Effect of Strand Blanketing on Performance of Pretensioned Girders

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SYNOPSIS

Reports a study of blanketing by plastic tubing used to prevent bond between strand and concrete near the ends of pretensioned prestressed girders. Tests of five 34-ft. T-beams indicated that the bonded embedment length required by Section 2611 of the 1963 ACI Building Code is inadequate for blanketed strand. Twice this anchorage length is needed to develop the ultimate flexural and shear strength of girders when blanketing is used.

INTRODUCTION

Two methods are in use for limitation of compressive and tensile concrete stresses near the ends of pretensioned prestressed concrete members. Some of the pretensioned reinforcement may be "harped", that is, deflected upward, near the ends of the members, a somewhat costly fabrication process; or, bond to the concrete may be prevented for some of the pretensioned reinforcement in the end regions. Bond may be prevented by various means; one method is the use of plastic tubing which is often referred to as "blanketing".

The purpose of the investigation reported herein was to explore the possible effects of blanketing on the flexural behavior at service load and on the ultimate flexural, bond, and shear strength of pretensioned prestressed girders.

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BACKGROUND AND SCOPE

Function of Bond

The initial function of bond is to transfer the prestress force from the pretensioned reinforcement to the concrete member. Laboratory studies of the bonded length of strand needed for such transfer have been reported previously[1]; and numerous other papers regarding bond in prestressed concrete are available. Under service loads, the stress in the prestressed reinforcement normally remains near the effective prestress level; the structural integrity of the member continues to depend largely on bond in the stress transfer regions; and flexural bond stresses are negligible.

For loads in the range from service load to ultimate flexural strength, however, stress in the prestressed reinforcement must eventually increase substantially beyond the prestress level. Previous laboratory work[2] has shown that a considerable bonded embedment length beyond the stress transfer length is required to develop these increased steel stresses.

If inadequate embedment length is provided, ultimate strength is governed by bond rather than by flexure[2]. Bond slippage of the strands occurs in three stages: (1) progressive bond slip begins at flexure cracks, (2) general bond slip is initiated along the entire embedment length, and (3) the mechanical interlocking between the helical strand surface and the concrete is destroyed.

It should be noted that the mechanical interlocking is adequate to maintain considerable strand stress even after extensive bond slip. In many cases the strand stress after general bond slip drops only toward the prestress level and not to zero as one might fear. Thus, the final effect of inadequate embedment length may be a premature flexural failure at a reduced strand stress, corresponding to a final bending moment less than the computed ultimate strength in flexure.

Bond Requirements

The 1963 ACI Building Code (ACI 318-63)[3], Section 2611, calls for a bonded embedment of prestressing strands from the cross section under consideration for a distance in inches not less than:

$$\left(f_{su} - \frac{2}{3} f_{se}\right) D \qquad (1)$$

in which

D = nominal strand diameter in inches

f_{su} = calculated stress in prestressing steel at ultimate load in ksi

f_{se} = effective steel prestress after losses in ksi.

The embedment length required by Eq. (1) was based on tests of beams with all strands bonded from the section of maximum moment to the beam ends. The end of the embedment length can then overlap the stress transfer length near the beam supports, where a state of flexural precompression exists even at high loads, and where a lateral compression is provided by the vertical beam support reaction. When strands are blanketed for a considerable distance into a member, however, both stress transfer and flexural bond embedment may take place in a concrete region subjected to tension, and even cracking, before the ultimate load is reached. Under these severe conditions, the embedment length given by Eq. (1) may be inadequate.

Scope of Tests

The investigation was carried out to evaluate separately the effects of blanketing on flexural behavior and on shear capacity.

Three girders were designed and tested for the study of flexural behavior. Girder 1 had no strands blanketed and was used for comparison. Girder 2, designated as "partially blanketed", had strands so blanketed that the embedment lengths were twice those computed by Eq. (1). The "fully blanketed" girder, Girder 3, was designed with embedment of the blanketed strands equal to the lengths required by Eq. (1). All three girders were over-reinforced with stirrups to prevent interference of shear distress with flexural and bond behavior. The girders were subjected to 5 million cycles of service load and subsequently tested statically to failure.

