Permanent File

PORTLAND CEMENT ASSOCIATION
RESEARCH AND DEVELOPMENT LABORATORIES
Development Department • Bulletin D34

PRECAST-PRESTRESSED CONCRETE BRIDGES
I. PILOT TESTS OF CONTINUOUS GIRDERS

By Paul H. Kaar, Ladislav B. Kriz
and Eivind Hognestad

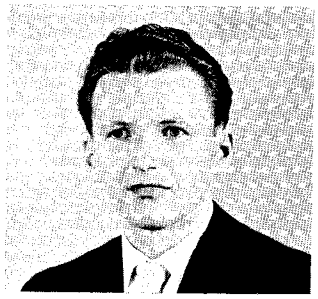
Reprinted from the copyrighted
Journal of the PCA Research
and Development Laboratories
Vol. 2, No. 2, 21-37, (May, 1960)

PRECAST-PRESTRESSED CONCRETE BRIDGES
I. PILOT TESTS OF CONTINUOUS GIRDERS

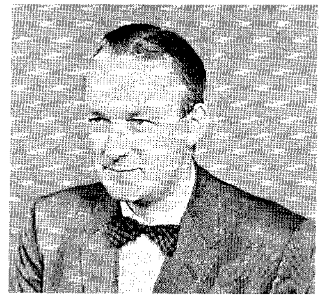
By Paul H. Kaar, Ladislav B. Kriz
and Eivind Hognestad

PORTLAND CEMENT ASSOCIATION
RESEARCH AND DEVELOPMENT LABORATORIES
5420 Old Orchard Road
Skokie, Illinois

Paul H. Kaar



Ladislav B. Kriz



Eivind Hognestad

Precast-Prestressed Concrete Bridges

1. Pilot Tests of Continuous Girders

By
Paul H. Kaar,* Ladislav B. Kriz
and Eivind Hognestad
Research and Development Laboratories
Portland Cement Association

SYNOPSIS

An extensive laboratory investigation on precast-prestressed concrete bridges is being reported in a series of Development Department Bulletins. The type of bridge which is under study involves I-shaped girders with a continuous situ-cast deck slab. In current practice, each bridge span is usually constructed of separate precast girders without utilizing the benefits which may result from continuity in the longitudinal direction. This bulletin presents the results of an experimental investigation regarding the feasibility of establishing continuity between precast girders from span to span. Adequate continuity and negative support moment resistance were obtained by placing deformed bar reinforcement in the deck slab across the piers supporting the girders.

BRIDGE TEST PROGRAM

During the past five years, precast-prestressed concrete girders have become extensively used in construction of highway bridges in the United States and Canada. Prestressed girders have been used in bridge spans of over 100 ft, though more commonly for spans of 40 to 80 ft. Several large multi-span bridges have been built, notably the bridges across the Tampa Bay. A vast construction activity is represented by the numerous smaller bridges being built or projected as grade separations for the National System of Interstate and Defense Highways.

*Development Engineer, Associate Development Engineer, and Manager, respectively, Structural Development Section.

An extensive experimental and analytical investigation directed toward improvement in prestressed bridge construction is in progress at the Research and Development Laboratories of the Portland Cement Association. The bridge type selected for study involves the composite construction of I-shaped longitudinal girders and a continuous deck slab cast after erection of the girders. Although each bridge span consists of separate precast girders, continuity is established in the longitudinal direction. Deformed bar reinforcement is placed in the situ-cast deck slab across the girder supports to develop the live-load negative bending moments at the supports. The resulting combined application of precast, situ-cast, prestressed and conventionally reinforced concrete gives rise to several questions regarding properties of structural concrete that are not thoroughly covered by tests. The investigation is, therefore, being carried out in several related stages:

1. *Girder Continuity.* The results of the first stage are reported in this bulletin and concern a study of the feasibility of establishing live-load continuity between precast girders from span to span by placing deformed bar reinforcement in the situ-cast deck slab. It was found that such a continuity connection is funda-

mentally sound, and suitable design criteria were explored.

2. *Horizontal Shear Connections.* A study is in progress to explore the usefulness of various means of horizontal shear transfer between precast girders and a situ-cast deck slab. Adhesive bond, roughness, keys, and stirrups are included as test variables.

3. *Bridge Design Studies.* Based on available information regarding structural concrete behavior and on the information obtained in the two first stages, a tentative typical bridge structure was designed for H20-S16 loading. These design studies brought out a need for additional test tata. Such data are being obtained in the stages 4 to 8 below, which involve tests of ½-scale models of various components of the selected bridge design.

4. *Shearing Strength.* Composite girder-slab members are being tested to develop design criteria for the stirrup reinforcement required to ensure adequate shearing strength for the continuous bridge girders. Each test member consists of one I-shaped precast girder and its portion of the situ-cast bridge deck.

5. *Flexural Strength.* This stage concerns tests of single-span and continuous girder-slab members as needed to extend the findings of Stage (1) to the girders of the selected bridge design.

6. *Creep and Shrinkage Studies.* Long-term tests are in progress on composite girder-slab members to study effects of creep and differential shrinkage on continuity behavior.

7. *Repeated Loading.* Repeated load tests are in progress to ensure the adequacy, under conditions of repeated loading, of the girder continuity connection.

8. *Reverse Bending.* In continuous bridges with more than two spans, small positive moments may develop under some loading conditions at interior girder supports. Tests will, therefore, be made to develop connection details to provide the required positive moment resistance in addition to a substantial negative moment resistance.

9. *Bridge Test.* Following completion of Stages 1 through 8, a complete two-lane, ½-scale, continuous bridge will be constructed and tested. Measurements made under a variety of service-load patterns will be followed by a test to destruction.

As the results of the various stages of this bridge investigation become available, they will be published in a series of PCA Development Department Bulletins. To a considerable extent, each stage will represent a completed project and will be published as such. Aside from their application to the specific bridge problems involved, the results of these individual stages contribute importantly toward improvements in precast-prestressed building construction.

CONTINUITY

Most multi-span precast-prestressed concrete bridges constructed in North America have consisted of a series of simple spans. Continuous construction has only been used in a few cases, even though continuity leads to several attractive advantages.

Advantages of Continuity

A continuous multi-span bridge structure may be superior to a series of simple spans in several respects:

1. Mid-span bending moments and deflections are reduced when continuity between adjacent spans is established. Hence, continuous construction will permit the use of more slender girder sections. When a standard section is used, continuity will permit a reduction in positive moment reinforcement and prestressing force. Either alternative can lead to reduced costs, provided that the means used to establish continuity are not too costly.

2. Continuous construction eliminates joints between adjacent spans at intermediate supports, which is desirable: (a) to provide an improved riding surface for vehicles; (b) to prevent drainage of water and ice-removal salts through deck joints to piers and the girder ends, thereby improving durability; and (c) to eliminate the initial cost of joints and their subsequent maintenance.

3. The reserve load capacity of a simply supported girder can be governed by a single cross-sectional region. In a continuous structure, however, overload will lead to moment redistribution so that several sections govern ultimate strength. Accordingly, continuity tends to minimize reductions in overload capacity caused by variability in properties of concrete and reinforcement.

Continuity Connections

Various methods have been developed abroad for continuous construction by post-tensioning of situ-cast concrete girders. In this country, economic conditions have called for methods of prestressing permitting a high degree of mechanization. This has led to extensive use of pre-tensioning combined with precasting, since this procedure can most readily be adapted to factory production techniques. The present investigation deals only with methods of design and construction by which precast pre-tensioned girders can be connected to form continuous bridge structures. Advantages of continuity may then be realized without sacrificing the economies of factory production. Several continuity connections between precast girders have been used in bridge construction. A few typical examples are shown in Fig. 1.

Cap Cables. Particularly in France, precast girders have frequently been connected by "cap cables" as shown in Fig. 1a. Post-tensioned after erection of the precast girders, the cap cables establish bending moments across the girder supports. The cost of anchorages and stressing is relatively high for the short cables used, and considerable friction loss is associated with their sharp curvature. Thus, the degree of continuity actually realized, and the economy of the cap cable solution, are matters of technical controversy in Europe.

Post-tensioned Cables in Deck Slab[1]. Dr. Fritz Leonhardt has suggested the modification of the cap cable solution shown in Fig. 1b. Only slightly curved short cables embedded in the deck slab between the girders are post-tensioned after erection of the precast girders and construction of the situ-cast deck.

Post-tensioned Bolts [2,3]. Various continuity connections involving post-tensioned short bolts have been used in bridges, gantries, and harbor structures of the Netherlands. Two types of such bolted connections are shown in Fig. 1c.

Transverse Prestressing[4,5]. Transverse prestressing has been used in various ways to establish continuity. In the example from England, shown in Fig. 1d, tensile stress is transferred across a pier by pretentioned longitudinal "junction slabs". The method involving overlapping girders, shown in Fig. 1e, has been used in several bridges in the Netherlands.

