (375 lbs. per ft.).

Increase load on one half of span by increments up to equivalent of full working load of 750 lbs. per ft.

Load entire girder to full working load.

After maintaining full working load for several hours unload to minimum calibration load.

Reload to full working load and subsequently in increments of 150 lbs. per ft. up to equivalent of double the working load (1200 lbs. per ft.).

After maintaining the double working load for 36 hours, unload to minimum calibration load of 250 lbs. per ft.

Reload to double working load and then in increments of 100 lbs. per ft. up to appearance of first crack (1400 lbs. per ft.).

After maintaining cracking load overnight the load was gradually increased during the "visitors' day" up to a load of 2400 lbs. per ft., then decreased to 1400 lbs. per ft. and from there increased until a deflection of approximately 12″ was reached[1]).

For the destruction test jacks No. 1 and No. 8 were removed and the corresponding steel ingots were used to build up a single load of 59 tons at the center of the girder span. This single load together with jacking loads of 42 tons each at No. 2, 3, 4, 5, 6 and 7 brought about failure.

e) *Test Results*

In Fig. 13 a series of graphs show the variation of strains for top and bottom at center of span and the variations of center deflection and of the angular turn of the girder ends. A tabulation of these values is given in Table I.

The strains measured with the Rosette type strain gages near the ends of the girder web (gages No. 11, 17, 13, 19 and 12, 18) were not very significant and up to a load of 2600 lbs. per ft. ($3^1/_2$ times working load) did not exceed 90 micro inches per inch or approximately 590 psi compression. Only gage No. 17 showed tensile strains after a loading exceeding 3000 lbs. per ft. However, in view of the corroborating results from the other 5 gages in the rosettes, gage No. 17 must be considered an unreliable record.

[1]) Originally it had been intended to load and unload the girder a number of times to demonstrate to the visiting Engineers the closing of cracks. However, due to circumstances beyond our control, this procedure had to be omitted.

Average Values of Strain Gage Readings and Deflections at Midspan and of Angular Turns at Girder Ends

Test Loading (Equivalent Uniformly Distributed Load; lbs. per ft.)		Average of Gages No. 3, 4 and 8 Top Fibre Micro Inches Per Inch	Average of Gages No. 5, 6 and 8 Bottom Fibre Micro Inches Per Inch	Midspan Deflection Inches − Upward + Downward	Average Angular Turn at Ends of Girder Minutes
Prestressing					
64 Wires		+ 32	+135	− 0,05	− 0,27
128 Wires		+158	+350	− 0,16	− 0,87
192 Wires		+232	+466	− 0,73	− 4,0
256 Wires		+248	+627	− 1,29	− 7,0
Test Loading	0	0	0	0	0
	250	48	60	− 1,24	− 6,75
	375	66	80	− 1,16	− 5,75
	375—500	73	98	− 1,12	− 5,75
	375—625	85	108	− 1,00	− 4,75
	375—750	93	128	− 0,92	− 4,50
	500—750	113	145	− 0,79	− 3,75
	625—750	125	162	− 0,69	− 3,25
Held 30 Min.	750	135	168	− 0,54	− 2,75
Held 30 Min.	750	143	172	− 0,48	− 2,50
Unload	250	63	68	—	− 6,50
	250	48	60	− 1,24	− 6,75
	750	135	170	− 0,50	− 2,75
	900	158	212	− 0,19	− 1,0
	1050	175	232	− 0,03	− 0
Held For 37 Hours	1200	213	252	+ 0,30	− 1,75
Held For 37 Hours	1200	195	227	+ 0,31	− 1,75
Held For 37 Hours	1200	161	228	+ 0,29	− 1,75
Held For 37 Hours	1200	183	233	+ 0,31	− 1,75
Held For 37 Hours	1200	161	213	+ 0,31	− 1,25
Held For 37 Hours	1200	235	262	—	− 2,50
	250	55	82	—	− 5,75
	250	48	60	− 1,24	− 6,75
	1200	203	242	+ 0,31	+ 2,00
	1300	223	258	+ 0,42	+ 3,00 Cracking
	1400	253	313	+ 1,00	+ 6,00 Cracking
	1400	271	326	+ 0,65	+ 2,0
	1600	301	370	+ 1,09	+ 6,75
	1800	325	407	+ 1,40	+ 9,50
	2000	355	412	+ 1,65	+10,50
	2200	390	412	+ 2,09	+12,50
	2400	403	425	+ 3,52	+15,25
	1900	401	405	+ 2,28	+13,30
	1400	395	412	+ 2,21	+13,30
	2600	436	417	+ 2,84	+17,50
	3000	483	415	+ 3,76	+22,25
	3400	565	382	+ 5,21	+29,50
	3800	621	375	+ 6,59	+34,50
	4200	696	393	+11,34	+50,25
	4600	763	388	+13,21	+61,75
Held For 2 Days	5000	880	407	+14,09	+74,25
Held For 2 Days	5000	765	390	+10,34	+55,00
	± 5200	Failure			

