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PRECAST-PRESTRESSED CONCRETE BRIDGES
2. HORIZONTAL SHEAR CONNECTIONS

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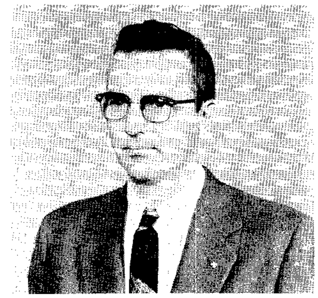
Precast-Prestressed Concrete Bridges 2. Horizontal Shear Connections



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SYNOPSIS

An extensive investigation of precast prestressed bridges is being reported in a series of Portland Cement Association Development Department bulletins. This bulletin presents an initial study of composite action between precast girders and a situ-cast deck slab. Sixty-two push-off specimens and ten T-shaped girders were tested to explore various means of horizontal shear transfer at the contact surface between precast and situ-cast concrete in composite construction. Adhesive bond, roughness, keys and stirrups were included as test variables. Experimental results are given in terms of a basic shearing strength related to the concrete-to-concrete joint characteristics, plus a term related to the percentage of stirrup reinforcement.

BRIDGE TEST PROGRAM

An extensive bridge test program was outlined in the Portland Cement Association Development Department bulletin D34, "Precast-Prestressed Concrete Bridges, Part 1 – Pilot Tests of Continuous Girders". The type of bridge involved consists of I-shaped longitudinal girders connected laterally by a continuous situ-cast deck slab. Though each bridge span consists of separate precast girders, continuity is established also in the longitudinal direction subsequent to erection of the girders. Bulletin D34 presented the results of an investigation regarding the feasibility of establishing continuity between precast girders from span to span. Continuity for live loads was obtained by placing deformed bar reinforcement in the deck slab across the girder supports to transfer negative bending moments.

This development of improved precast prestressed concrete bridges involves a combined application of precast and situ-cast concrete. Precast girders and a situ-cast deck slab can work together efficiently as a T-section only if adequate transfer of horizontal shear exists. The investigation reported herein was therefore undertaken, as a second stage of the bridge test program, to explore various means of horizontal shear transfer at the contact surface in composite construction.

COMPOSITE CONSTRUCTION

The term "composite girder" usually indicates the combined use of different construction materials, such as concrete and structural steel or concrete and timber. However, this term also denotes the application of a situ-cast concrete floor slab or a bridge deck to a precast concrete girder. If the situ-cast deck concrete is to be considered in design as increasing the stiffness and strength of the precast girder section by acting as a compression zone in regions of positive bending moments, and if reinforcement placed in the deck slab is to resist negative girder moments, then composite action must be provided between the two parts. To be considered as truly composite, shear forces must be transmitted across the contact surface between the two pieces in the same manner and with the same deformations as if the entire section were monolithically-cast structural concrete. If there is a weakness at the contact surface, the member is only partially composite, with stiffness characteristics between those of the composite and the two-piece system.

Practical use of composite concrete construction has been reported in the literature principally in connection with prestressed concrete. Tests of a full scale prestressed concrete girder with a situ-cast top slab was reported by Dean and Ozell(1). The precast girder had a wood float finish on the contact surface, and vertical stirrups were used to connect the girder to the slab. The horizontal shearing stress at the connection reached a calculated maximum of 265 psi when the girder failed in flexure without damage to the joint. Three other reports (2,3,4) give similar data of tests in which the connection between precast and situ-cast concrete remained intact. Although these reports do not indicate the ultimate strength capacity of a bonded connection, they lend confidence to the use of composite construction for both buildings and bridges.

A report by Evans and Parker(5) describes tests in England of composite girders in which the precast portion was surrounded on three sides by situ-cast concrete. In this case, lateral shrinkage of the situ-cast concrete was expected to aid the natural bond of the connection. Quantitative information is not available on joint shearing stress at failure; but it was concluded from these tests that good bond could be obtained by roughening the surface, and the beam would then act monolithically.

An experimental study of bond by Felt(6) indicated that shearing strengths at the contact surface between old pavements and a new resurfacing concrete of 250 to 500 psi can be expected. The highest strengths were generally obtained when the old surface was dry and slightly rough. The report emphasized the great importance of having the old surface clean and free of laitance and other inferior material when the new concrete is cast.

To extend the data regarding composite girders available in the literature, an exploratory test series was carried out at the Portland Cement Association Research and Development Laboratories during 1957-58. Push-off tests of small specimens and tests of T-shaped girders were made to develop quantitative information regarding horizontal shearing strength. Bond, roughness, keys, and stirrups were used separately and in various combinations to provide horizontal shear connections.

PUSH-OFF TESTS

Test Specimen and Materials

The push-off specimen shown in Fig. 1 was used to explore the load-deformation characteristics of various contact surfaces subjected to a shearing force. Each test specimen was composed of two parts. The precast girder part was rectangular with an 8-in. width and a 12-in. depth. The situ-cast top deck slab was 7 in. thick and 24 in. wide. The contact length between the girder and slab parts was a variable: 6, 12, or 24 in. were used. Fig. 2 shows the forms prepared to cast a slab on a girder.

Most specimens contained stirrups arranged in the same manner as in customary construction. The stirrups were U-shaped with the open end extending four inches into the slab concrete from the precast girder section. Generally, these stirrups were positioned at the center of the shear length. However, in a few cases two stirrups were used at the quarter points of the shear length, or three stirrups were used at the quarterpoints and center.

When keys were used, they were formed as depressions 2½ in. deep. Blocks of foamed plastic 5x5x2½ in. were placed in the girder concrete to form the keys. The plastic was removed before casting of the slab.

Effects of concrete strength were not investigated systematically in this series of tests.

The two concretes used were made from a blend of Type I cements with Elgin sand and gravel of ¾-inch maximum size. Water-cement ratio by weight was 0.64 and 0.50, for slabs and girders, respectively. Mixing took place in a 6-cubic foot non-

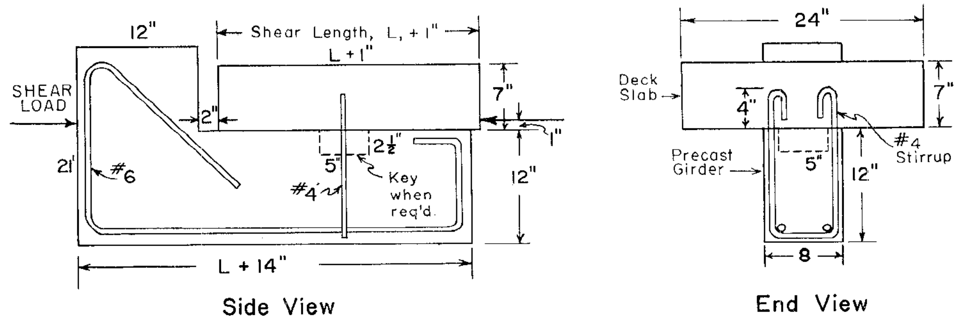

Fig. 1 — Pushoff Specimen in Casting Position.

tilting mixer, and both the specimens and 6x12-in. cylinders were compacted with an internal spud vibrator. Both slab and girder concretes had approximately 5 percent intentionally entrained air.

The reinforcement crossing the contact surface, in the form of U-shaped stirrups, was of intermediate grade with deformations conforming to ASTM A-305. The individual yield points are shown in Table 1.

The specimens were identified by the code shown in Table 1. For example, a specimen identified by the letters BRKS had a contact surface which was bonded (B), rough (R), with a key (K), and stirrups (S) crossing the joint. In all cases, if any of these letters is absent from the designation, the opposite variable was involved. Thus, if (B) is absent, adhesive bond was destroyed; if (R) is missing, the contact surface was smooth, etc. Following the letter designations, a number—6, 12, or 24 indicates the shear length involved. The final number following the dash serves to identify successive companion specimens.

Nature of Contact Surfaces

Various treatments of the contact surface between the girder and slab part of the specimens were as follows:

Smooth. Contact surface trowelled to a relatively smooth condition.

Rough. Contact surface roughened by scraping the concrete with the edge of a sheet of steel. No attempt was made to smooth the aggregate into the paste. The final finish was one with depressions and peaks approximately 3/8-in below and above the average level.

Bond. No attempt made to destroy adhesive bond. Deck concrete cast directly on to a dry girder surface.

Unbonded. Contact surface painted with a silicone compound which prevented the new concrete from bonding to the hardened concrete.

Smooth Aggregate Bare. After trowelling, a retarding compound (Rugasol) was applied to prevent set of the top fraction of an inch. Twenty-four hours later this top paste was washed away with a water jet, leaving the top aggregate bare of paste. The roughness was approximately 1/16-inch above and below the average level.

Rough Aggregate Bare. Following the

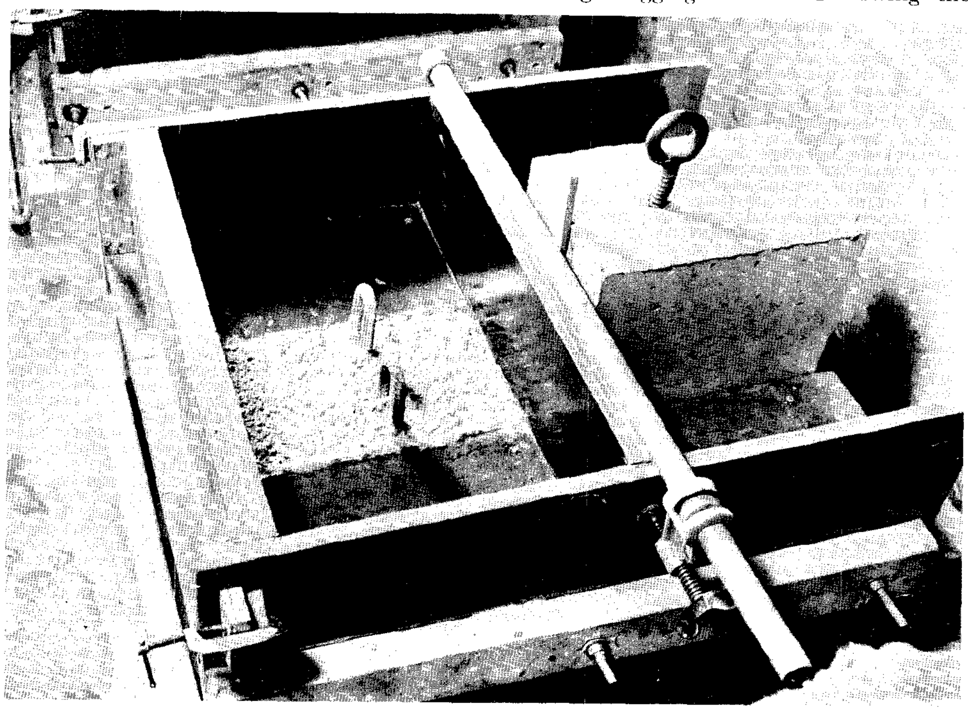

Fig. 2.— Forms Prepared for Slab Casting.