Conventional Bar Reinforcement. The post-tensioned continuity connections shown in Figs. 1b to 1e involve considerable field labor. This is not objectionable for relatively large structures. For short bridges, however, simple field operations are desirable to permit rapid construction. To this end, reinforcement may be embedded across supporting piers in a situ-cast deck slab and used to resist negative bending moments due to live load, as shown in Fig. 1f. This solution is characterized by simplicity of construction operations in the field, and it has been widely used in European and American precast building construction. It has also been used in American prestressed bridge construction to eliminate joints between adjacent spans, notably in the Illinois Toll Highway System[6], although the full structural design advantages of continuity have often not been utilized. This method has shown promise as an effective and economical method of establishing continuity in grade-separation bridges constructed under conditions existing in the United States and Canada. This solution was therefore selected for further study.

Deck Reinforcement Connection

As described briefly above and shown in Fig. 1f, a bridge made continuous for live loads by bar reinforcement placed in the deck slab is constructed as follows:

1. Precast-prestressed concrete girders are first placed on pile bents or piers with girder ends from adjacent spans spaced about four inches apart. To simplify precasting operations, it is desirable to develop a design in which the use of end blocks and draped prestressing reinforcement is avoided.

2. Diaphragms extending laterally between girders immediately above supporting piers are situ-cast in the field. In this manner, the girder ends are encased, and the space between adjacent girders is concreted. Since the diaphragm is wider than the space between girders, the concrete transferring compression between two girders over a support is strengthened considerably by lateral restraint.

3. The deck slab is cast before the diaphragm concrete has hardened, using forms supported by the precast girders acting as simple beams. Deformed bar

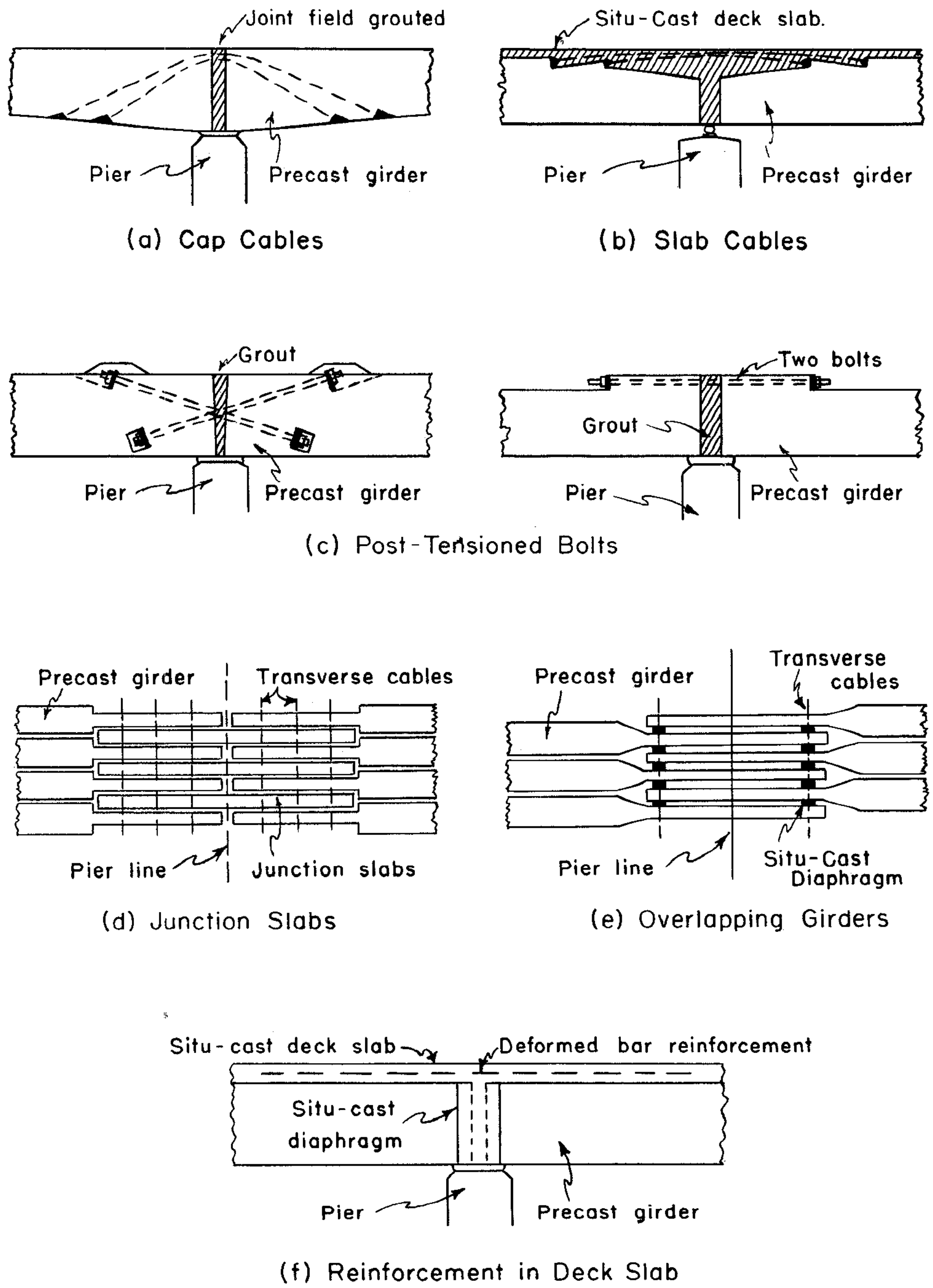


Fig. 1 — Examples of Continuity Connections.

reinforcement is embedded in the deck slab to resist negative girder moments due to live loads. In the resulting composite continuous structure, horizontal shearing strength between girder and slab is obtained by bond to a rough girder surface and by stirrups protruding from the girder. To simplify girder manufacture, it is desirable to develop methods which avoid the use of indented shear keys.

Working Stress Analysis

As a case of extreme severity, a girder without end blocks and without deflected strands was considered. This girder was analyzed for negative bending under the conditions shown in Fig. 1f. An area of prestressing steel equal to 0.6 per cent of the I-shaped girder cross section was assumed located in the lower flange and prestressed to 150,000 psi. Three different amounts of deck reinforcement were considered, and a concrete strength of 5,000 psi was assumed for the girder. Under these conditions, the bottom flange of the I-shaped girder is stressed to 0.40 f'_c by prestress alone, and analysis of such prestressing, plus application of a negative bending moment, gave the results shown in Table 1 when the straight-line theory was used.

It is seen that only very low deck reinforcement steel stresses can be developed if the stress in the bottom flange is limited to 0.45 f'_c, in which f'_c is the 5,000-psi. cylinder strength of the girder concrete. Similarly, if the composite section is considered loaded to a deck steel stress of 20,000 psi, high concrete stresses are calculated for the bottom flange. It is reasonable, therefore, that the deck reinforcement connection has so far been used in practice with caution and to a limited extent only.

An underestimate of the strength of the compression zone in bending is characteristic of the straight-line theory. Furthermore, if ultimate strength under negative bending moment is approached, high inelastic concrete strain in the bottom flange leads to a local reduction of the prestress force in regions of high moment near the girder ends. Also, the prestress is actually not fully effective at the girder ends, as assumed in the elastic analysis. The transfer of stress by bond from pre-tensioned steel to concrete takes place gradually over a transfer length. Finally, a full-scale bridge with a deck reinforcement continuity connection was tested prior to the Illinois Toll Highway construction[7]. Complete continuous behavior was observed and maximum girder moments nearly four times design values were sustained safely.

Objective of Pilot Tests

The laboratory investigation reported herein was carried out in the Portland Cement Association Research and Development Laboratories to explore: (1) the strength of the deck reinforcement continuity connection between precast girders subject to negative bending moment, and (2) the strength and moment redistribution of continuous girders with such a negative moment connection.

TESTS AND ANALYSIS

Scope of Tests

Fifteen T-shaped girders were tested in the three groups of tests outlined in Table 2 and Fig. 2.

Group I. Three 12-ft prestressed I-shaped girders were provided with deck slabs containing different amounts of bar reinforcement, and were tested to failure by negative moment.

Group II. Nine girders, each made up of two 6-ft I-shaped girders connected by a diaphragm and a deck slab, with different amounts of prestressed and deck slab reinforcement, were tested to failure by negative moment.

Group III. Three continuous girders 34 ft long were tested. Each girder was made up of one 22-ft I-shaped girder, to each end of which 6-ft girders were connected by diaphragms and the deck slab. Testing took place under combined positive and negative moments.

TABLE I — WORKING STRESS ANALYSIS

Deck reinforcement, per cent of bd in Fig. 2	Deck reinforcement steel stress, f_s, for $f_c = 0.45f'_c$	Bottom flange concrete stress, f_c, for deck steel $f_s = 20{,}000$ psi
0.83%	700 psi	2800 psi
1.66	660	3340
2.49	620	3820

Each test girder was identified by three numbers, as shown in Table 2. The first number refers to the group classification explained above, the second to the percentage of prestressed steel, and the third to the percentage of deck steel.