Table I

f) *Modulus of Elasticity of Girder Concrete*

To determine the actual value of the modulus of elasticity of the concrete in the test girder, 4 separate methods were used and compared.

The first of these was the calculation of Ec from the deflections and end slopes as measured for various load increments. The average value resulting from this procedure was $Ec = 7\,570\,000$ psi for loads ranging from 250 to 750 lbs. per ft. and $Ec = 6\,530\,000$ psi for loads ranging from 250 to 1200 lbs. per ft.

The second method was the comparison of actual bending strains to calculated bending stresses at the quarterpoint and at the center of the span for similar load ranges. The average value for Ec according to these calculations is $Ec = 6\,100\,000$ psi.

A third method consisted of a series of vibration tests which yielded oscillograms with an average frequency of 2,16 cycles per second. This corresponds to an average Ec of 6 550 000 psi.

Finally a series of compression cylinders were tested under different loads at the age of 17, 21 and 38 days. The resulting stress strain diagrams reveal an average value of Ec of 3 600 000 psi. This is an unusually low value for this type of concrete. The results of the deflection and bending strain methods seem to better represent the actual behavior of the girder than the relatively small size standard test cylinders.

For the interpretation of the test results a value of $Ec = 6\,700\,000$ psi therefore seems to be reasonably well chosen.

g) *Comparison of Calculated and Measured Strains and Stresses*

For the comparison of calculated to measured stresses and strains the properties of the girder cross section at midspan must be calculated both for the condition before grouting in the cables and after grouting when the grout and the transformed cable area become part of the effective area.

For these two conditions the section properties at midspan of the test girder are as follows:

	Area	Center of Gravity from Bottom	Moment of Inertia	Section Modulus Bottom	Section Modulus Top
	in.^{2}	in.	in.^{4}	in.^{3}	in.^{3}
Concrete Only	1347	47,1	1 085 000	23 050	34 000
Concrete + Wire + Grout (For $n = 6$)	1495	43,3	1 273 000	29 400	35 700

The midspan moment produced by prestressing is calculated as follows:

Force: 256 Wires at 0,0593 in.^{2} at 126 000 psi $=$ 1 915 000 lbs.
Lever Arm: $47{,}10 - 8{,}78 = 38{,}32$ in.
Moment: $1\,915\,000 \times 38{,}32'' = 73\,400\,000$ in. lbs.

The corresponding fibre stresses immediately after prestressing are:

At Top $$fc = \frac{1{,}915{,}000}{1347} - \frac{73{,}400{,}000}{34{,}000} = -730 \text{ psi}$$

At Bottom $$fc = 1420 + \frac{73{,}400{,}000}{23{,}050} = +4600 \text{ psi}$$

It is assumed that at the time of the test the total wire stress losses due to shrinkage and plastic flow amount to approximately 10% of the initial wire stress. The resulting fibre stresses due to prestressing and dead load at the time of the test are then as follows:

At Top $$fc = -655 + \frac{12 \times 5377}{34000} = +1240 \text{ psi}$$

At Bottom $$fc = +4150 - \frac{12 \times 5377}{23050} = +1350 \text{ psi}$$

For subsequent loadings the section modulus of the grouted girder is used in the calculation of stresses.

The moment curve produced by the testing loads up to cracking load (1400 lbs. superimposed testing load) is very nearly a parabola simulating the bending action of uniformly distributed load.

The bending moment in foot pounds at midspan is approximately equal to 3000 times the total equivalent uniform load per foot of girder.