TABLE I — PUSH-OFF TEST RESULTS (Stirrup effects subtracted)

Speci-men No.	Length inches	Con-tact Sur-face	Stirrup Yield Point psi	Concrete Strength psi		Initial Peak Values'		Average Stress in psi at noted inches of slip				Notes
				Slab	Girder	Stress psi	Slip inches	0.005	0.010	0.015	0.020	
BRS6-1	6		50,000	3500	4610	680	0.0090	615	680	625	525	
BRS6-2	6		50,000	3240	5320	395	0.0020	355	350	340	330	
BRS6-3	6		51,000	3700	5320	640	0.0030	620	570	540	507	
BRS12-1	12		48,500	4130	5670	490	0.0020	487	510	467	412	
BRS12-2	12		47,000	3580	4970	455	0.0012	496	540	—	—	
BRS12-3	12		51,000	3310	4080	350	0.0012	295	255	245	230	
BRS12-4	12		51,000	3310	4080	355	0.0012	393	390	340	300	Rough—Aggr. Bare
BRS12-5	12	Rough	51,000	3310	4080	310	0.0010	343	240	197	175	Smooth—Aggr. Bare
BRS12-6	12		50,000	3960	4420	365	0.0012	291	260	245	232	
BRS12-7	12		50,000	3960	4420	430	0.0014	390	350	300	245	Smooth—Aggr. Bare
BRS12-8	12	and	50,000	3960	4420	440	0.0014	435	448	420	372	Rough—Aggr. Bare
BR12-1	12		none	3040	4960	416	0.0011	—	—	—	—	
BR12-2	12		none	3980	5340	555	0.0018	—	—	—	—	
BR12-3	12	Bonded	none	4150	5270	455	0.0014	—	—	—	—	
BR12-4	12		none	4080	4990	350	0.0008	—	—	—	—	Rough—Aggr. Bare
BR12-5	12		none	4080	4990	362	0.0009	—	—	—	—	
BR12-6	12		none	3720	5050	410	0.0010	—	—	—	—	Rough—Aggr. Bare
BR12-7	12		none	3720	5050	408	0.0014	—	—	—	—	Rough—Aggr. Bare
BR12-8	12		none	3720	5050	405	0.0012	—	—	—	—	Rough—Aggr. Bare
BRS24-1	24		49,000	3540	5740	467	0.0007	455	468	445	408	2 Stirrups
BRS24-2	24		52,000	3430	4610	345	0.0006	—	—	—	145	2 Stirrups
BRS24-3	24		50,000	3420	5000	400	0.0010	357	364	350	330	3 Stirrups
BRS24-4	24		50,000	3510	6040	445	0.0008	442	470	470	460	3 Stirrups
BS6-1	6		50,000	3240	5320	157	0.0003	110	90	80	70	
BS6-2	6		50,000	3240	5320	225	0.0009	165	140	145	135	
BS6-3	6		50,000	3700	5070	230	0.0015	170	155	145	135	
BS6-4	6		50,000	3700	5070	215	0.0010	132	115	115	110	
BS6-5	6		50,000	3700	5070	240	0.0013	150	130	120	115	
BS12-1	12	Bond	50,150	4050	4870	165	0.0005	15	15	10	0	
BS12-2	12	Only	50,150	3660	5170	110	0.0006	40	40	40	30	
B12-1	12		none	4050	4870	125	0.0007	—	—	—	—	
B12-2	12		none	3660	5170	230	0.0005	—	—	—	—	
B12-3	12		none	3980	5340	130	0.0005	—	—	—	—	
B12-4	12		none	4150	5270	90	0.0002	—	—	—	—	
B12-5	12		none	4080	4990	120	0.0003	—	—	—	—	
B24-1	24		none	4220	4660	109	0.0003	—	—	—	—	
B24-2	24		none	4220	4660	94	0.0003	—	—	—	—	
B24-3	24		none	4220	4660	100	0.0003	—	—	—	—	
RS6-1	6		50,000	3500	4620	—	—	222	245	248	250	
RS6-2	6		51,000	3700	5320	—	—	307	328	320	300	
RS12-1	12		48,500	4130	5670	—	—	300	307	307	280	
RS12-2	12	Roughness	47,000	3580	4970	—	—	195	215	222	220	
RS24-1	24	Only	49,000	3540	5740	—	—	215	232	232	208	2 Stirrups
RS24-2	24		52,000	3430	4610	—	—	177	195	197	190	2 Stirrups
RS24-3	24		50,000	3420	5000	—	—	230	255	250	232	2 Stirrups
RS24-4	24		49,000	3510	6040	—	—	302	320	312	290	2 Stirrups
KS12-1	12	Keys†	48,500	3510	5370	—	—	692*	697*	732*	732*	
KS12-2	12	in Smooth	50,000	3490	4520	—	—	617*	750*	808*	808*	
KS24-1	24	Unbonded	49,000	3620	4880	655*	0.0014	770*	1020*	1040*	982*	3 Stirrups, 2 Keys
KS24-2	24	Surfaces	50,000	4150	5250	—	—	655*	828*	885*	867*	3 Stirrups, 2 Keys
RKS12-1	12	Keys†	48,500	3510	5370	290	0.0014	270	305	320	300	
RKS12-2	12	in	50,000	3490	4520	290	0.0018	310	330	330	315	
RKS12-1	12	Rough	none	3040	5420	250	0.0014	270	273	255	—	
RKS24-1	24	Unbonded	50,000	3620	4880	220	0.0020	250	255	—	—	3 Stirrups, 2 Keys
RKS24-2	24	Surfaces	51,000	4150	5250	240	0.0020	285	340	360	350	3 Stirrups, 2 Keys
BRKS12-1	12		48,500	3510	5370	440	0.0020	445	440	410	370	
BRKS12-2	12	Keys†	50,000	3490	4520	455	0.0016	420	395	365	340	
BRK12-1	12	in	none	3040	5420	440	0.0015	420	470	—	—	
BRK12-2	12	Rough	none	3980	5340	545	0.0020	—	—	—	—	
BRK12-3	12	Bonded	none	4150	5270	415	0.0012	442	452	—	—	
BRKS24-1	24	Surfaces	50,000	3620	4880	445	0.0010	435	490	400	380	3 Stirrups, 2 Keys
BRKS24-2	24		51,000	4150	5250	440	0.0011	425	475	470	440	3 Stirrups, 2 Keys

†All keys 5 x 5-in. extending 2½ in. into beam surface; 5 x 5-in. key section (root area) is 26 per cent of contact area.

*Stress based on root area of key.

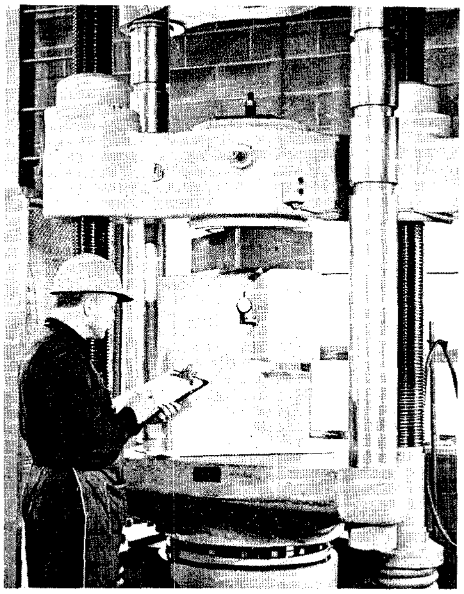

Fig. 3 — Pushoff Test.

rough treatment, the surface paste was prevented from setting and was removed twenty-four hours later with a water jet. The roughness was approximately the same as the "rough" above, but the projecting aggregate was bare of paste.

Stirrups. When stirrups were included, they were ½-inch intermediate grade deformed bars in the shape of a "U" with hooks on the ends. The hooked ends projected from the girder into the slab 4 inches. These stirrups were at the center of the joint length in most cases. Exceptions to this spacing are noted in a few cases in Table 1.

Keys. The keys included in these tests were 5 inches square in the direction of the shear force and 2½ inches deep into the girder concrete. Thus, the deck concrete filled a hole into the girder surface to form a key integral with the slab.

Casting and Curing

The precast girder part was cast with the contact surface at the top. The surface was trowelled or roughened as noted and the specimen was left to harden for one day in the form. The girder part was then stripped of its form and the slab form was mounted as shown in Fig. 2. The slab concrete was cast when the girder was approximately 24 hours old. The composite section was cured under wet burlap for seven days and then dried seven days before testing. Test cylinders were treated similarly and were tested the same day as the corresponding push-off specimens.

Test Method

The push-off specimen was rotated ninety degrees from the casting position to the testing position and placed in a 400,000 pound capacity hydraulic testing machine as shown in Fig. 3. The contact surface was vertical and the applied load tended to move the slab longitudinally with respect to the beam. The specimen was so placed that the resultant load entered the slab along a line parallel to the contact surface and one inch within the slab portion. The end of the girder contacting the lower head, and the end of the slab contacting the upper head, were bedded in rapid-hardening plaster of paris.