Test Girders

The specimens chosen for this laboratory investigation represent a ½-scale model of a 44-inch full-scale bridge girder. The model cross section is shown in Fig. 2. The prestressing steel was seven-wire, stress-relieved strand of ¼ in. diameter, and of 0.036 sq in. area. Yield stress at 1 per cent offset was 260,000 psi, while the ultimate strength was 270,000 psi and effective prestress was 150,000 psi. All strand was free of rust and cleaned of surface oil by washing in carbon tetrachloride. In the prestressed girders, 17 and 25 strands of prestress steel corresponded to 0.6 and 0.9 per cent of the girder cross section. In addition, three girders entirely without prestressed reinforcement were tested.

The deck slab was 3 in. thick and 24 in. wide. It contained either 0.83, 1.66 or 2.49-per cent steel based on bd in Fig. 2. All negative reinforcement and the stirrups were intermediate grade deformed bars (ASTM A305), having an average yield point of 48,000 psi.

The top surface of the girders was finished rough and without shear keys. Stirrups protruded from the girders into the deck slab to transfer horizontal shear. These stirrups also resisted diagonal tension in the girder webs. Group I and II girders, having 0.83, 1.66, and 2.49-per cent deck slab steel, contained U-shaped No. 3 stirrups spaced at 9, 6, and 5 in., respectively. Girders III-0.6-0.83a, III-0.6-0.83b, and III-0.6-1.66 had No. 3 stirrups spaced at 6, 3, and 3 in., respectively.

Concrete

The concrete girders and deck slabs were cast using 6.2 bags per cu yd of a blend of Type 1 portland cement and ¾-in. maximum size aggregate. Water-cement ratio was maintained between 0.50 and 0.60 by weight. The slump varied from 3 to 5 in., and an air entraining agent was added to provide from 4.5 to 5.5 per cent air in the concrete. Moist curing under burlap at about 70 F took place for the first three days after casting of both girders and slabs. Subsequently, the specimens were stored at approximately 70 F and 50 per cent relative humidity. Concrete cylinder strengths are given in Table 2, based on tests of three 6 by 12-in. cylinders per batch cast and cured with the individual girder and deck slab specimens. At the time of testing,

TABLE 2 — GIRDER TESTS

Girder No.	Per Cent Steel		Concrete Strength psi		Observed ult. moment M_{test} in. — kips	Neglecting prestress		Including prestress	
	Girder[1]	Deck Slab[2]	Girder	Deck Slab		M_{calc} in. — kips	$\frac{M_{test}}{M_{calc}}$	M_{calc} in. — kips	$\frac{M_{test}}{M_{calc}}$
I-0.6-0.83	0.6	0.83	5950	3980	−2480	−1900	1.30	−1960	1.26
I-0.6-1.66	0.6	1.66	5950	3980	−3640	−3640	1.00	−3460	1.05
I-0.6-2.49	0.6	2.49	5950	3980	−3520	−5040	0.70	−4220	0.83
II-0-0.83	0	0.83	4840	3630	−2270	−1890	1.20	No prestress	
II-0-1.66	0	1.66	4840	3630	−3280	−3530	0.93	No prestress	
II-0-2.49	0	2.49	4840	3630	−4250	−4530	0.94	No prestress	
II-0.6-0.83	0.6	0.83	5030	3210	−2140	−1890	1.13	−1900	1.13
II-0.6-1.66	0.6	1.66	5030	3210	−3020	−3560	0.85	−3260	0.93
II-0.6-2.49	0.6	2.49	5030	3210	−4040	−4670	0.86	−3575	1.13
II-0.9-0.83	0.9	0.83	5380	3670	−2440	−1900	1.28	−1860	1.31
II-0.9-1.66	0.9	1.66	5380	3670	−3490	−3600	0.97	−3070	1.14
II-0.9-2.49	0.9	2.49	5380	3670	−4000	−4880	0.82	−3350	1.19
III-0.6-0.83a	0.6	0.83	5850	4390	−2570 +2900	−1910	1.34	−1960 +3090	1.31 0.94
III-0.6-0.83b	0.6	0.83	5410	3830	−2740 +3020	−1900	1.44	−1930 +3060	1.42 0.99
III-0.6-1.66	0.6	1.66	5370	3430	−4160 +2100	−3600	1.16	−3340 +3020	1.24 0.70

[1] Per cent of cross section of precast girder; maximum concrete prestress 2100 and 3200 psi for 0.6 and 0.9 per cent prestress steel, respectively.
[2] Per cent of bd in Fig. 2, average yield point 48,000 psi.

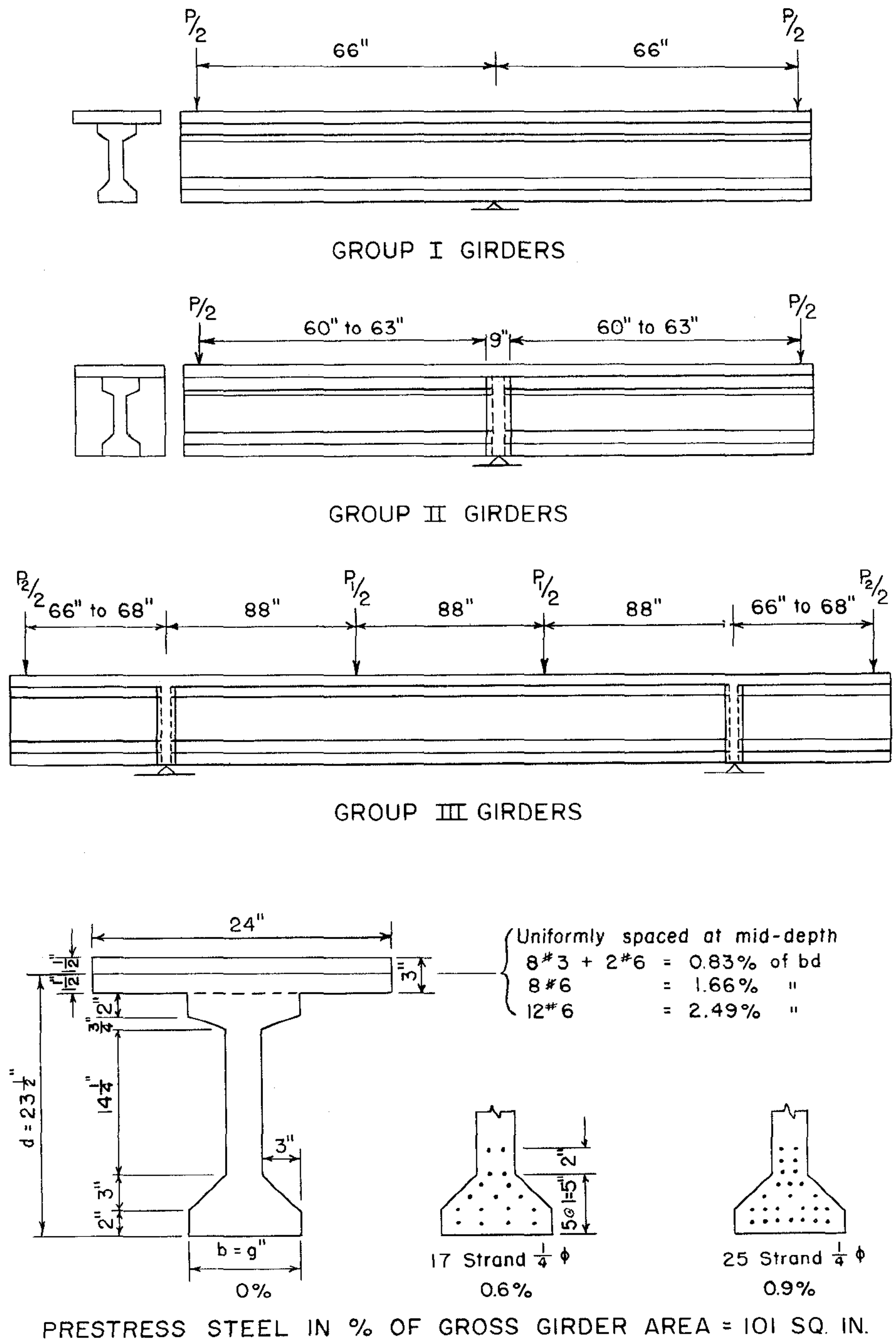


Fig. 2 — Cross Sections and Loading Arrangements of Girders.

the girder and slab concretes were about 15 and 7 days old respectively.

Fabrication

Before tensioning of the strand, cylindrical load cells were mounted directly under seven of the strand vises to measure the strand load. These cells were $1\frac{3}{16}$-in. diameter steel cylinders drilled for strand insertion and instrumented with SR-4 electric strain gages.

Within each group, girders having the same amount of prestressed reinforcement were cast simultaneously in a 50-ft prestressing bed, so that concrete strength and the prestress force were nearly identical. Pre-tension in the girders was released at about six days when concrete strength was approximately 4000 psi. At this time the girders were separated by torch-cutting the strand.

At a girder age of eight days, the separate girder components for Groups II and III were positioned with a 3-in. gap, and the deck slab was cast together with a diaphragm 9 in. thick, using the same concrete mix as for the girders. Testing took place at a girder age of about 15 days.