The measured strain differences between the 746 lbs. and the 375 lbs. test load were extrapolated towards the "0" load in order to find the total strain and stress increments at significant loading stages. Unfortunately, the strain gage measurements during the prestressing operation cannot be interpreted and accepted without serious reservations. This is due to the fact that the supporting plates under the ends of the girder were placed after the prestressing was completed. The girder therefore did not lift itself off the cribbing as is normally the case and its dead load did not become effective simultaneously with the prestressing force. This is the reason why, for purposes of interpretation and comparison, the calculated stresses after prestressing are used in the stress tabulation (Fig. 14). Differences between calculated and measured strains seem to indicate that a greater portion of the shrinkage and plastic flow has actually taken place than the 10% that was assumed for purposes of comparison at the age of the girder when the tests were performed.

h) *Observations and Comments on Significant Loading Stages*

Before prestressing of the test girder it was found upon inspection that a vertical crack had occurred approximately 17 feet east of midspan, probably due to local settlement of the timber cribbing. The crack closed up after completion of the prestressing. At this time a strain gage was placed across the

Loading condition	Load increment	Equivalent uniform superimposed testing load lbs./ft.	Total Equivalent uniform load lbs./ft.	Total moment foot kips	Concrete stresses at midspan: Calculated (with 10% loss at age of test) Top	Concrete stresses at midspan: Calculated (with 10% loss at age of test) Bott.	Concrete stresses at midspan: From strain measurements eav. = 6 700 000 psi Top	Concrete stresses at midspan: From strain measurements eav. = 6 700 000 psi Bott.	Measured deflection inches midspan – upward + downward
Due to prestressing (at age of 30 days)					– 655	+ 4150	—	—	·
Dead load Girder (incl. grout)	1800	0	1800	5,377	+ 1240	+ 1350	+ 867[1]) + 1240	+ 2200[1]) + 1248	– 1,29″
D.L. Girder + superimposed D.L. of road-deck and misc. D.L.	250 50	250 300	2050 2100	6,125 6,275	+ 1492 + 1542	+ 1034 + 973	+ 1562[2]) —	+ 852[2]) —	– 1,24 —
Above D.L. + live load + impact	445	745	2545	7,605	+ 1990	+ 430	+ 2182	+ 110	– 0,50″
Above D.L. + double live load + double impact	445	1190	2990	8,935	+ 2435	– 112	+ 2562	– 358	+ 0,30
Load at which first crack opened	210	1400	3200	9,555	+ 2642	– 362	+ 2994	– 888	+ 1,00
Shortly before failure	3600	5000	6800	—	—	—	—	—	+ 14,1″
At failure	200	± 5,200	± 7000	20,800 ± [3])	—	—	—	—	± 25″

Remarks. [1]) For $E_c = 3{,}500\,000$ psi measurements unreliable (see text).
[2]) Extrapolated value for 250 lb. load plus calculated value for dead load.
[3]) Produced by approx. $38^1/_2$ tons on jacks 2, 3, 4, 5, 6, 7 and 60 tons of ingots at midspan.