Movements of the slab with respect to the girder were measured by 0.0001-in. dial gages located at the center of the joint length on both sides of the beam as seen in Fig. 3. Load was applied in increments, and slip readings were taken at each load level. Two companion specimens were tested in most cases. One was loaded to failure in increments, and the other was loaded to failure with selected returns to zero load for measurement of residual slip.

TEST RESULTS FOR PUSH-OFF TESTS

Push-off specimens were tested with and without stirrups, and different contact lengths were used to provide stirrup reinforcement of different percentages. All push-off specimens without bond necessarily had stirrups to hold the parts together

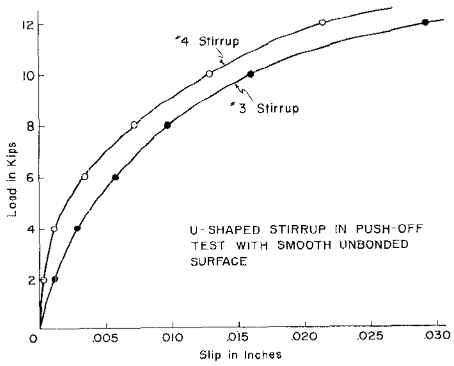

Fig. 4 — Load-Slip Curves for Stirrups Alone.

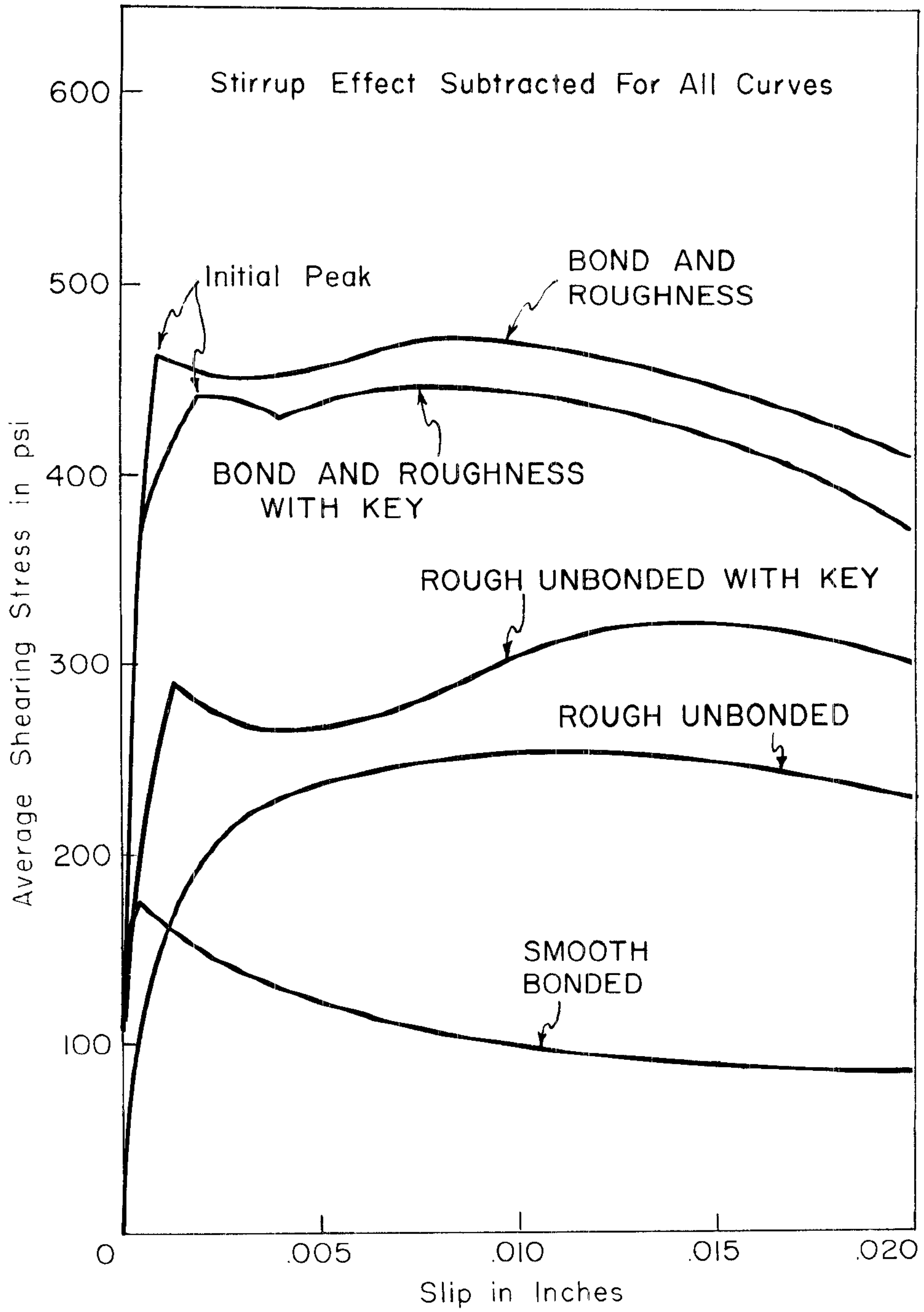

Fig. 5 — Typical Shear-Slip Curves.

during testing. In order to establish a common basis of comparison between various contact surfaces, the effect of stirrup reinforcement was isolated. To determine the effect of stirrups alone, auxiliary tests were made. Load-slip curves were determined for each stirrup spacing and shear length with the contact surface smooth and unbonded. The average load-deformation curve shown in Fig. 4 for No. 4 stirrups was chosen from these tests, and used in analysis of the push-off tests. The curve for the No. 3 stirrup shown in Fig. 4 was determined for use in analysis of girder tests.

Test results for bonded specimens indicated that the difference between the load-deformation curves for specimens with and without stirrups approximated closely the stirrup-only curve shown in Fig. 4. The load-slip curves for all push-off tests were therefore reduced to an effect of contact surface alone by subtracting the load-deformation curve for stirrup-only from the curves of all individual test specimens.

Effect of Bond

The nature of failure in the push-off tests is illustrated by the shear-slip curves in Fig. 5. These curves show typical relationships between slip deformation and shearing stress for the various types of connections. Detailed information from every push-off test is given in Table 1. It is noted in Fig. 5 and Table 1 that specimens for which bond was utilized as part of the connection, developed a high shearing stress at low slip. This might be called a rigid type of connection in contrast to the unbonded joints where considerable slip must take place before high shearing stress is reached.

Effect of Keys

The shear-slip curves in Fig. 5 and Table 1 for specimens with keys, bond and roughness indicate only slight changes attributable to the keys. This indicates that the contact area is acting as a unit and fails as a unit, without the key actually acting as a key. It appears that bond must be destroyed in order for a key to act. This is substantiated by the tests combining roughness and keys without bond. Some benefit from the keys was indicated by a slight increase in the average stress at the maximum points on the shear-slip curves for keys and rough surface as compared to rough surface alone. Also, the key added an initial peak stress at low slip which in effect made the connection more rigid.

For key connections in smooth unbonded surfaces, the initial peak is not characteristic. The shear-slip curves in Fig. 6 are based on the shear load divided by the root area of the key or keys. The shear-slip curves are similar in shape to the typical curve for rough unbonded specimens shown in Fig. 5. It is clear that the key connection is not rigid, and that considerable slip movement is required to develop the ultimate root stresses of 700 to 1000 psi. This again substantiated that bond must be destroyed before a key can act, so that the contributions of the two to shearing strength are not additive.

Effect of Shear Length

The effect of shear length can be seen in Fig. 7 in which initial peak stresses are plotted for all specimens covered in the group of variables: bond, roughness, and shear length. It will be recalled that the slip measurements were made at the mid-point of the contact length. There is a noticeable tendency for the shorter shear lengths to give higher average stresses when bond is utilized. This characteristic leads to the conclusion that in a push-off type of test, the high bond stress can only exist over a relatively short length, near the point of load application. Failure will be progressive from the load point along the contact surface length toward its free end. Investigations of bond between reinforcing steel and concrete have led to similar

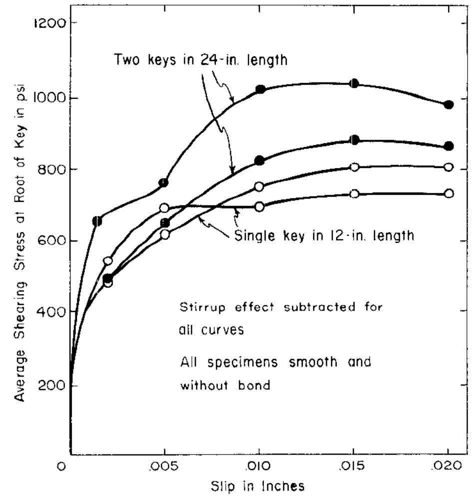

Fig. 6 — Shearing Stress at Root of Key.