The shear investigation involved static testing to destruction of a non-blanketed girder, Girder 4, and a girder with partially blanketed strands, Girder 5. The girders were similar to those in the flexure study except that the number of stirrups was reduced in order that any effects on shear capacity of blanketing strands would be demonstrated. The stirrups in Girders 4 and 5 had a spacing 1½ times that computed by Section 2610 of the 1963 ACI Building Code, assuming a concrete strength of 5000 psi. It should be noted that Girder 5 had a slightly heavier web reinforcement than Girder 4, due to the smaller prestress in the blanketed regions.

TEST GIRDERS

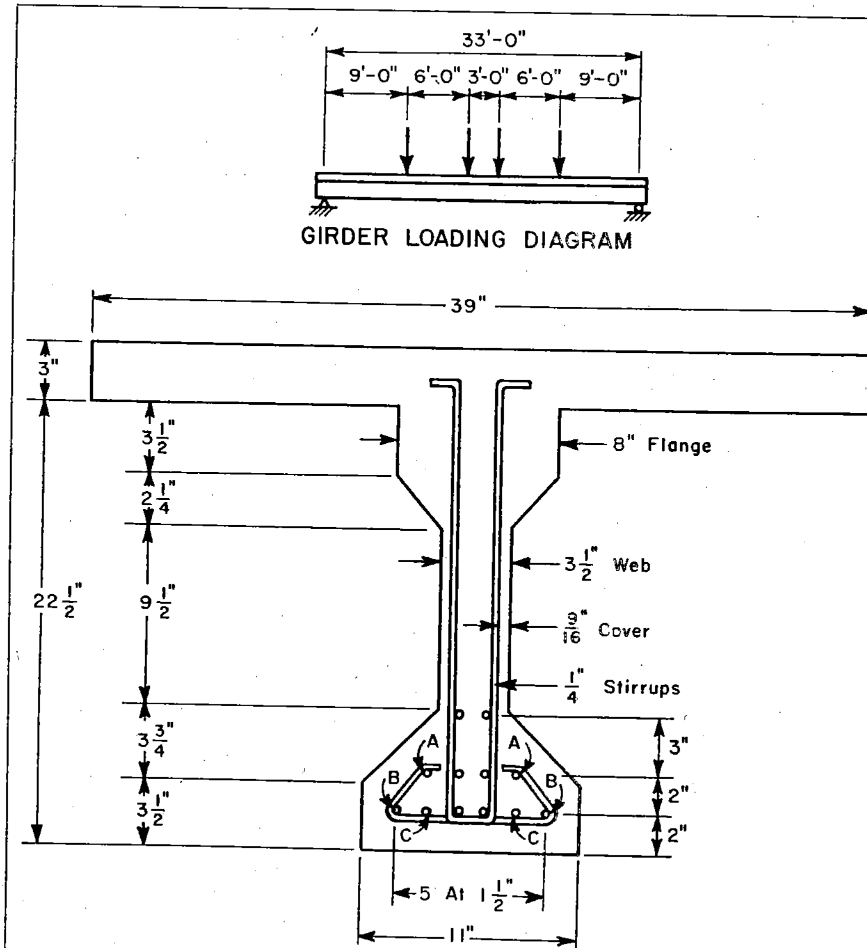
All girders tested had the cross section shown in Fig. 1. The precast I-section is a Type III AASHO-PCI standard prestressed concrete bridge beam to half-scale. The girders were 34 ft. long, simply supported over a 33-ft. span. After the girder cross-section and span were chosen, the girder design consisted of calculating the prestress force and location to utilize the allowable stresses given in Table 1. Twelve seven-wire strands of ⅜-in. diameter were used in the arrangement shown in Fig. 1. In the design it was assumed that the strand would be tensioned initially to 175 ksi and that the tension after all losses would be 140 ksi.