Fig. 14. Stress Tabulation

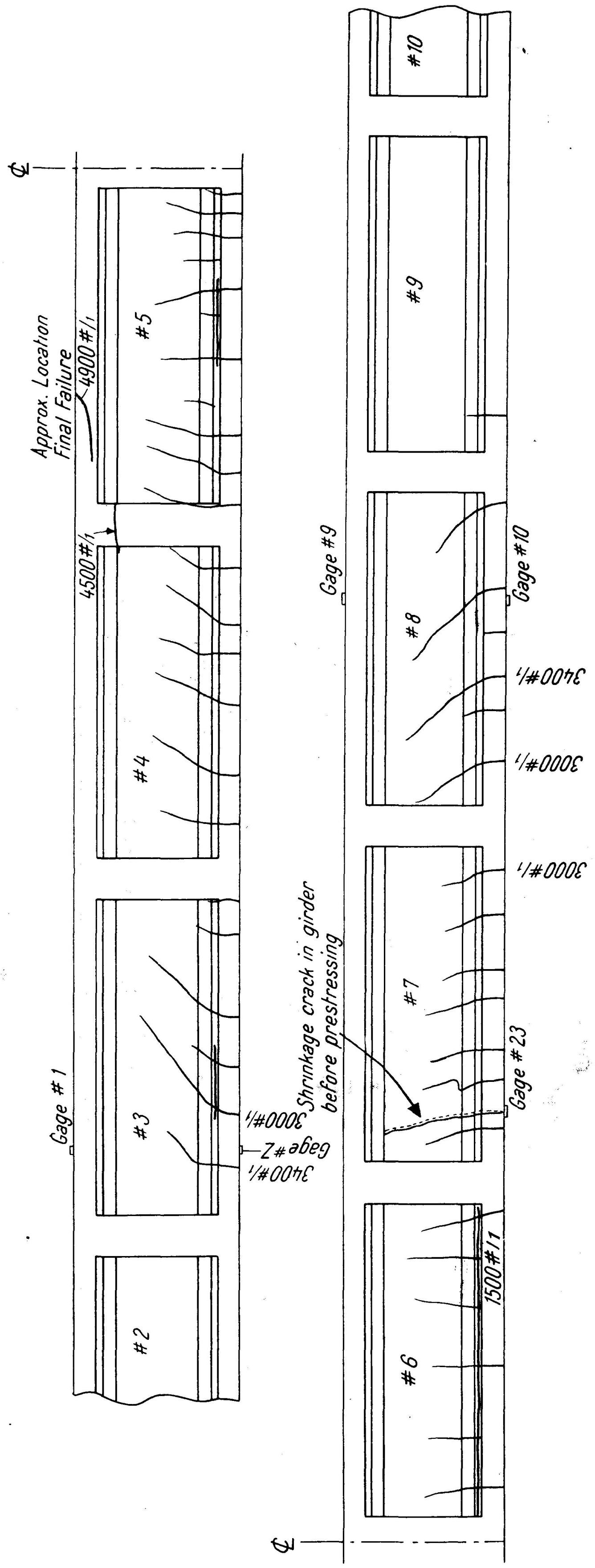


Fig. 15. Crack Patterns

Part elevation of girder, looking North, cracks on opp. side approx. similar.

crack in order to be able to qualitatively observe its behavior. The bending moment at the location of the crack equals 0,955 of the midspan moment.

A marked sudden increase in the readings of gage 23 at a test load of 1400 lbs./ft. indicated that the crack had opened again. The transfer of longitudinal stresses in the tension zone was obviously influenced by the existence of this crack.

Further local cracks occurred at a load of 1500 lbs./ft. near midspan (see Fig. 15). Two of them were longitudinal. One was located at the junction of the bottom flange with the southern face of the web and another one in the underside of the bottom flange. These unusual cracks were probably caused by a horizontal deformation of the test girder before prestressing due to its exceptionally prolonged exposure. The southern bottom cable must have exerted a sufficiently large outward force to cause these cracks.