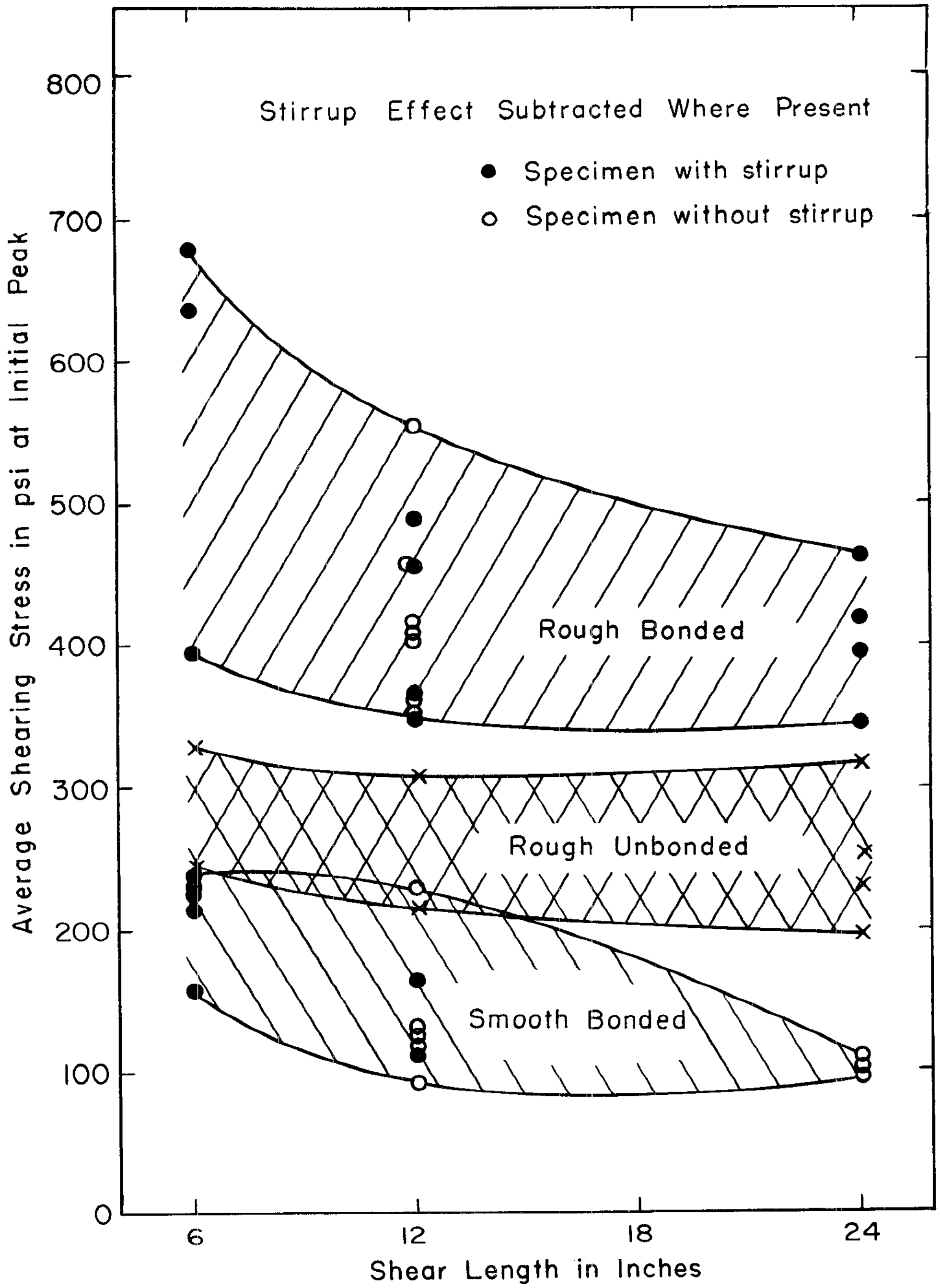

Fig. 7 — Effect of Shear Length.

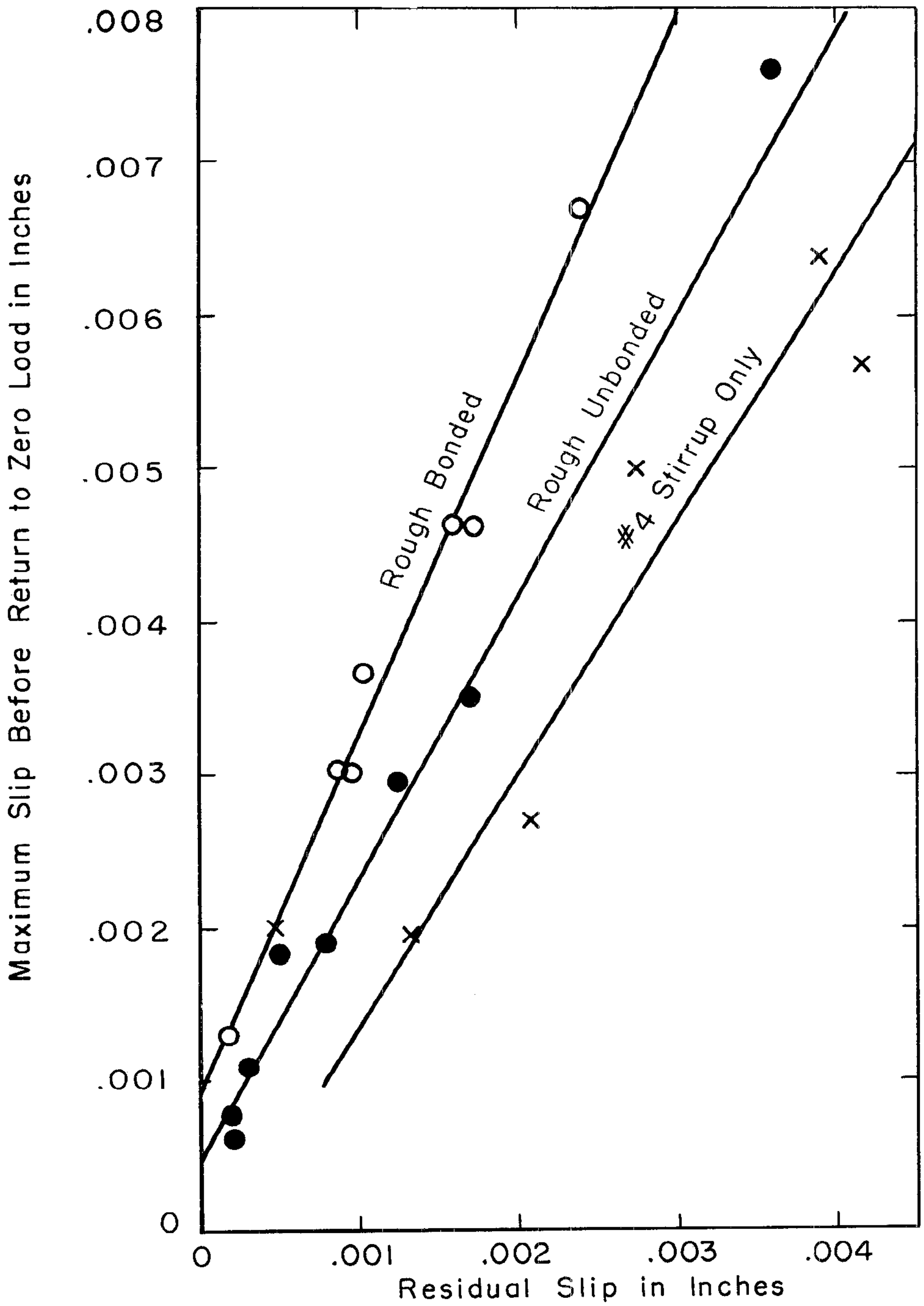

Fig. 8 — Residual Slip in Push-Off Tests.

conclusions regarding the embedment-length effect in pull-out tests of smooth bars.

In the tests with roughness without bond, there is less tendency for the shorter lengths to produce higher average stress. The plotted points for rough unbonded surfaces come from maximum points on the typical average shear-slip curves which occur at relatively high values of slip, e.g. 0.010 inch. This would indicate that the entire contact area can be active in resisting slip.

Effect of Concrete Compressive Strength

Concrete strength was not investigated systematically as a variable in this series of tests, but the specimens with rough bonded surfaces and 12-inch shear lengths provide some indication of the effect of variation in strength of the slab concrete. Initial peak values for these sixteen tests indicate a definite effect of concrete strength on initial peak shearing stress. Shearing stress appears to be approximately proportional to the concrete strength of the slab. However, further tests are needed before the effect of variation in concrete strength can be established with assurance.

Effect of Bare Aggregate

The application of a retarder to delay the set of the surface concrete of the precast beam was used on some specimens to provide a means of obtaining a rough surface with the exposed aggregate clean of paste. The data in Table 1 indicate that this treatment gives results comparable with the "rough and bonded" surface treatment method used in these tests. The results for trowelled surface with aggregate bare are also similar to "rough and bonded". This suggests that the shear-carrying capacity of the connection is not sensitive to variations in the depth of roughness.

Residual Slip Characteristics

Nineteen of the tests reported in Table 1 were conducted with selected returns to zero load during the testing procedure. By this method an evaluation of residual slip was possible.

The results of these tests are shown in Fig. 8, in which the residual slip is plotted versus the maximum slip reached prior to the return to zero load. In addition to the three curves shown, data for "smooth bonded" push-off tests follow the "rough bonded" line, but only as far as approximately 0.002-in. maximum slip, at which time large slips and large related residual slips develop. All combinations of surfaces which included keys also followed the residual slip curve shown in Fig. 8 for rough bonded surfaces.

Design Considerations

Previous studies of composite structural steel and concrete[7,8], have established a precedent of utilizing in design a stud or other flexible shear connector at a shearing strength corresponding to a residual slip of 0.003 in. This slip value was chosen to represent the maximum useful shearing capacity, as residual slip increases rapidly at loads above this so-called "critical load."

The concrete-to-concrete connections tested indicate similar performance when bond is absent. However, when roughness and bond are utilized together, an early peak stress is found at a very small slip, Fig. 5. This peak is followed by an increasing shearing capacity at increasing slip. The slope of the increase beyond the peak is dependent on the amount of stirrup reinforcement. In this case, the early peak stress at small slip shown in Fig. 7 is probably a suitable basis for practical design.

The shearing strength of keys cannot be added to the contribution of bond and roughness. It seems advisable for practical purposes, therefore, to avoid the use of keys and to rely on a combination of bond, roughness, and stirrups. If keys cannot be avoided, it may be necessary to assume that the entire shear force is transferred by the keys alone.

GIRDER TESTS

The composite girders used to test the action of horizontal shear connections in flexure were designed in such a way that the horizontal shear at the girder-slab contact surface reached high values at loads well below flexural failure. The section shown in Fig. 9 was so chosen that the neutral axis of bending strains was near the contact surface both before and after flexural cracking took place, based on full composite action at the joint.