The ultimate flexural strength of the girder was computed using Eqs. (26-4) and (26-6) in Section 2608 of the 1963 ACI Code assuming an ultimate strand strength, f'_s, of 250 ksi. The working load moment was calculated from the requirement of Section 1.13.6 of the 1961 AASHO Standard Specifications for Highway Bridges[4] that every section of the girder have an ultimate strength of at least $(1.5D + 2.5L)$ where D is the dead load effect and L is the effect of design live load including impact. The magnitude and location of the live loads were governed by the working load moment, with the further requirement that the moment diagram should be similar to that of a reasonable moment envelope for highway bridge girders. The live load consisted of four equal concentrated loads in the arrangement shown in Fig. 1.

The moment diagram at failure due to dead and live loads was computed assuming that the maximum moment was equal to the ultimate moment of the non-blanketed girder cross-section. From this diagram, locations were determined at which it was possible to reduce the amount of tensile reinforcement by blanketing of strands. The position of the blanketed strands in the girder cross-section and the lengths of the blankets from the girder ends are given in Fig. 1.

Table 1—Design Stress Limitations
AASHO Standard Specifications for Highway Bridges 1961

STAGE	STRESS	LIMIT
At transfer of prestress, $f'_{ci} = 4000$ psi	Compression Tension	$0.6 f'_{ci} = 2{,}400$ psi $3\sqrt{f'_{ci}} = 190$ psi
At full design load, $f'_c = 5000$ psi	Compression Tension at bottom face of girders	$0.40 f'_c = 2{,}000$ psi zero



BLANKET LENGTHS

	Strand A	Strand B	Strand C
Fully Blanketed Girder	3'-11"	6'-11"	10'-11"
Partially Blanketed Girder	——	2'-4"	6'-4"

Fig. 1—Cross-Section and Loading Arrangement of Test Girders

Stresses in the concrete at initial prestress and at other periods in the load history are shown in Fig. 2. End stresses were computed at the inside limit of the 26-in. prestress transfer zone. This transfer length was determined in previous work[1].

FABRICATION AND INSTRUMENTATION

The girders were fabricated and tested by methods commonly used in the PCA Structural Laboratory. A general description of facilities and