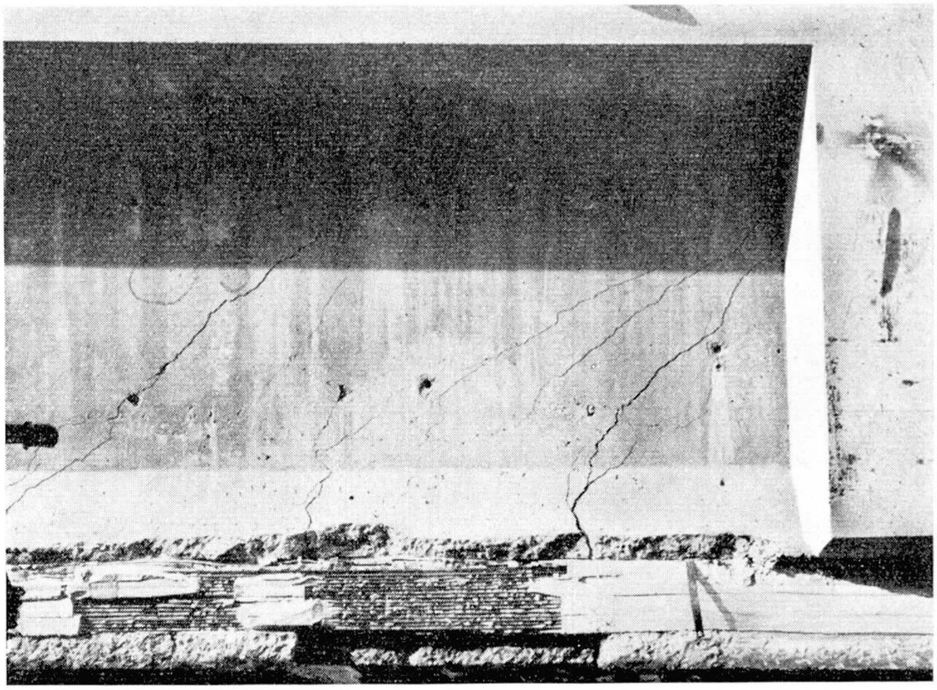


Fig. 16a

Subsequent cracking patterns are shown on Fig. 14. It is significant that up to a test load (1200 lbs./ft.) which corresponds to twice the working load in the bridge, no cracks due to loading could be observed. The midspan deflection increment due to this test load was 1,7″ or a little less than $^1/_{1000}$ of the span length. For a single working load (750 lbs./ft.) this deflection ratio is only $^1/_{2000}$ of the span length. For a depth to span ratio of 1 to 24, as in this instance, the total deflection at working load is therefore $2^1/_2$ times less than what is commonly allowed by the A.A.S.H.O. Code for steel plate girders ($^1/_{800}$ of span length).