The calculation of horizontal shearing stress was based on the usual equation $v = VQ/Ib$, in which Q is the first moment about the neutral axis of all areas from the horizontal section considered to the extreme compression edge. This equation was

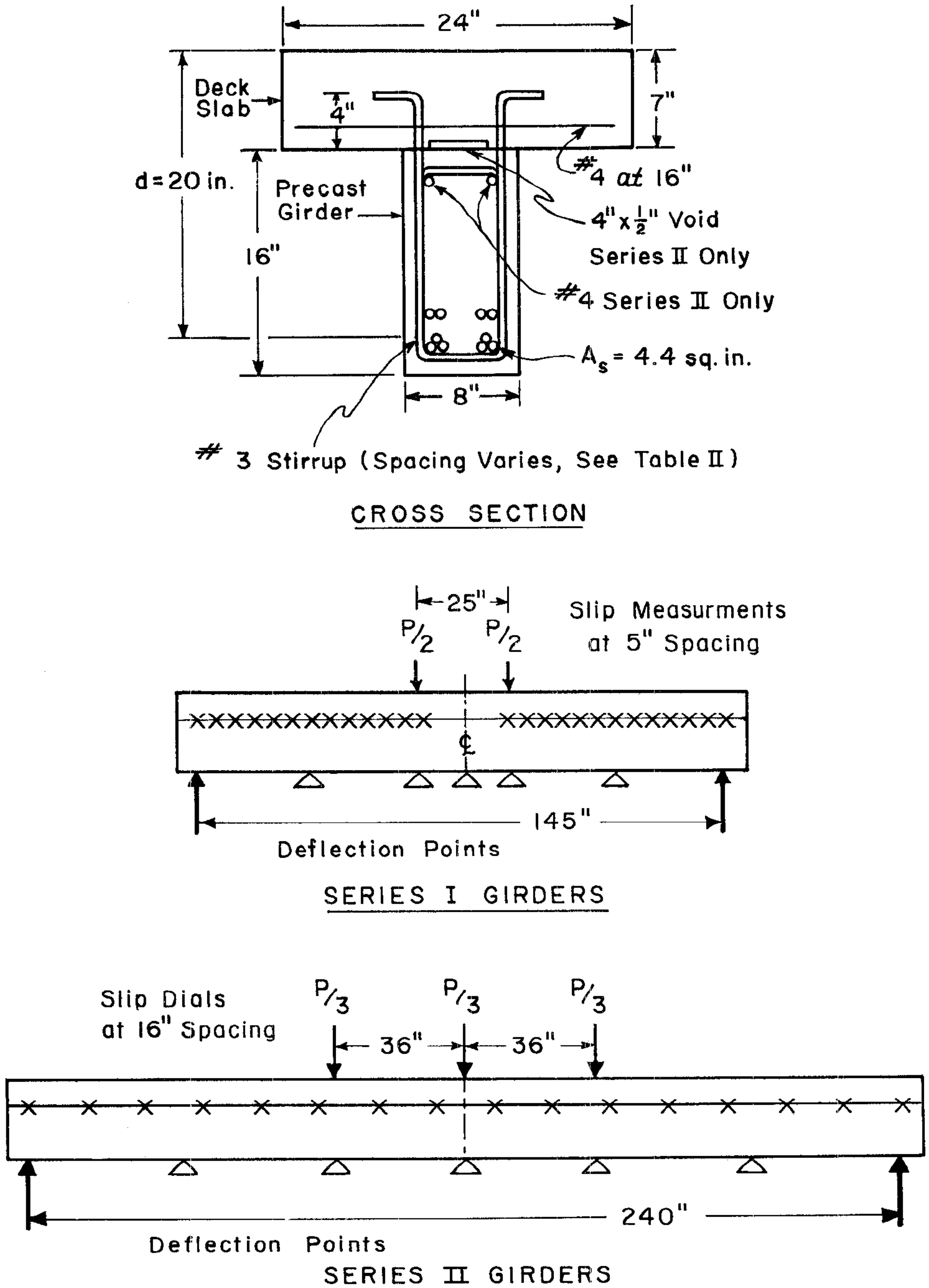

Fig. 9 — Girder Test Specimens.

applied to the contact surface section, considering the cracked transformed cross section of the T-shaped composite girder, and the resulting relationship between shearing force and shearing stress was used in relating stress to slip. The shearing force of the girder, V, is partly due to the live loading and partly due to dead weight. The deck was cast on the precast girder with uniform support under the girder. Thus the dead weight of the complete section, 300 pounds per foot, adds to the shear diagram. It will be noted later that the girder slip curves indicate that initial slip occurred near the quarter-point of the span. Because of this, the shearing stress used in preparation of shear-slip curves is derived from live load shear plus dead load shear at the quarter-point of the girder span.

The equation for horizontal shearing stress cannot be considered as an exact representation of actual stress conditions after discontinuities develop due to cracking, and especially after some slip has taken place. However, these calculated stresses do provide a common basis for comparison and are so used.

Ten girders were tested as shown in Fig. 9 in two series, six in Series I, and four in Series II. The girders of Series II have a reduced shear section at the contact surface. External dimensions and tensile reinforcement were the same for the two test series. In both groups the variables of bond and roughness combined with stirrup reinforcement were explored as shown in Table 2. In addition to a monolithically cast control specimen in each series, Series I included two girders made without stirrups crossing the joint plane.

Materials for Girder Tests

The concretes used in the girder tests were of the same materials as described for the push-off tests. Individual strengths are shown in Table 2.

The longitudinal and shear reinforcement conformed to ASTM A-305 for deformations. The longitudinal bars were of high strength steel while the stirrup steel was of intermediate grade. Individual yield points are shown in Table 2.

Casting of Girder Specimens

The precast girder parts were cast in plywood forms with the composite contact surface horizontal at the top. The concrete was placed by spud vibrators in both the girders and companion 6x12-in. cylinders. After consolidation was completed, the contact surface was prepared, smooth or rough as previously noted, and the girder part was cured wet for seven days. Seven days drying followed the curing, and the top deck was then cast. The precast girder was supported uniformly along its length during the casting of the deck. Another cycle of seven days wet curing and seven days drying preceded testing of the composite girders.

Test Method

Series I Girders. The six girders of this series were tested over a 145-in. simple span with two loads 25 in. apart centered in the span. The testing was carried out in a hydraulic testing machine as shown in Fig. 10. The load was applied in increments, deflection and slip measurements were after stable conditions were observed.

Deflection measurements were made with dial gages graduated to 0.001 inch.

TABLE 2 — GIRDER TESTS

Specimen Number	Joint Surface	No. 3 Stirrup Spacing inches	Stirrup Yield Point psi	No. 6 Tension Reinforcement Yield Point psi	Concrete Cylinder Strength	
					Slab psi	Girder psi
SERIES I						
BRS-I	Rough + Bonded	6	49,300	87,200	3120	4480
RS-I	Rough, Unbonded	6	49,300	87,200	2060	4150
BS-I	Smooth + Bonded	6	49,300	87,200	3000	4670
BR-I	Rough + Bonded	none[1]	none	87,200	3170	4200
MS-I	Monolithic	6	49,300	87,200	2860	5050
M-I	Monolithic	none[1]	none	87,200	3320	5790
SERIES II[2]						
BRS-II	Rough + Bonded	16	53,300	88,400	2500	4930
RS-II	Rough, Unbonded	16	53,300	88,100	3130	4680
BS-II	Smooth + Bonded	16	53,300	88,600	3520	4810
MS-II	Monolithic	16	53,300	93,600	4060	5790

[1] No. 3 at 6 in. for the girder stem only (not crossing contact surface)
[2] Series II girders had additional stirrup reinforcement in the girder stem, two No. 3 stirrups between each pair of connection stirrups.

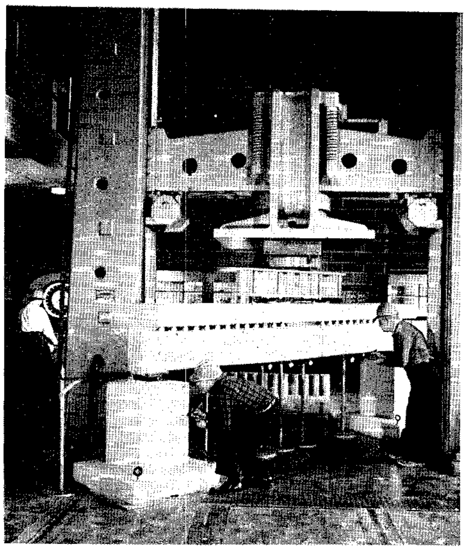

Fig. 10 — Girder Test, Series I.

The gages in most cases were placed at mid-span of the girder, under each load point, and at the center of the shear spans.

Slip between the deck slab and the girder was observed by dial gages graduated 0.001 in. and mounted in the corner at the contact surface as shown in Fig. 11. The gage was connected to an insert in the bottom surface of the deck slab, with its stem bearing against a steel bracket from the side surface of the girder. These supports and extensions were mounted so that their connection points were in a vertical plane, and bending strains were eliminated by supporting the gage at the same level as the bracket from the girder. Shear deformations of the concrete in this local region could not be eliminated, and it is probable that these contribute slight errors before slips resulting from cracking develop. In Series I the slip gages were mounted, in most cases, at a spacing of five inches from load point to the girder end on both sides of mid-span.

Series II Girders. The four girders of this series were tested over a 20-ft. simple span with three concentrated loads spaced 3 ft. and centered in the span. The loading was applied by hydraulic rams, utilizing a test floor as shown in Fig. 12. All testing procedures were identical to those used for the girders of Series I, except that slip gages were spaced 16 in. apart over the full girder length.