Test Procedure

Group I and Group II girders were tested in negative bending over a central support. A testing machine applied load to one end, while the other end was anchored to the laboratory floor, as shown in Fig. 3a. A load cell was incorporated into the anchoring device so that the anchoring force could be measured. For the girders in Group II, the support was directly under the diaphragm simulating the pier support of a multi-span highway bridge.

Group III girders were supported at points under the two diaphragms spaced 22 ft apart as shown in Fig. 3b. The girder was loaded by the testing machine at the third-points of the 22-ft span. The cantilevering 6-ft girder ends, attached to the 22-ft section by the diaphragms and the reinforced deck slab, were anchored to the laboratory floor. Load cells were included in the anchoring system so that anchoring forces could be measured. Hydraulic rams were incorporated into the anchoring systems of Girders III-0.6-0.83b and III-0.6-1.66, so that rotation at the supports could be prevented by controlled cantilever loading. A machinist's level was mounted on the girder deck over the support. As the test progressed, load was applied independently in the girder span and at the cantilever ends so that the level bubble remained balanced. Thus, Girder III-0.6-0.83a was tested with restraint at the supports; Girders III-0.6-0.83b and III-0.6-1.66 were tested with full fixity at the supports.

Strains in the negative reinforcing steel over the support were measured by at least five SR-4 gages for each girder. SR-4 gages were also attached to the girder web and bottom flange. Strains were recorded continuously on strip-chart recorders. Deflection was measured by dial gages.

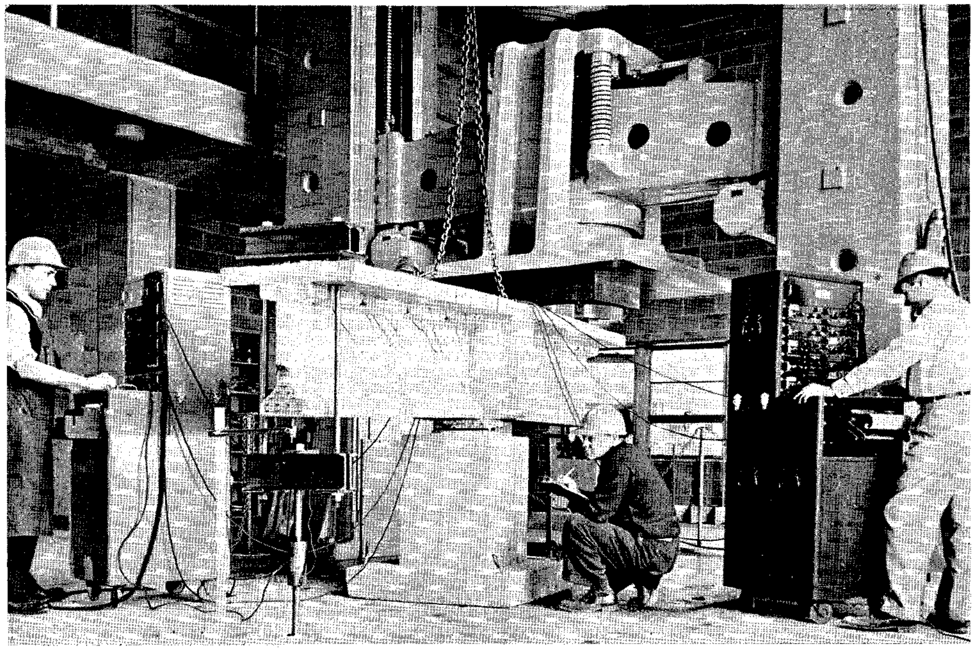
Ultimate Flexural Strength Analysis

Ultimate flexural strength for the composite section subjected to a negative bending moment was calculated using a rectangular stress block for concrete in compression as shown in Fig. 4. A trapezoidal stress-strain diagram was used for the deck slab deformed bar reinforcement. The calculations were made with and without consideration of the effects of the prestressing force.

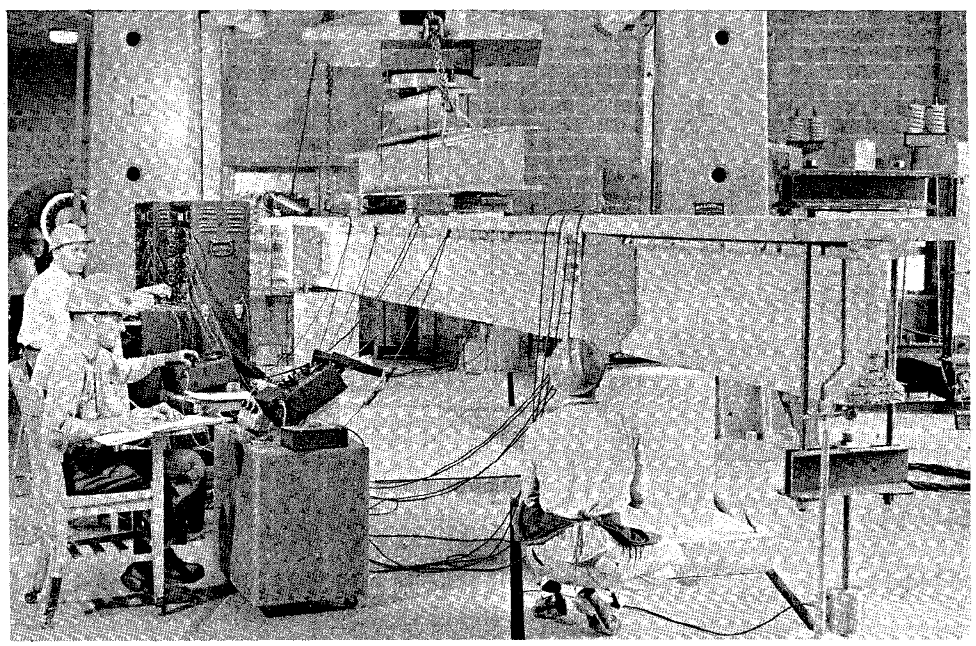
Similar calculations using curved stress-strain relationships for the concrete produced calculated ultimate moments slightly closer to the test results. Similarly, use of the actual stress-strain curve for the deck reinforcing steel increased the calculated moments somewhat for the girders with 0.83 per cent deck slab steel for which some strain hardening took place.

The improvement gained by the more complex calculation methods was small. The simplicity of the rectangular stress block analysis, therefore, warrants its use for practical design purposes, and all comparisons of observed and calculated moments presented herein are based on the rectangular distribution of concrete stress.

For the girders of Group II and III, the moment calculations presented are applicable to a section at the face of the support diaphragm and involve the concrete strength of the girder. Failure is, of course, also possible at a section through the diaphragm between the two connected girders. This is particularly so since the diaphragm concrete strength was about 2000 psi less than that of the girder concrete. It was found in all tests reported herein, however, that the diaphragm concrete transferring compression between two girder flanges was strengthened by lateral restraint, so that failure always took place outside the diaphragm.



(a) — Group I and Group II Girders.



(b) — Group III Girders.

Fig. 3 — Test Methods.

Notation. The letter symbols used in this bulletin are generally defined when they are first introduced. The common symbols are listed below for convenient reference:

A_s = area of deformed bar tension reinforcement

A_{sp} = area of prestressed reinforcement

b = width of section

C = internal compression force in concrete at ultimate moment

c = depth to neutral axis

d = effective depth of deformed bar reinforcement

E_s = modulus of elasticity of reinforcement

ϵ_{ce} = concrete strain due to effective prestress at extreme compression edge

ϵ_s = strain in deformed bar reinforcement

ϵ_u = ultimate compressive strain in concrete

f'_c = compressive strength of 6 by 12-in. cylinders

f_{se} = effective prestress in reinforcement

f_s = stress in deformed bar reinforcement

f_{sp} = stress in prestressed reinforcement at ultimate moment

f_y = yield point of deformed bar reinforcement

jd = internal moment arm of force C

k_1c = depth of rectangular stress block

M_{ult} = ultimate moment

M_{calc} = calculated ultimate moment

M_{test} = measured ultimate moment

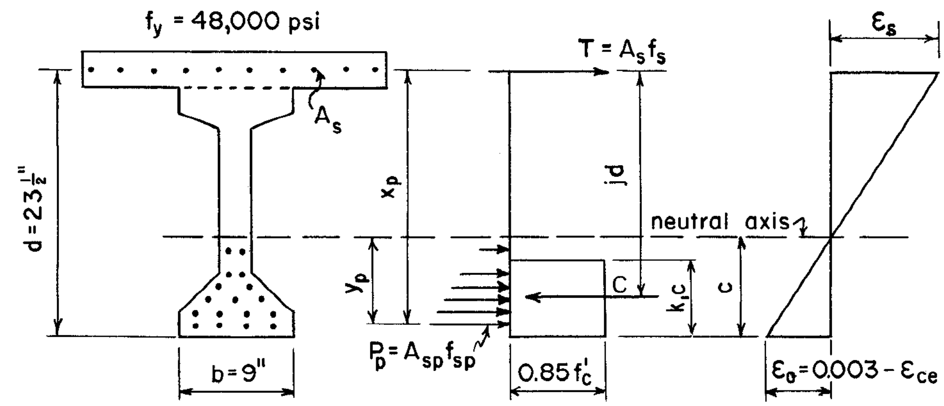
T = tensile force in deformed bar reinforcement at ultimate moment

x_p = internal moment arm of prestressed reinforcement

y_p = depth of neutral axis from prestressed reinforcement

Neglecting Prestress Effects. When tension controls, the deck reinforcement yields at the ultimate moment. The tension force, $T = A_sf_y$ in Fig. 4, then equals the compression force in the concrete, C, which acts at the centroid of the flange portion subjected to a uniform stress equal to $0.85f'_c$. The depth to the neutral axis, c, can thus be calculated, and the ultimate moment is obtained as A_sf_y times the moment arm jd. Then, assuming an ultimate concrete strain of 0.003, the strain in the tension reinforcement, ϵ_s, is calculated by linearity of strains. When this steel strain is greater than the yield strain of 0.00165, tension does indeed control.