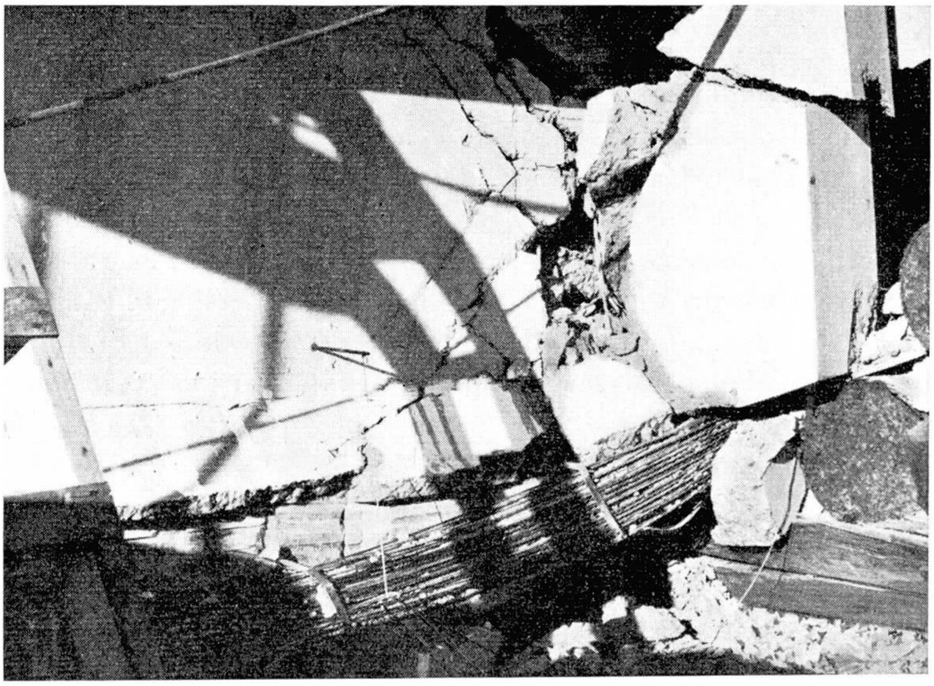


Fig. 16b

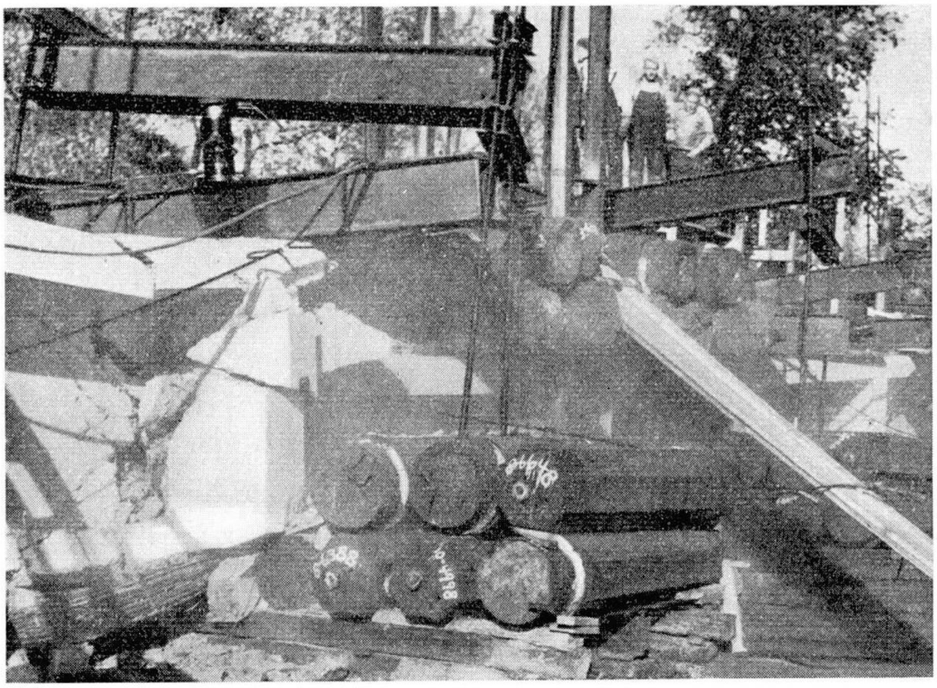


Fig. 16c

Fig. 16. Failure Cracks and View of Girder After Failure

Since a successful failure test was a prerequisite for the start of construction operations it was unfortunately not possible to make deflection and strain readings on an extended time schedule. However, it was equally valuable to have the unusual opportunity to make a large scale failure test.

Failure took place at a total midspan moment of 20 800 000 ft. lbs. which is 2,75 times the total working load moment. Since the dead load portion of the working load is permanent and not subject to variation, it is the excess of the failure load over and above the dead load which gives a measure of safety against occasional overload. The failure test demonstrated that the bridge could be loaded with eleven times the standard live load before failure occurs.

6. Conclusion

The construction of the first prestressed concrete bridge in the U.S.A. afforded a welcome opportunity to introduce in this country tested methods of prestressing for bridge girders. With minor field modifications the Blaton-Magnel method of cable stressing was found to be consistently effective.

The designers were necessarily somewhat restricted due to certain requirements made by the owner with reference to cross section and construction sequence. In spite of these limitations the prestressed concrete design proved to be very economical and competitive.

The load and failure test of the 160-ft. girder at Philadelphia demonstrated that present methods of design are adequate and on the safe side. The results of the test also emphasized again the importance to differentiate clearly between safety against cracking and safety against failure. While the first can be calculated fairly closely on the basis of the stresses in a homogeneous material the second is influenced by a radical redistribution of stresses in a semi-elastic state of the material. Due to the contributory influence of plastic deformation near the failure load, the shape of the cross section of the girder (shape factor) is of importance over and above the normal stiffness which it provides. Numerous previous tests have shown that the failure load is practically independent of the cracking load.

It is logical to separately prescribe safety requirements for each loading stage depending on whether it is of a permanent and well-defined or of a temporary nature. The probable frequency of overloads, the degree of uniformity of materials used, tolerances in dimensions and initial prestress, etc., all should be evaluated on a rational basis before design and safety requirements are specified.

The valuable experience gained from the full size girder test and from the construction of the bridge at Philadelphia will help to make possible further improvements both in the methods of construction and in the determination of design criteria for prestressed concrete structures.

Summary

The paper outlines first the structural and economic reasons which induced The City of Philadelphia to decide on a prestressed concrete bridge design as compared to other alternate designs.

A general description of the prestressed structure and the materials used for its construction follows.

In a general way all the important operations of the construction procedure are then explained. This includes pouring and prestressing of the 160′ main span girders, installing of girders and pouring of road deck and sidewalks.

The main portion of the paper is devoted to a discussion of the design considerations for the prestressed bridge girders and to a report and interpretation of a loading and destruction test of a full size 160′ span girder.

The various loading stages which the design has to satisfy are described and the corresponding stress limitations and calculated stress diagrams are given. Provisions made in the design for anticipated stress losses due to shrinkage, plastic flow of concrete and creep of steel are discussed.

The actual loading and destruction test of a full size 160′ span girder on the job site, made in compliance with the contract specifications, demonstrated the very ample factor of safety and made possible a study of the behavior of the girder at various loading stages including failure load.

The results of the strain gage and deflection measurements are tabulated and compared with calculated values. The interpretation of the test results is discussed with reference to the special characteristics of the concrete and the prestressing steel used for construction of the bridge.

In conclusion the significance of the test demonstration and the importance of crystallising proper design criteria for prestressed concrete design is pointed out. It is felt that only after considerable further work with semi-elastic materials can the field of prestressed concrete design and construction be properly covered by a special code.