GIRDER TEST RESULTS

Failure of all girders of Series I, as illustrated by Fig. 13, may be described as a shear-compression failure preceded by loss of composite action over most of the length outside the load points. After initial flex-

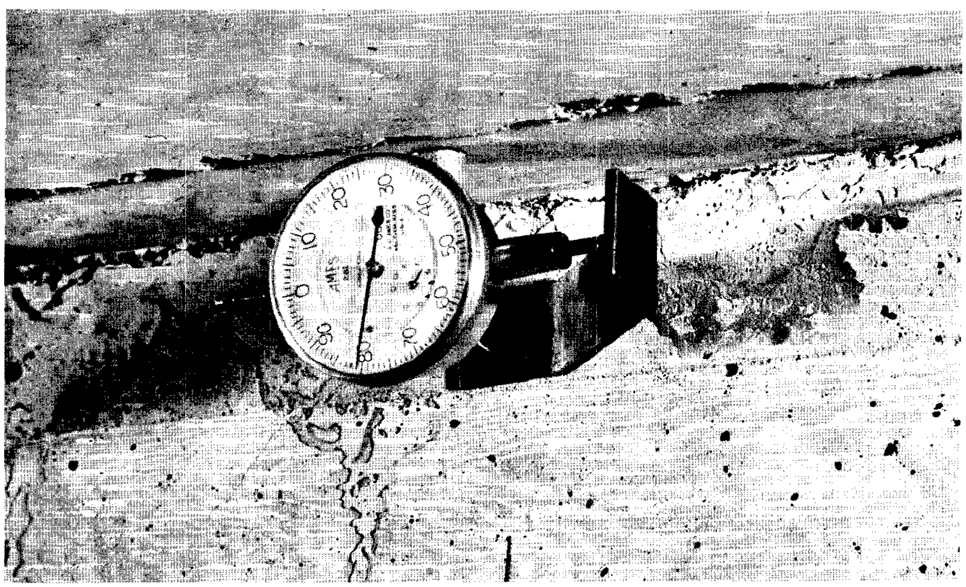

Fig. 11 — Slip Gage Mounting.

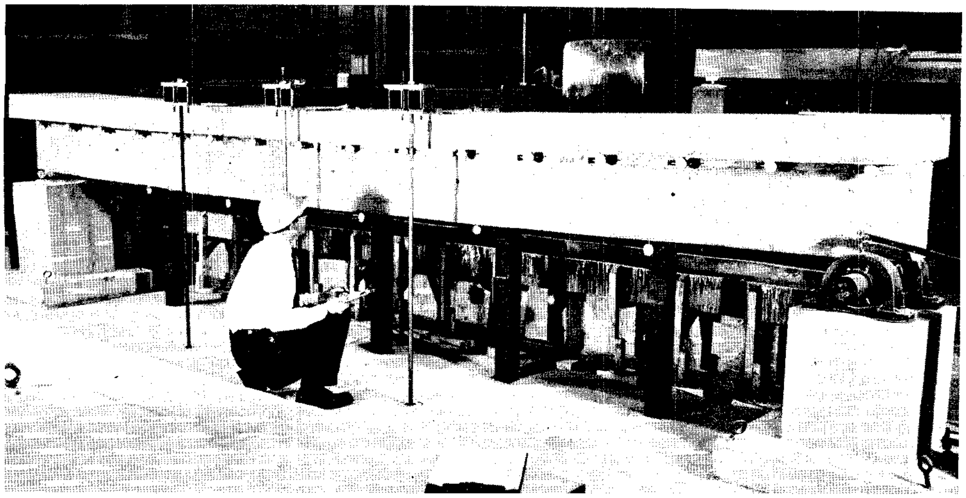

Fig. 12 — Girder Test, Series II.

ural cracking of the girder, cracks in the girder progressed upwards to the bottom of the deck slab at a small load increase. As the loading was continued, additional cracks formed in the girder, inclining toward the load point. As these cracks reached the contact surface between the deck and the girder, they tended to travel along the joint for short distances. As the shear caused slip to develop between girder and slab, the girder began to act as a partially composite member. The first evidence of such action was the closing of the upper parts of flexural cracks that had reached the contact surface, thus indicating compression in the top of the precast girder. The next evidence of non-composite action was flexural cracking of the bottom of the deck slab. Increased loading then caused long diagonal tension cracks to ex-

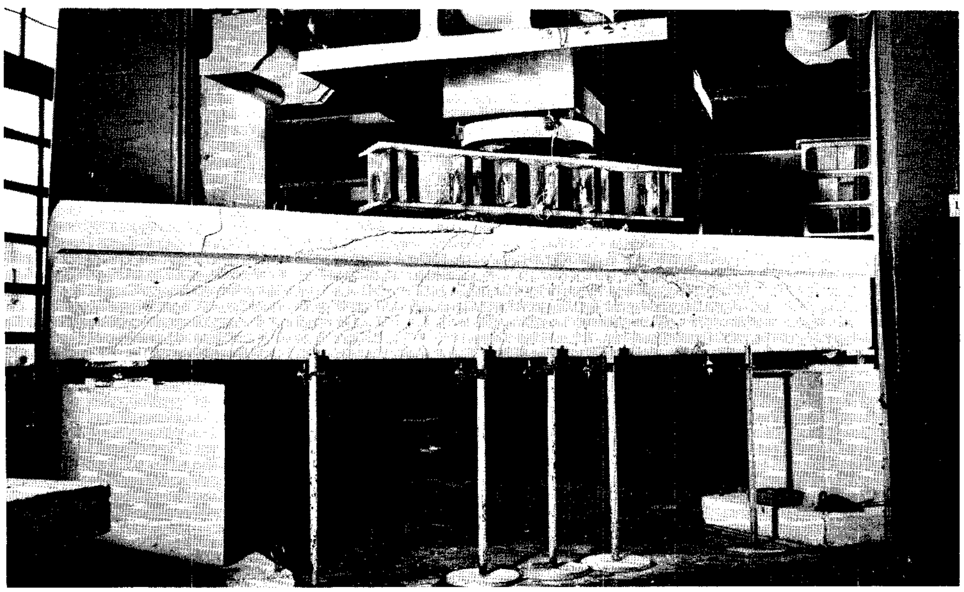

Fig. 13 — Series I, Girder After Test.

tend along the contact surface and into the deck slab, thus precipitating a shear-compression failure.

Slip of the deck developed toward the girder ends, but did not progress along the entire contact surface to the beam end. A failure plane occurred diagonally from the contact surface to the bottom of the girder. This left the shear connection intact at the girder ends; the rotation of the diagonal end block caused a crack in the deck slab above the diagonal failure plane, as shown in Fig. 14.

The girders of Series II also failed as non-composite members, but by flexural compression crushing of the top of the precast girders below the contact surface. The behavior under load was similar to that of the Series I girders except that diagonal cracks did not cause failure.

Deflection

A summary of the deflection test data for Series I and II, plotted versus calculated horizontal shearing stress, is given in Fig. 15.

It will be noted that the deflection of girders with a bonded contact surface follows the curve for the monolithically cast specimen until changes in the conditions at the contact surfaces cause deviations. The specimens with a smooth bonded surface start to show deviations of contact surface properties at approximately 340 psi horizontal shearing stress for Series I, and 310 psi for Series II. This deviation is more marked in Series II in which a smaller amount of stirrup reinforcement is involved.

Girders with bond and roughness indicate a deviation from a nearly linear deflection curve at gross shearing stresses of 620 psi for Series I, and 520 psi for Series II. In Series I the monolithic girder deviated from the linear deflection path at a lower shear stress than did the companion beam with a rough and bonded contact surface.

The girders with roughness and without bond deflect more, from early load to failure, than the monolithic or rough, bonded beams. This indicates a partial composite action rather than full composite action.

The two girders in Series I without stirrups crossing the contact surface followed similar deflection curves. There was no marked difference between the monolithic girder and that with a rough, bonded connection.

Slip Curves

A typical example of the gradual development of slip between deck slab and precast girder is shown in Fig. 16. The slip at the contact surface was usually a

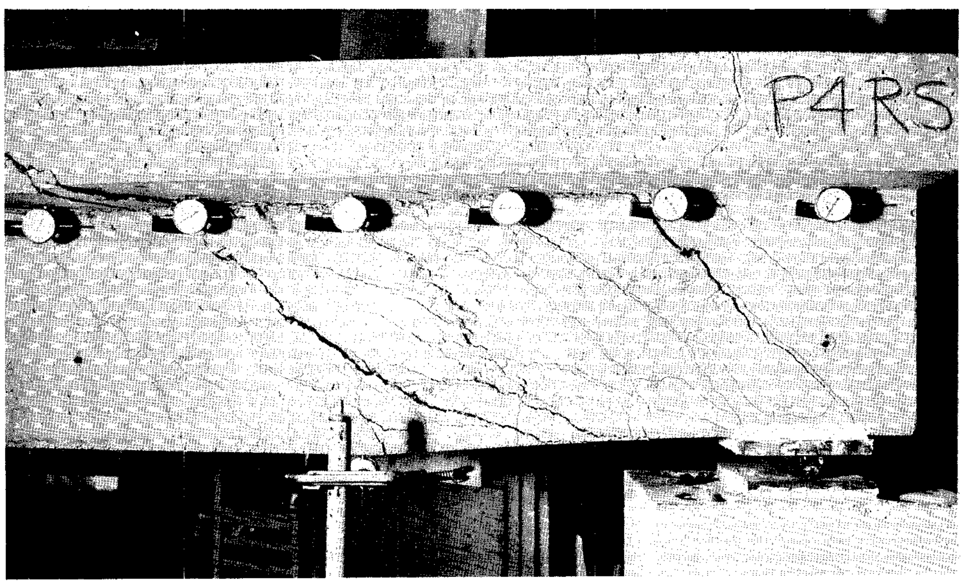

Fig. 14 — Diagonal Cracking in Series I.