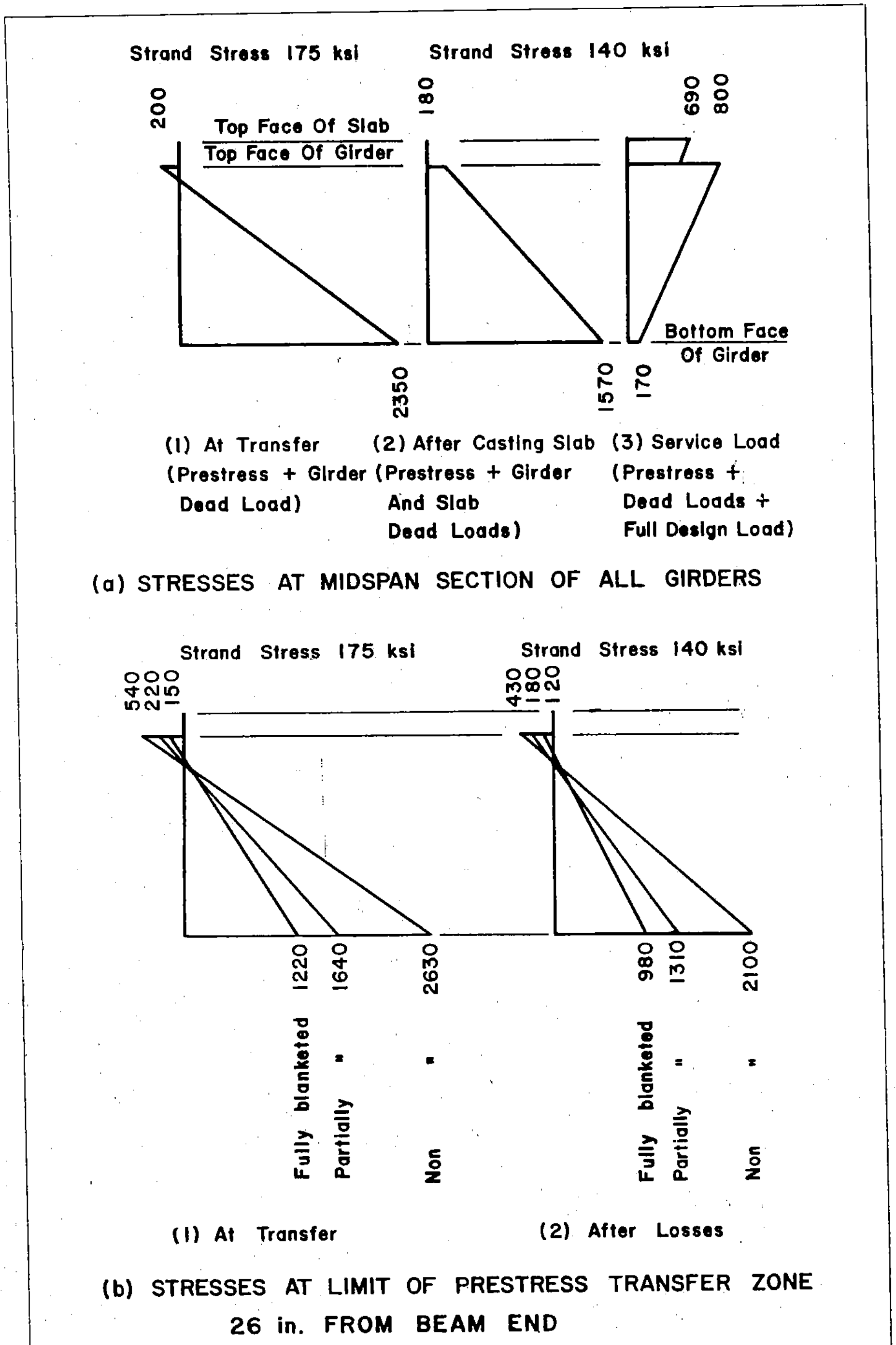


Fig. 2—Design Stresses

test methods is given elsewhere[5].

Fabrication

The precast beams were produced in a prestressing bed set up on the laboratory test floor. The strands were tensioned individually using a 20-in. stroke center-hole ram. The tension in the strand was measured by a load cell placed between the strand grip and the anchor plate at the dead end of the bed. The 175-ksi strand tension was attained by initially overstressing the strand so losses would not decrease the prestress below the target value. Strain measurements of the strand after stress release, casting of the deck slab, and storage, showed 140 ksi after losses to be a realistic figure.

The stirrups were positioned and the concrete cast on the day following strand tensioning. Moist curing of the beams was discontinued after three days and the strands were cut by an acetylene torch.

The formwork for the deck slab was cantilevered from both sides of the precast beam in such a manner that the slab dead weight together with the forms was carried by the beam acting as a simple span. In this way usual field practice was simulated. The deck slab was moist cured for three days after casting, and the specimens were then exposed to laboratory atmosphere of 70° F. and 50 percent relative humidity.

Materials

The prestressing steel used was seven-wire stress relieved strand of ⅜-in. diameter and 0.080-sq. in. area. All strand was free of rust and was cleaned of surface oil before tensioning. The properties of the strand are listed in Table 2.

The stirrups used in Girders 1, 2,

Table 2
Properties of Prestressing Strand

GIRDER NO.	YIELD STRESS* KSI	TENSILE STRENGTH KSI
1	266	277
2	266	277
3	266	277
4	270	279
5	262	272

*Stress at one percent offset

and 3 were plain No. 2 bars. The stirrups in Girders 4 and 5 were made of No. 2 deformed bars and were selected from a group in which the yield stress ranged from 50.5 ksi to 54.5 ksi.