Compression controls when the tension steel does not yield before the ultimate moment is reached. For this case, calculations may be based on an iteration process. An estimate is made of the neutral axis position, c in Fig. 4, and the forces



$k_1 = 0.85$ for $f'_c \leqq 4000$ psi, and the value of k_1 was reduced by 0.05 for each 1000 psi over 4000 psi

Fig. 4 — Rectangular Stress Block Used for Flexural Strength Calculations.

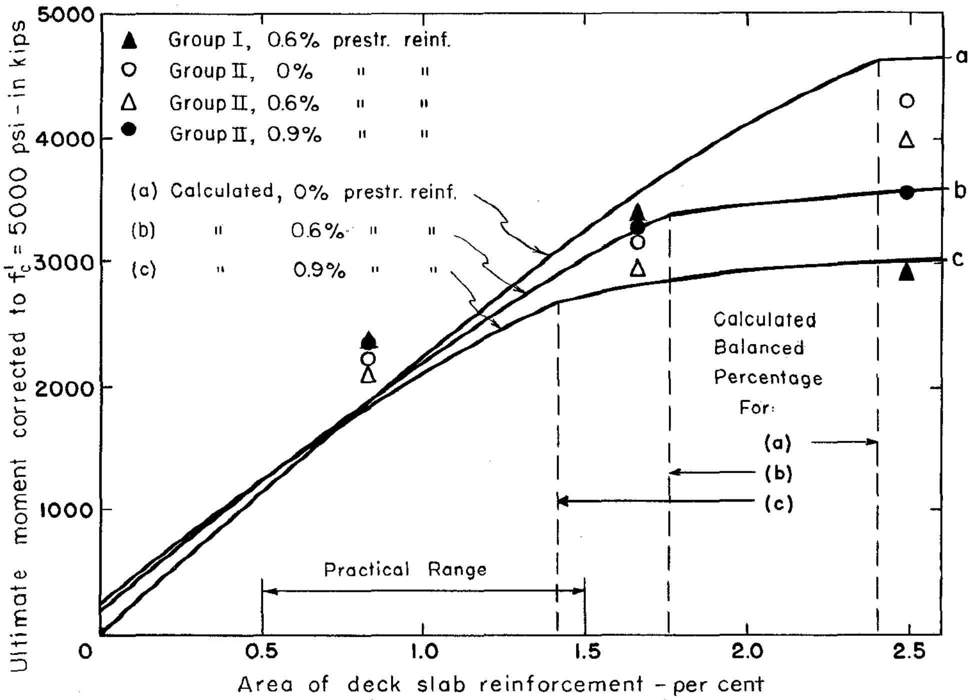


Fig. 5 — Ultimate Moments for Groups I and II.

C and T are calculated. Corrected estimates of c are then made until the two internal forces become equal, and the ultimate moment is calculated as A_sf_s times jd, in which steel stress, f_s, is less than the yield point.

Ultimate moments calculated in this manner for each girder are given in Table 2. Calculated moments for an assumed girder concrete strength of 5000 psi are given by curve (a) in Fig. 5.

Including Prestress Effects. To estimate effects of prestress on ultimate moments, further assumptions were made as outlined in Fig. 4. The prestress transfer length was conservatively assumed equal to zero, and calculations were made for an effective prestress, f_{se}, of 150,000 psi. Assuming a maximum concrete strain of 0.003, the stress in each layer of prestressed reinforcement at the ultimate moment, f_{sp}, can then be calculated by linearity of strains. When minor effects of prestress strains in the concrete are neglected:

$$f_{sp} = f_{se} - (0.003 - \epsilon_{ce}) E_s \frac{y_p}{c} \qquad (1)$$

in which y_p is positive below the neutral axis and negative above that axis, and ϵ_{ce} is the concrete strain due to effective prestress at the extreme compression edge.

Internal equilibrium then gives:

$$T = A_sf_s = C - \Sigma A_{sp} f_{sp} \qquad (2)$$

in which A_{sp} is the area of prestressed steel in each layer; and the ultimate moment is then given by:

$$M_{ult} = Cjd - \Sigma A_{sp}f_{sp} x_p \qquad (3)$$

Moments calculated in this manner for a girder concrete strength of 5000 psi and for 0.6 and 0.9 per cent prestressed reinforcement are given by curves (b) and (c) in Fig. 5. Various positions of the neutral axis were assumed, and corresponding values of M_{ult} and A_s were calculated by Eqs. (1) to (3).

Comparison. It is seen in Fig. 5 that, for 0.6 per cent prestressed reinforcement and a deck slab reinforcement less than 0.8 per cent consideration of the prestressed reinforcement increases the calculated ultimate moment. This results from the fact

TABLE 3 — CONDITIONS NEAR ULTIMATE STRENGTH

Girder No.	Concrete Strain*		Deck Steel Strain‡		Horizontal Shear at Ult. Load in psi
	Millionths	Per cent of Ultimate Load	Millionths	Per cent of Ultimate Load	
I-0.6-0.83	3,370+	100	15,900	95	200
I-0.6-1.66	3,380+	100	1,660	100	295
I-0.6-2.49	4,530	100	1,280	100	285
II-0-0.83	3,600	100	1,920	92	198
II-0-1.66	4,160+	100	1,410	100	280
II-0-2.49	2,280	100	1,510	100	363
II-0.6-0.83	2,010	100	15,470	96	189
II-0.6-1.66	1,420+	100	3,580	100	268
II-0.6-2.49	2,210+	100	1,460	98	361
II-0.9-0.83	2,740	100	11,880	91	202
II-0.9-1.66	3,520	100	1,640	100	294
II-0.9-2.49	3,040	100	1,200	95	337
III-0.6-0.83a	1,780	100	2,180	100	336
III-0.6-0.83b	1,830+	100	18,480	99	353
III-0.6-1.66	2,670	87	2,770	87	385†

* Measured by 6-in. SR-4 gages extending from 2 to 8 in. from the support diaphragm, including strain due to prestress.
‡ Yield strain of deck steel is 1650 millionths.
+ SR-4 gages were outside region of concrete crushing.
† Failure by horizontal shear.

that some layers of strand are located above the centroid of the compressive force in the concrete. When the deck reinforcement exceeds 0.8 per cent, however, consideration of prestress decreases the calculated moment because the prestress force reduces the available internal compression force, C. Thus, the balanced percentage of deck reinforcement separating the regions of tension and compression control is reduced from 2.4 to 1.8 per cent when 0.6 per cent prestress is considered.

In the practical range of deck slab reinforcement from 0.5 to 1.5 per cent, Fig. 5 indicates that the calculated effects of prestress are small. Furthermore, in the regions of high negative moment near the diaphragms of the connected girders of Groups II and III, the effective prestress is reduced. The transfer of prestress by bond from the pre-tensioned steel to the concrete takes place over a considerable transfer length. For practical design purposes, therefore, prestress effects should be checked by Eqs. (1) to (3); but in most cases it is probably adequate to neglect effects of prestress on ultimate negative moment. Since anchorage may be inadequate, the increase of ultimate moment, calculated for small amounts of deck reinforcement by including prestress effects, should not be utilized in practice.

DISCUSSION OF TEST RESULTS

Prestress Transfer Length

In considering the performance of the diaphragm-connected girders, prestress transfer mechanism is of importance. This mechanism has been discussed by Janney[8]. The distance over which the prestress transfer extends varies with intensity of prestress, surface condition of the strand, and concrete quality. The initial transfer length for clean, smooth 1/4-in. diameter strand prestressed to 150,000 psi, acting with 5500 psi concrete, was found to be about 20 in. by auxiliary tests of the investigation reported herein.

Groups I and II

Measured ultimate negative moments, M_{test}, are compared with the calculated moments, M_{calc}, in Table 2. Calculated moments are listed separately neglecting and including prestress effects.

The ultimate moments are also plotted in Fig. 5 as a function of deck slab reinforcement percentage. The theoretical curves in that figure involve a 5000-psi girder concrete strength. Therefore, the observed ultimate moment of each test girder was corrected to a 5000-psi concrete strength for plotting. This was done by dividing each test moment by the calculated moment based on the actual girder concrete strength, and then multiplying by the calculated moment based on a concrete strength of 5000 psi. Prestress effects were included for these calculated moments.