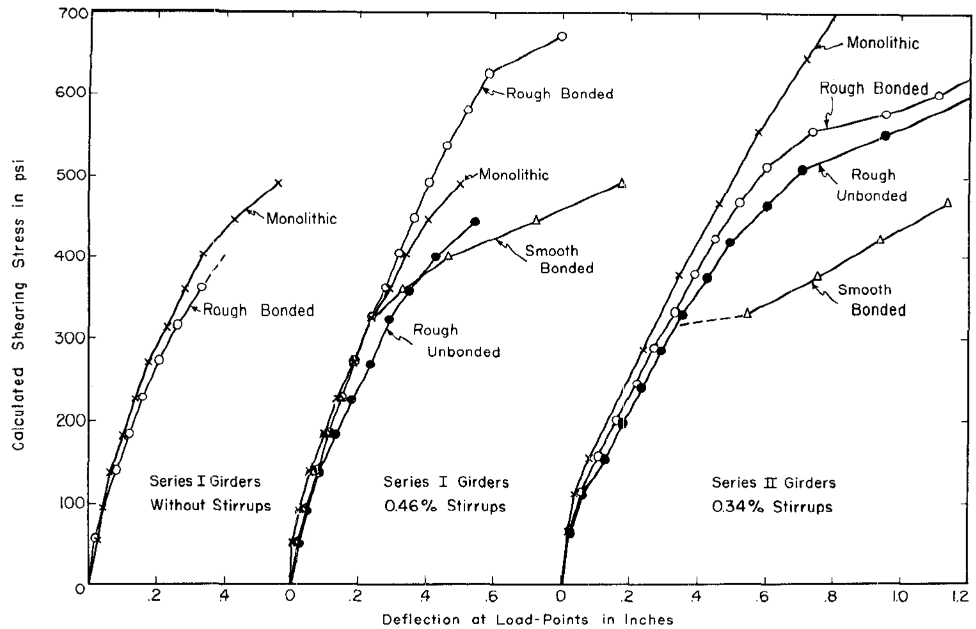

Fig. 15 — Deflection of Test Girders.

maximum at most load levels at approximately one-quarter of the span from the girder ends. One exception to this trend was the girder of Series II with a smooth bonded connection which developed slip to the very end of the girder as soon as bond failed. The gradual development of slip shown in Fig. 16 is significant in that even though there are large local slips, the girder cannot become completely noncomposite until slip takes place at the girder end. In past work, therefore, end slip has often been used as a criterion to judge the degree of composite action. However, in reinforced concrete girders such as those of Series I, the "end slip" did not occur at all, and the slip movements were taken up as additional width of the diagonal cracks. When the slip had progressed far enough toward the girder end, the "end block" condition developed as shown in Fig. 14. For some girders, however, slip did not progress far enough toward the end to produce the "end block" condition. The slip movements involved then added width to the diagonal tension cracks and precipitated a "shear-compression" failure of the girder.

Series II girders with connections all developed end slips of various amounts. Diagonal tension stress in the precast girder was lower than for Series I, and the reinforcement at the top of the precast girder probably strengthened the girder so that the connection became the path of least resistance.

The slip measurements for all girders are summarized as maximum slip versus calculated horizontal shearing stress in Fig. 17. The two slip curves presented for each girder were each calculated as the average of the high values of slip near the quarter-points of the girder span. These values represent slip averaged over a length of about 30 inches.

Some slip curves in Fig. 17 indicate a change in slope at a shear far below the value at which the connection starts to affect the beam action. For example, the curves for the rough bonded girder of Series I have two changes in slope, a slight one at 270 psi shear and another at 540 psi. Previously it was noted that the deflection curve for this girder deviated from a smooth curve at 620 psi. Thus, the final sharp increase in deflection was due to an accumulation of slip which began at about 500 psi. During testing of this girder, it was noted that flexural cracks extended up to the contact surface at a horizontal shear of 230 psi. It is reasonable to assume,

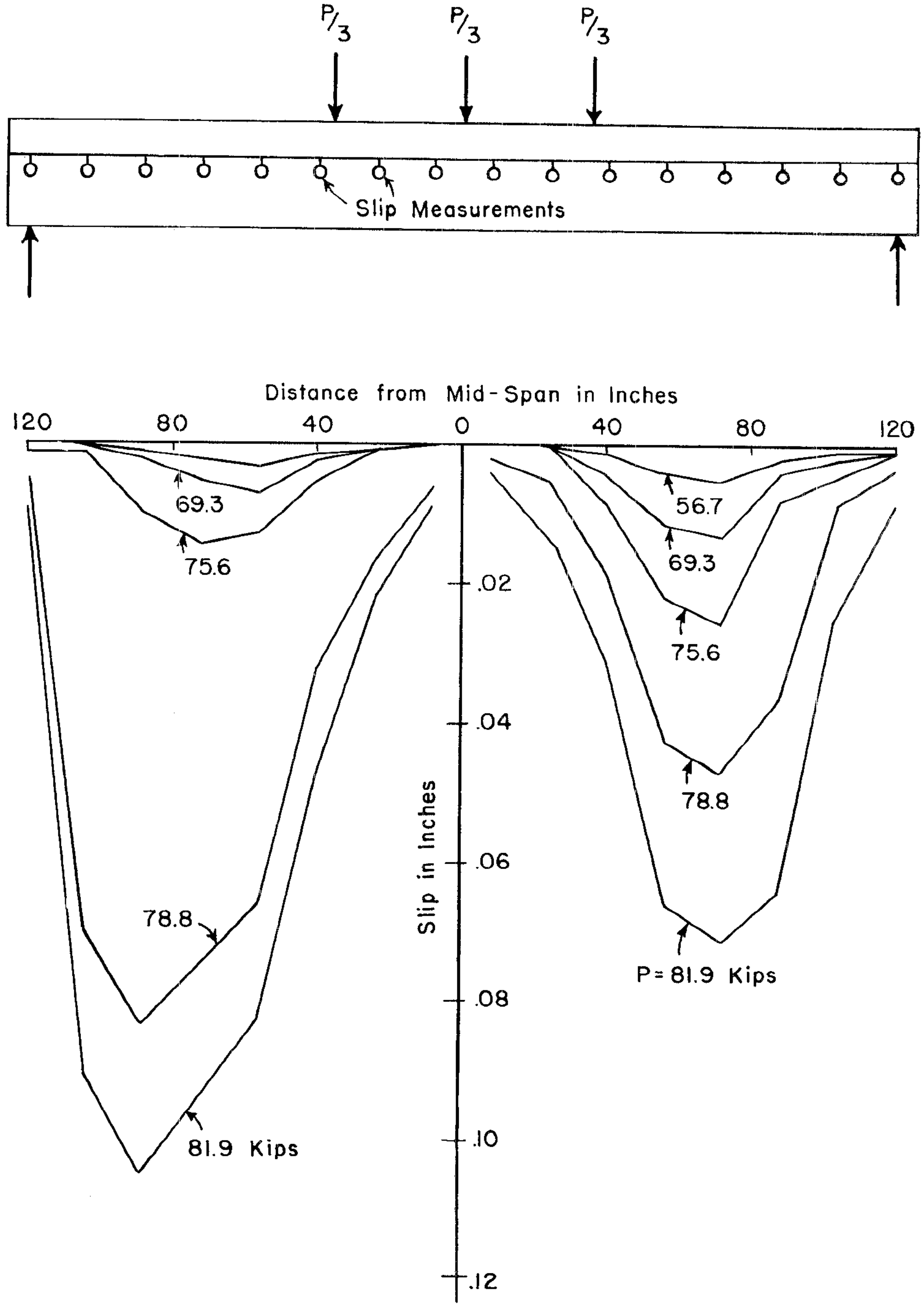

Fig. 16 — Typical Slip Development, Girder BRS-11.

therefore, that these cracks affected the slip measurements, and may have caused the first break in the slip curve, although they did not cause actual slip of the joint.

The rough, unbonded girder of Series I has a shear-slip curve, as seen in Fig. 17, developing slip rapidly at low shear. In this case, vertical cracks did not reach the contact surface before slip developed, and the resulting partial loss of composite action stopped their development. Slip continued to increase with increase in load, following a smooth curve, until a shear-compression failure developed. The rough unbonded girder of Series II, however, did not act like its companion in Series I. Although the bond breaking compound was applied, the slip curve indicated an action similar to that of a girder with a rough bonded contact surface. Thus, although an attempt was made to destroy bond in a manner which was always successful in the push-off tests, the performance of this girder indicated complete bond integrity on one half of the span and only a partial loss on the other half.

The girders with a smooth bonded contact surface in Series I and II, both developed a sudden deviation from a perfect connection to one with large slip values. This is as expected from the push-off tests. The shears at first deflection deviation of these girders are 340 and 310 psi, as compared to slip deviations of 330 and 250 psi, respectively.

The two girders of Series I without stirrup reinforcement in Fig. 17 indicate a change in slope of the slip curve at approximately the same shear at which vertical cracks first reached the contact surface, and continued loading increased these slips until a shear-compression failure took place. Both beams follow similar slip curves, indicating that lack of stirrups rather than differences in connection properties, was the primary factor causing failure.

A comparison of the shear-deflection curves in Fig. 15 and the shear-slip curves in Fig. 17 indicates that serious lack of composite action as indicated by breaks in the deflection curves took place at slip of approximately 0.005 in. Thus, a slip of 0.005 in. seems to be a critical value beyond which composite action is rapidly destoyed.

Comparison of Push-off and Girder Tests

A comparison of the slip curves derived from girder tests and push-off tests is given in Fig. 18. The push-off test data reported in Table 1 for a shear length of six inches, for which the effects of the No. 4 stirrups of the push-off specimens were subtracted, were modified by adding the stirrup effect for the No. 3 stirrups of the

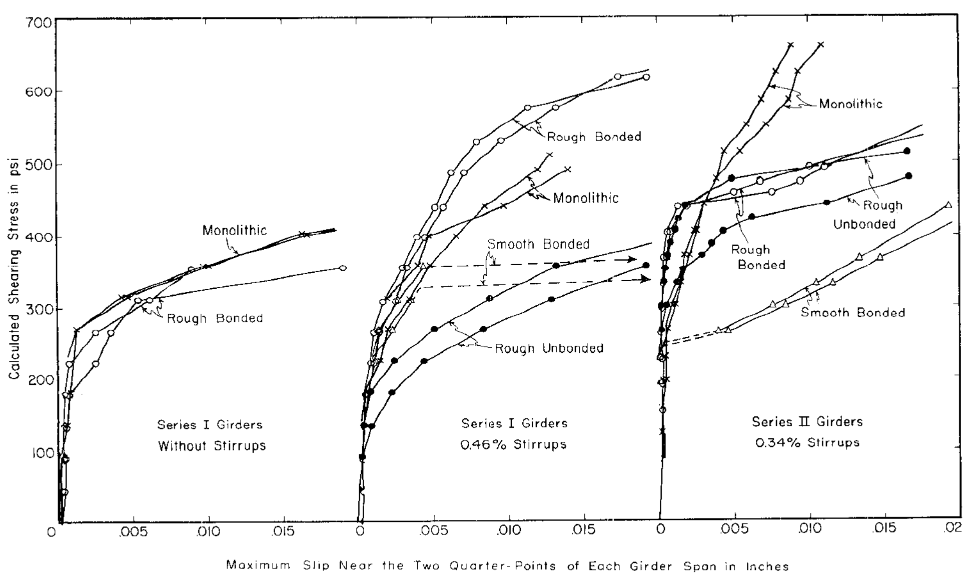

Fig. 17 — Shear-Slip Relationships for Test Girders.