Zusammenfassung

Es werden zuerst die konstruktiven und wirtschaftlichen Gründe behandelt, die die Stadt Philadelphia veranlaßten, sich für den Bau einer vorgespannten Brücke gegenüber andern Bauweisen zu entscheiden.

Hierauf folgt die Beschreibung der vorgespannten Konstruktion und der verwendeten Baustoffe.

Ganz allgemein werden dann die wichtigsten Bauvorgänge erklärt, so das Betonieren und Vorspannen der 160′ langen Hauptträger, das Verlegen der Träger und das Betonieren der Fahrbahndecke und der Gehwege.

Der Hauptteil der Arbeit befaßt sich mit der Konzeption der vorgespannten Brückenträger sowie dem Bericht und der Auswertung einer bis zum Bruch durchgeführten Belastungsprobe eines Trägers von 160′ Länge (natürlicher Größe).

Die verschiedenen Lastfälle, die im Projekt berücksichtigt werden mußten, werden beschrieben und die entsprechenden zulässigen Spannungen und errechneten Spannungsdiagramme angegeben. Ferner werden die beim Entwurf angenommenen Spannungsverluste infolge Schwinden und Kriechen des Betons und Kriechen des Stahls erläutert.

Der tatsächliche Belastungs- und Bruchversuch eines 160′ langen Trägers in natürlicher Größe auf dem Bauplatz, der in Übereinstimmung mit den Vorschriften des Bauvertrages durchgeführt wurde, zeigte einen hohen Sicherheitsgrad und ermöglichte die Beobachtung des Verhaltens des Trägers bei verschiedenen Belastungsstufen bis zur Bruchlast.

Die Ergebnisse der Spannungs- und Durchbiegungsmessungen wurden zusammengestellt und mit den berechneten Werten verglichen. Die Auswertung der Versuchsergebnisse ist wiedergegeben unter Berücksichtigung der besonderen Eigenschaften des Betons und der Armierungseisen, die für den Bau der Brücke verwendet wurden.

Als Schlußfolgerung wird auf die Bedeutung von Versuchen und der Ausarbeitung von Entwurfsgrundlagen, die der Bauweise des vorgespannten Betons angepaßt wurden, hingewiesen. Es sind wohl noch bedeutende Forschungen und Versuche mit halbelastischen Baustoffen notwendig, bevor endgültig für Projektierung und Ausführung auf dem Gebiete des vorgespannten Betons besondere Vorschriften aufgestellt werden können.

Résumé

L'auteur expose, tout d'abord, les raisons technique et économiques qui ont conduit la Ville de Philadelphia à préférer la construction en béton précontraint à d'autres modes de construction. Il décrit ensuite la disposition employée et les matériaux utilisés.

Il indique, dans leurs grandes lignes, les principales opérations de construction, en particulier, coulée et précontrainte de la poutre principale de 49 m de longueur, mise en place des poutres, bétonnage du tablier et des passerelles.

La partie principale de ce mémoire porte sur la conception des poutres en béton précontraint destinées à la construction des ponts, ainsi que sur la description et la discussion d'un essai de mise en charge et de rupture sur une poutre de 49 m de longueur (grandeur naturelle).

L'auteur expose les différents cas de charge prévus; il indique les charges admissibles correspondantes et les diagrammes des contraintes calculées. Il

discute les dispositions prises dans le projet pour tenir compte des pertes de charge à prévoir du fait du retrait, de l'écoulement plastique du béton et de celui de l'acier.

L'essai de mise en charge et de rupture d'une poutre de 49 m de longueur, en grandeur naturelle, sur place, effectué conformément aux prévisions du contrat, a mis en évidence la valeur très élevée du coefficient de sécurité et a permis d'observer le comportement de la poutre sous différentes charges, jusqu'à la charge provoquant la rupture.

L'auteur rassemble les résultats des mesures de contraintes et de déformations et les compare aux valeurs calculées. Il discute l'interprétation des résultats des essais, en tenant compte des caractéristiques particulières du béton et des armatures qui ont été employés pour la construction de cette poutre.

Il conclut en attirant l'attention sur l'importance des essais, ainsi que de l'étude minutieuse des conditions qui doivent servir de base à l'emploi du béton précontraint. Il émet enfin l'opinion que des recherches considérables devront encore être effectuées sur les matériaux semi-élastiques avant qu'il ne soit possible d'établir un code spécial concernant l'étude et la construction des ouvrages en béton précontraint.

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