Measured strains at or near ultimate strength and horizontal shearing stress at ultimate strength are tabulated in Table 3. The concrete strain given was measured at the lower edge of the girder flange, mid-length of the strain gages being 8 in. from the girder ends. Since the gages were attached after prestress release, 850, 450, 680, and 450 millionths were added to the measured strains of Girders I-0.6, II-0.6, II-0.9, and III-0.6, respectively. This correction for strain at prestress release was based on auxiliary tests. Horizontal shearing stress at ultimate strength was calculated as shearing force at supports divided by $\frac{7}{8}$ bd.

Ultimate Negative Moment. Neglecting prestress effects, the M_{test}/M_{calc} ratios for the girders of Groups I and II in Table 2 and Fig. 5 show a considerable effect of the amount of deck slab reinforcement. The average M_{test}/M_{calc} ratios are 1.23, 0.94, and 0.83 for 0.83, 1.66, and 2.49-per cent deck slab steel, respectively. The high average ratio of 1.23 resulted from strain hardening of the deck slab steel. The low average ratios for the high percentages of deck steel stem from a reduction in available internal compressive force caused by prestress effects.

When prestress effects are included in the calculations of ultimate moment, the average M_{test}/M_{calc} ratios become 1.23, 1.01, and 1.02 for 0.83, 1.66 and 2.49-per cent deck steel, with a grand average value of 1.09. Thus, the analysis of prestress effects reflects the test findings realistically.

Failure Modes. The concrete and deck steel strains given in Table 3 are indicative of the failure modes involved. All girders with 0.83-per cent deck steel were clearly controlled by tension; extensive yielding of the deck steel developed before crushing of the concrete took place regardless of the various types of prestress effects. For Girders II-0 and II-0.6 with 1.66-per cent deck steel, tension was also controlling, while for these girders with 2.49-per cent deck steel the mode of failure was nearly balanced. The heavily prestressed Girders I-0.6 and II-0.9 with 1.66-per cent deck steel were nearly balanced, and those

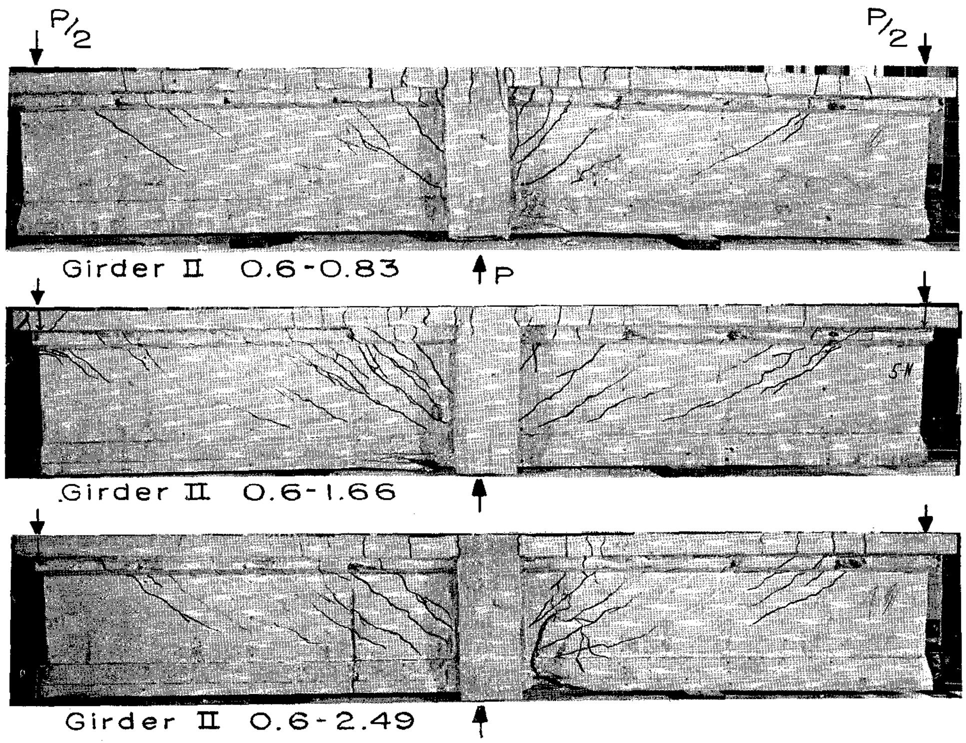


Fig. 6—Typical Crack Patterns and Failures Group II Girders.

with 2.49-per cent deck steel were controlled by compression. Thus, the calculated shift of the balanced percentage due to prestress effects shown in Fig. 5 was experimentally verified. A typical set of girders is shown in Fig. 6 after test to failure. In no case did the girders pull out of the diaphragm. Ultimate strength was always controlled by sections outside the diaphragm.

Horizontal Shear. The horizontal shearing stresses given in Table 3 for Groups I and II were sustained without shear failure. In some cases, cracking took place along short lengths of the contact surface between girder and slab; but there was no case of a general separation of the deck slab from the girders. The No. 3 stirrups spaced 9, 6, and 5 in. for the girders with 0.83, 1.66, and 2.49-per cent deck steel, respectively, were adequate to hold the two parts together.

Practical Design. It may be concluded from the tests of Groups I and II that the ultimate negative moment is governed by tension within the practical range of deck steel reinforcement ranging from 0.5 to 1.5 per cent. This is so, even for 0.6 per cent of straight prestressing reinforcement providing an effective concrete prestress of 2000 psi. For the less severe cases of girders with end blocks or draped prestressing reinforcement, tension control is then evident. In calculations of ultimate negative moment, prestress effects should be checked by Eqs. (1) to (3), but in most cases such effects can probably be neglected.

Group III

The three girders of Group III were tested to explore the strength and moment redistribution of continuous girders. Accordingly, observed strength and behavior is compared with calculations based on limit design principles.

Limit Design Ultimate Load. The limit design load for Group III girders was calculated as the load required to form three plastic hinges, one over each support, and one in the region of maximum positive moment. The necessary mechanism for collapse is then formed. The ultimate negative moments given in Table 2 were calculated by the rectangular stress block, including prestress effects. The ultimate positive moments were calculated according to Section 209.2.1. of "Tentative Recommendations for Prestressed Concrete" by ACI-ASCE Joint Committee 323(9). The limit design load, P_1 in Fig. 2, was determined as twice the algebraic sum of the calculated negative and positive moments for each girder divided by the 88-in. moment arm.

The observed ultimate negative moments given in Table 2 represent the average tie-down force as measured by load cells times the cantilever moment arm plus the dead-load moment of the cantilever. One half of the load applied in the girder span, including dead load effects and the weight of the load-distributing steel girder seen in Fig. 3, was multiplied by the 88-in. moment arm to obtain the sum of observed negative and positive moment. The observed ultimate positive moment was then obtained by subtracting the observed negative moment from this sum.

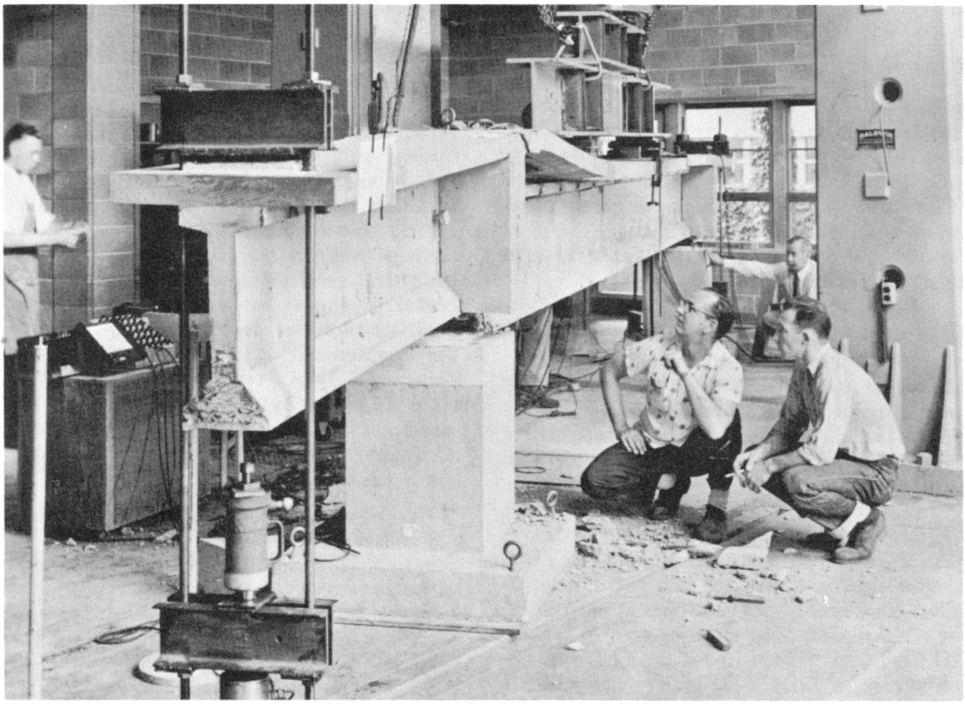
Ultimate Strength. As shown by the deck steel strains in Table 3, general yielding of the deck reinforcing steel developed over the supports of all girders well before ultimate strength was reached. The positive moment in the field then increased at a more rapid rate as redistribution of moments took place. The mid-span deflection of all girders exceeded 3 in. at ultimate strength.