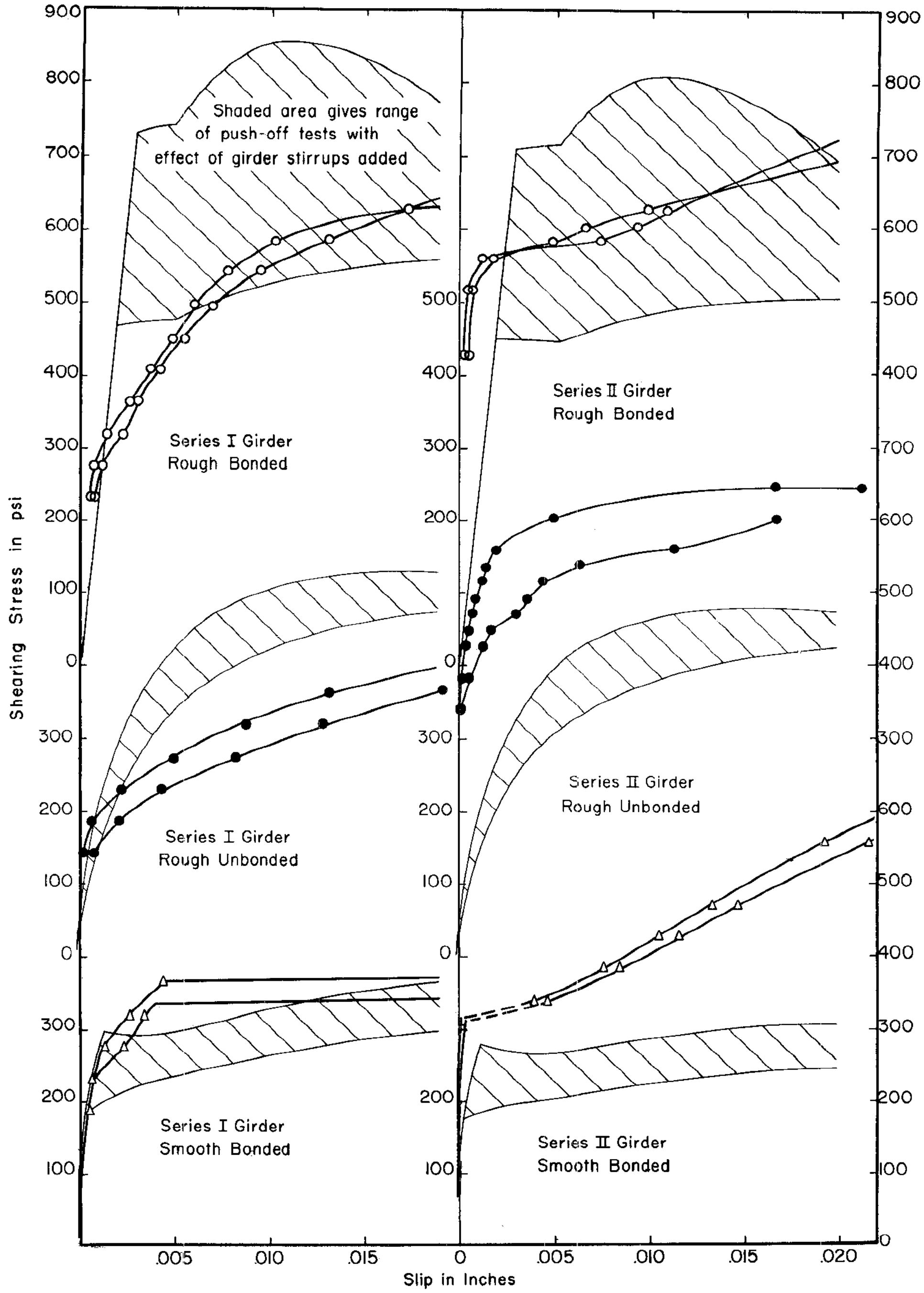

Fig. 18 — Comparison of Shear-Slip Relationships for Push-Off and Girder Tests.

girders according to Fig. 4. This was done separately for Series I and II in proportion to girder stirrup spacing. Only the maximum and minimum push-off curves for the various surface textures are shown in Fig. 18 as enclosing a range of stress at various slips. This range should define the connection capability as observed in push-off tests.

The shear-slip curves for the girders are reproduced unmodified in Fig. 18 along with the adjusted push-off curves corresponding to each girder. The push-off tests for rough and bonded contact surfaces cover the range of the girder slip curves for both series. A wide variation exists between girder test and push-off test results for specimens with rough unbonded surfaces. The Series I girder curve lies 30 per cent below, and Series II girder curve lies 50 per cent above, their corresponding push-off curves. It should be recalled here that the Series II girder with unbonded roughness is suspected to have developed bond stress even though a bond-breaking treatment was applied. For the two beams with a smooth bonded joint, the push-off curves fall below the girder shear-slip curves.

In summary, it may be stated that the push-off tests give a good representation of the character of the stress-slip curves for the girders tested. Quantitatively, the push-off test curves are conservative for a smooth bonded connection, representative for a rough bonded connection, and inconclusive for a rough unbonded connection.

CONCLUDING REMARKS

1. The push-off tests, which have been shown to demonstrate characteristics of slip versus shearing stress similar to those obtained from girder tests, are a valuable aid in establishing the strength of horizontal shear connections for composite action.

2. At slips of about 0.005 in., at the contact surface, the girder deflection curves begin to deviate from a smooth curve. This slip value of 0.005 in. is a higher value of slip than was measured in any push-off test before the bond failed. In other words, as may be expected, as long as a girder has integrity of the connection by bond, the girder is fully composite.

3. The girder and push-off tests reported herein indicate a maximum shearing stress for composite action of 500 psi for a rough bonded surface and 300 psi for a smooth bonded surface. The compressive strengths of the concretes were 3000 and 5000 psi for the slab and girder, respectively. In addition to these values, approximately 175 psi shear capacity may be added for each per cent stirrup reinforcement crossing the joint. This stirrup effect was derived from push-off shear-slip curves for stirrups at a slip of 0.005 in. The girder tests reported had stirrup percentages of 0.46 and 0.34 for Series I and II, respectively. The above-mentioned stirrup effects are related to push-off tests of No. 4 stirrups. The contribution of other stirrup sizes will probably vary with stirrup diameter, concrete strength, and possibly stirrup percentage.

4. Push-off tests indicate that keys used with a rough bonded contact surface do not change the strength of the connection. The slip movements required to develop the keys are greater than the movements for a bonded surface. Therefore, the effects of the two are not additive.

5. All connections without bond studied by push-off tests exhibited a much more flexible character than those with bond. The critical slip of 0.005 in. occurred after a smooth increase of slip with stress. The deflection curve for a beam with this type of connection began to deviate noticeably from a smooth curve at slips of about 0.005 in.

6. The girder tests indicated that, when bond is absent at the connection, roughness can contribute 150 psi shearing strength at 0.005 in. slip, plus a value for the stirrups in the girder. If the push-off test data on keys are extended to girders, it appears that keys covering 50 per cent of the contact area could develop an average shearing strength of 325 psi at 0.005 in. slip when bond is not present.

7. In the development of precast prestressed bridges, of which this study is a part, it seems advisable to continue work only with horizontal shear connections effected by a combination of a rough, bonded contact surface and stirrups extending from the precast girders into the situ-cast deck slab.

8. Toward a general design solution for horizontal shear connections, further work is needed primarily regarding effects of concrete strength, stirrup size, stirrup percentage, and repeated loading. Girders with the slab portion in compression and girders with the slab in tension should be considered.

REFERENCES

1. Dean, W. E., and Ozell, A. M., "No Shear Keys Are Needed Here," *Engineering News-Record*, **156**, 61-62 (June 7, 1956).
2. Dean, W. E., "Beam Test Shows Need for Web Steel," *Engineering News-Record*, **157**, 36-37 (December 20, 1956).
3. Ozell, A. M., and Cochran, J. W., "Behavior of Composite Lintel Beams in Bending," *Journal of the Prestressed Concrete Institute*, **1**, No. 1, 38-48 (May, 1956).
4. Samuely, Felix, "Some Recent Experience in Composite Precast and In-Situ Concrete Construction, with Particular Reference to Prestressing," *Proceedings of the Institution of Civil Engineers* (London), **1**, Part III, 222-279 (August, 1952).
5. Evans, R. H., and Parker, A. S., "Behavior of Prestressed Concrete Composite Beams," *Journal of the American Concrete Institute* (May, 1955); *Proceedings*, **51**, 861-878 (1955).
6. Felt, E. J., "Resurfacing and Patching Concrete Pavements with Bonded Concrete," *Highway Research Board Proceedings*, **35**, 444-469 (1956); *PCA Development Bulletin D8*.
7. Viest, I. M., "Investigation of Stud Shear Connectors for Composite Concrete and Steel I Beams," *Journal of the American Concrete Institute* (April, 1956); *Proceedings*, **52**, 875-891 (1956).
8. Viest, I. M., Siess, C. P., Appleton, J. H., and Newmark, N. M., "Studies of Slab and Beam Highway Bridges. Part IV: Full-Scale Tests of Channel Shear Connectors and Composite T-Beams," *University of Illinois Engineering Experiment Station*, Bulletin No. 405, December, 1952, 155 pp.

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