Strength characteristics of the girders are summarized in Table 4. It is seen that the two girders with 0.83-per cent deck steel developed negative moments in excess of the calculated values, while the positive moments were slightly less than those calculated. Full redistribution of moments took place so that the ultimate loads

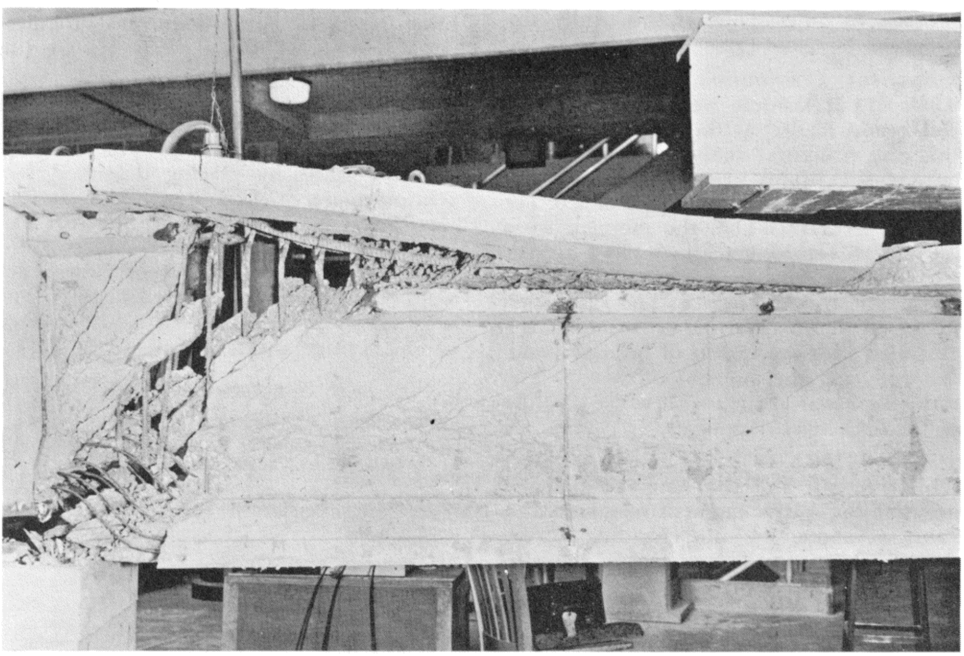
TABLE 4 — STRENGTH OF GROUP III GIRDERS

Girder No.	M_{test}/M_{calc}		Limit design ultimate load, P_1		
	Negative	Positive	Test, Kips	Calc., Kips	P_{test}/P_{calc}
III-0.6-0.83a	1.31	0.94	124.3	114.8	1.08
III-0.6-0.83b	1.42	0.99	131.0	113.4	1.16
III-0.6-1.66	1.24	0.70	142.3	144.5	0.98*

* Failure by horizontal shear.



(a) — General View.



(b) — Failure Region.

Fig. 7 — Girder III-0.6-1.66 After Test.

exceeded those calculated by limit design by 8 and 16 per cent, respectively. The final failure of Girder III-0.6-0.83a took place by crushing of the lower girder flange near one of the support diaphragms. Girder III-0.6-0.83b failed by crushing of the deck slab in the positive moment region near one of the loads applied in the girder span. The strength of both girders was controlled by flexure. The horizontal shearing stresses of 336 and 353 psi, given in Table 3, were sustained safely.

Girder III-0.6-1.66 contained 1.66 per cent deck reinforcement, and it therefore sustained negative moments higher than the other two girders of this group. At 80 per cent of the ultimate test load, horizontal cracks formed at the slab-to-girder contact surface, between the diaphragm and girder loading point. These cracks gradually lengthened and widened until the deck slab buckled from the girder as the ultimate load was reached at a horizontal shearing stress of 385 psi. The ultimate load was 98 per cent of that calculated by limit design. The rupture area of Girder III-0.6-1.66 is shown in Fig. 7. This was the only case of failure by either shearing or buckling of the deck slab. Further studies of horizontal shearing strength are in progress in other stages of this bridge test program.

Moment Distribution. Girders III-0.6-0.83b and III-0.6-1.66 were tested as fixed-end beams loaded at the third-points. For this case structural analysis for homogeneous, elastic beams gives a ratio between negative and positive moments of 2.0. For the Girder III-0.6-1.66, the moment ratio actually observed remained close to the value of 2.0 until horizontal cracking began at the contact surface. The ratio then increased to 2.5 since the horizontal crack extended into the region of positive bending, reducing the moment of inertia. The ratio decreased again to 2.0 when yielding of the deck steel developed.

Girder III-0.6-0.83b had a light negative reinforcement, and the observed ratio of negative to positive moment remained at a value of about 1.4 until yielding of the deck steel began at 65 per cent of the ultimate load. The ratio then decreased until ultimate strength was reached at a ratio of 0.91.

In this manner, these tests confirm previous experiments[10,11] which show that flexural cracks developing at loads well below the working load result in a change of the moments of inertia and, consequently, alter the moment distribution. The amount by which observed moments vary from those calculated by elastic theory depends on the relative amount of reinforcement in the various sections. If the sections of negative bending in a fixed-end beam are lightly reinforced as compared to the sections of positive bending, moment redistribution at service loads will result in a decrease of the ratio between negative and positive moment, as the cracking will be more extensive at the lightly reinforced sections. A heavy reinforcement at the sections of negative bending, on the other hand, will result in moment redistribution in a direction opposite to that of the previous case.

Practical Design. It may be expected on the basis of these tests that, within the practical range of deck reinforcement percentage, the continuity connection used permits sufficient redistribution of moments to develop an ultimate strength at least equal to that calculated by limit design. The extent to which limit design can be used safely in practical design of bridges of this type is being given further study in other stages of this bridge test program.

CONCLUSION

This pilot investigation was carried out to explore properties of a continuity connection between precast-prestressed girders established by placing deformed bar reinforcement in the situ-cast deck slab. It was found that such a continuity connection is fundamentally sound, and suitable fundamental design criteria were developed. Further studies of this connection are, therefore, in progress in other stages of this bridge investigation.

REFERENCES

1. Leonhardt, F., **"Spannbeton fur die Praxis,"** (Prestressed Concrete for Practice), Berlin, Wilhelm Ernst und Sohn, 472 pages, 1955.

2. Eeckhout, A. D., "Overhead Pipe-Line Gantry with Precast-Prestressed Girders," Amsterdam, *Cement-Beton* **7,** No. 7-8, 214-215 (August, 1955).

3. Kroes, M., and Groenhart, P., "De Steiger Voor de Derde Pointverbinding te Amsterdam," (Approach Jetty for a Ferry at Amsterdam), Amsterdam, *Cement-Beton* **8,** No. 23-24, 563-570 (December, 1956).

4. ANON, "The Northam Bridge, Southampton, England," Copenhagen, *Fifty Years of Civil Engineering 1904-1954*, Christiani and Nielsen, 131-136 (February, 1954).

5. Bouvy, J. J. B. J. J., "A Special Method of Assembling Precast Beams into Continuous Structures by Prestressing on the Site," Amsterdam, **Federation Internationale de la Precontrainte, Second Congress**, Session 2, Paper No. 10, 414-419, 1955.

6. Bender, M. E., "Prestressed Concrete Bridges for the Illinois Toll Highway," *Proceedings, World Conference on Prestressed Concrete, San Francisco, July, 1957*, A11, 1-11.

7. Janney, J. R., and Eney, W. J., "Full-Scale Test of Bridge on Northern Illinois Toll Highway," *Proceedings, World Conference on Prestressed Concrete, San Francisco, July 1957*, A12, 1-18.

8. Janney, J. R., "Nature of Bond in Pre-Tensioned Prestressed Concrete," *Journal of the American Concrete Institute* (May, 1954); *Proceedings*, **50**, 717-736 (1954), *PCA Development Bulletin D2*.

9. ACI-ASCE Joint Committee 323, "Tentative Recommendation for Prestressed Concrete," *Journal of the American Concrete Institute* (January, 1958); *Proceedings*, **54**, 545-578 (1958).

10. Hajnal-Konyi, K., and Lewis, H. E., "Moment Redistribution in Continuous Beams Reinforced with Plain Deformed Bars," *Symposium on the Strength of Concrete Structures* Paper 4, 36 pp., London, May 1956.

11. Mattock, A. H., "Redistribution of Design Bending Moments in Reinforced Concrete Continuous Beams," *Proceedings of the Institution of Civil Engineers*, **13**, 35-46 (London), May, 1959, *PCA Development Bulletin D30*.

Bulletins Published by the Development Department Research and Development Laboratories of the Portland Cement Association

D1 —"Influence of Soil Volume Change and Vegetation on Highway Engineering," by E. J. FELT.

Reprinted from *Twenty-Sixth Annual Highway Conference of the University of Colorado,* January, 1953, 52-76.

D2 —"Nature of Bond in Pre-Tensioned Prestressed Concrete," by JACK R. JANNEY.

Reprinted from *Journal of the American Concrete Institute* (May, 1954); *Proceedings,* **50,** 717-736 (1954).

D2A—Discussion of the paper "Nature of Bond in Pre-Tensioned Prestressed Concrete," by P. W. ABELES, K. HAJNAL-KONYI, N. W. HANSON and Author, JACK R. JANNEY.

Reprinted from *Journal of the American Concrete Institute* (December, Part 2, 1954); *Proceedings,* **50,** 736-1 to 736-11 (1954).

D3 —"Investigation of the Moisture-Volume Stability of Concrete Masonry Units," by JOSEPH J. SHIDELER.

Published by Portland Cement Association, Research and Development Laboratories, Skokie, Illinois, March, 1955.

D4 —"A Method for Determining the Moisture Condition of Hardened Concrete in Terms of Relative Humidity," by CARL A. MENZEL.

Reprinted from *Proceedings, American Society for Testing Materials,* **55,** 1-26 (1955).

D5 —"Factors Influencing Physical Properties of Soil-Cement Mixtures," by EARL J. FELT.

Reprinted from *Highway Research Board Bulletin 108,* 138-162 (1955).

D6 —"Concrete Stress Distribution in Ultimate Strength Design," by E. HOGNESTAD, N. W. HANSON and D. MCHENRY.

Reprinted from *Journal of the American Concrete Institute* (December, 1955); *Proceedings,* **52,** 455-479 (1956).

D6A (part of D6)—Discussion of the paper, "Concrete Stress Distribution in Ultimate Strength Design," by P. W. ABELES, A. J. ASHDOWN, A. L. L. BAKER, ULF BJUGGREN, HENRY J. COWAN, HOMER M. HADLEY, KONRAD HRUBAN, J. M. PRENTIS, E. ROSENBLUETH, G. M. SMITH, L. E. YOUNG, A. J. TAYLOR, and Authors, E. HOGNESTAD, N. W. HANSON, and D. MCHENRY.

Reprinted from *Journal of the American Concrete Institute* (December, Part 2, 1956); *Proceedings,* **52,** 1305-1330 (1956).

D7 —"Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams," by J. R. JANNEY, E. HOGNESTAD and D. MCHENRY.

Reprinted from *Journal of the American Concrete Institute* (February, 1956); *Proceedings*, **52**, 601-620 (1956).

D8 —"Resurfacing and Patching Concrete Pavement with Bonded Concrete," by EARL J. FELT.

Reprinted from *Proceedings of the Highway Research Board*, **35**, 444-469 (1956).

D9 —"Review of Data on Effect of Speed in Mechanical Testing of Concrete," by DOUGLAS MCHENRY and J. J. SHIDELER.

Reprinted from *ASTM Special Technical Publication No. 185*, 72-82 (1956). American Society for Testing Materials.

D10—"Laboratory Investigation of Rigid Frame Failure," by R. C. ELSTNER and E. HOGNESTAD.

Reprinted from *Journal of the American Concrete Institute* (January, 1957); *Proceedings*, **53**, 637-668 (1957).

D11—"Tests to Evaluate Concrete Pavement Subbases," by L. D. CHILDS, B. E. COLLEY and J. W. KAPERNICK.

Reprinted from the *Journal of the Highway Division, Proceedings of the American Society of Civil Engineers, Proc. Paper No. 1297*, **83**, 1-41, HW3, July, 1957.

D12—"Ultimate Strength of Reinforced Concrete in American Design Practice," by E. HOGNESTAD.

Reprinted from *Proceedings of a Symposium on the Strength of Concrete Structures*, London, May, 1956.

D13—"Effect of Variations in Curing and Drying on the Physical Properties of Concrete Masonry Units," by WILLIAM H. KUENNING and C. C. CARLSON.

Published by Portland Cement Association, Research and Development Laboratories, Skokie, Illinois, December, 1956.

D14—"Lightweight Aggregates for Concrete Masonry Units," by C. C. CARLSON.

Reprinted from *Journal of the American Concrete Institute* (November, 1956); *Proceedings*, **53**, 491-508 (1956).

D15—"Confirmation of Inelastic Stress Distribution in Concrete," by EIVIND HOGNESTAD.

Reprinted from the *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Proc. Paper 1189*, **83**, 1-17, ST 2, March, 1957.

D16—"Strength and Elastic Properties of Compacted Soil-Cement Mixtures," by EARL J. FELT and MELVIN S. ABRAMS.

Reprinted from *ASTM Special Technical Publication No. 206*, 152-178 (1957). American Society for Testing Materials.

D17—"Lightweight Aggregate Concrete for Structural Use," by J. J. SHIDELER.

Reprinted from *Journal of the American Concrete Institute* (October, 1957); *Proceedings*, **54**, 299-328 (1957).

D18—"Stresses in Centrally Loaded Deep Beams," by P. H. KAAR.

Reprinted from *Proceedings of the Society for Experimental Stress Analysis,* **15,** No. 1, 77 (1957).

D19—"Strength of Concrete Under Combined Tensile and Compressive Stresses," by DOUGLAS MCHENRY and JOSEPH KARNI.

Reprinted from *Journal of the American Concrete Institute* (April, 1958); *Proceedings,* **54,** 829-839 (1958).

D20—"General Considerations of Cracking in Concrete Masonry Walls and Means for Minimizing It," by CARL A. MENZEL.

Published by Portland Cement Association, Research and Development Laboratories, Skokie, Illinois, September, 1958.

D21—"Tests of Concrete Pavement Slabs on Gravel Subbases," by L. D CHILDS and J. W. KAPERNICK.

Reprinted from the *Journal of the Highway Division, Proceedings of the American Society of Civil Engineers, Proc. Paper 1800,* **84,** 1-31, HW 3, October, 1958

D22—"Shear Strength of Lightweight Reinforced Concrete Beams," by J. A. HANSON.

Reprinted from *Journal of the American Concrete Institute* (September, 1958); *Proceedings,* **55,** 387-403 (1958).

D23—"Performance of Subbases for Concrete Pavements Under Repetitive Loading," by B. E. COLLEY and J. NOWLEN.

Reprinted from *Highway Research Board Bulletin 202,* 32-58 (1958).

D24—"Precast Concrete Girders Reinforced with High Strength Deformed Bars," by J. R. GASTON and EIVIND HOGNESTAD.

Reprinted from *Journal of the American Concrete Institute* (October, 1958). *Proceedings,* **55,** 469-484 (1958).

D25—"Concrete Beams and Columns with Bundled Reinforcement," by N. W. HANSON and HANS REIFFENSTUHL.

Reprinted from the *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Proc. Paper 1818,* **84,** 1-23, ST 6, October, 1958.

D26—"Review of Limit Design for Structural Concrete," by C. W. YU and EIVIND HOGNESTAD.

Reprinted from the *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Proc. Paper 1878,* **84,** 1-28, ST 8, December, 1958.

D27—"Laboratory Tests of Sealers for Sawed Joints," by WILLIAM KUENNING.

Reprinted from *Highway Research Board Bulletin 211,* 1-12 (1959).

D28—"Flexural Bond Tests of Pre-Tensioned Prestressed Beams," by NORMAN W. HANSON and PAUL H. KAAR.

Reprinted from *Journal of the American Concrete Institute* (January, 1959); *Proceedings,* **55,** 783-802 (1959).

D29—"Effects of Research on Modern American Structural Concrete Design," by EIVIND HOGNESTAD.

Reprinted from the *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Proc. Paper 2034,* **85,** 1-9, ST 5, May, 1959.

D30—"Redistribution of Design Bending Moments in Reinforced Concrete Continuous Beams," by ALAN HANSON MATTOCK.

Reprinted from the *Proceedings of the Institution of Civil Engineers, Paper No. 6314,* **13,** 35-46, London, May, 1959.

D31—"Ultimate Strength Criteria for Reinforced Concrete," by LADISLAV B. KRIZ.

Reprinted from the *Journal of the Engineering Mechanics Division, Proceedings of the American Society of Civil Engineers, Proc. Paper 2095,* **85,** EM 3, 95-110, July, 1959.

D32—"Shear Strength and Elastic Properties of Soil-Cement Mixtures Under Triaxial Loading," by GLENN G. BALMER.

Reprinted from *Proceedings, American Society for Testing Materials,* **58,** 1187-1204 (1958).

D33—"Facilities and Test Methods of PCA Structural Laboratory," by EIVIND HOGNESTAD, N. W. HANSON, LADISLAV B. KRIZ and OTTO A. KURVITS.

Reprinted from the *Journal of the PCA Research and Development Laboratories,* **1,** No. 1, 12-20, 40-44 (1959); **1,** No. 2, 30-37 (1959); **1,** No. 3, 35-41 (1959).

D34—"Precast-Prestressed Concrete Bridges 1. Pilot Tests of Continuous Girders," by PAUL H. KAAR, LADISLAV B. KRIZ and EIVIND HOGNESTAD.

Reprinted from the *Journal of the PCA Research and Development Laboratories,* **2,** No. 2, 21-37 (May, 1960).

Printed in U.S.A.