The concrete used in the fabrication of the precast beams contained a blend of Type III portland cement and ¾-in. maximum size aggregate. A blend of Type I portland cement was used in the deck slabs. In both cases the concrete was air entrained. The concrete cylinder strengths given in Table 3 are based on tests of 6 x 12-in. cylinders taken from alternate batches. The cylinders were cured and stored with the individual beam and deck slab specimens. All concrete strengths for Girders 4 and 5 are at the time of the test to failure.

The blankets covering the strand were polyethylene flexible tubing of 7⁄16-in. internal diameter with a 1⁄16-in. wall. The ends of the tubes covering the strand were taped shut to prevent entry of cement grout. Fig. 3 shows the strand blankets and taping of the ends.

Instrumentation

Strains were measured by monitoring SR-4 electrical resistance strain gages by means of continuous strip chart recorders. The strand gages were aligned along one helix

Table 3—Properties of Concrete

Girder No.	Compressive Strength, psi			
	At Start of Cyclic Loading		At Static Test to Failure	
	Deck	Beam	Deck	Beam
1	4050	8380	5320	9260
2	3810	6570	5680	7460
3	4540	7820	5600	8230

Girder No.	Split Cylinder psi		Compressive Strength psi	
	Deck	Beam	Deck	Beam
4	507	571	4940	7000
5	565	657	4910	7930

of the strand. In all girders, various strands in the bottom layer were gaged 1.5 ft. and 5.5 ft. either side of mid-span. In addition, the middle layer of reinforcement in Girders 1, 2, and 3 had gages 8.5 ft. from mid-span. Gages on the concrete at mid-span were placed on the top face of the deck, on the girder side an inch below the deck, and on the girder tension face. On one side of midspan in Girders 4 and 5, the stirrups between the support and the outer load point were gaged at the neutral axis of the composite section.

Deflections were measured with a precision level sighting to scales attached to the deck over the supports and at midspan. Girders 4 and 5 also had scales at the ¼-points of the span, and for Girders 2 and 3 scales were placed at even more frequent intervals along the span.

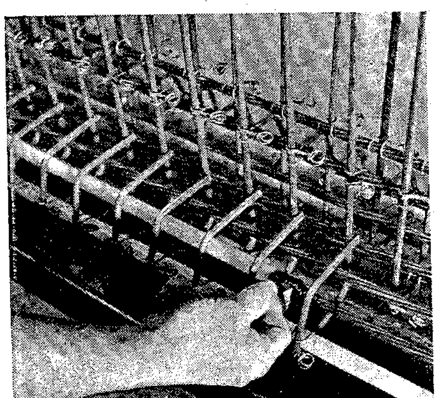


Fig. 3—Close-up of Strand Showing Strand Blanket

End slip of the strands relative to the concrete was detected by dial gages mounted on the projecting ends of the strands.

In the static tests to failure, the force at each load point was measured by load cells positioned in the loading assembly.

TEST PROCEDURE

Girders 1, 2, and 3 were subjected to 5 million cycles of the design live load prior to static testing to failure. The dynamic load was applied by four rams of an Amsler hydraulic pulsator unit at the load locations shown in Fig. 1. A girder under cyclic loading is shown in Fig. 4. Each girder was first loaded statically through five live load cycles. In the first and fifth cycles, the loads were applied in 500-lb. increments at each load point, and strains and deflections were recorded at each load level. Thereafter, the girder was loaded dynamically, with static tests carried out after approximately

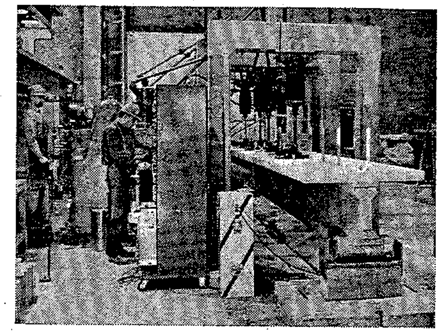


Fig. 4—A Test Girder Ready for Cyclic Loading

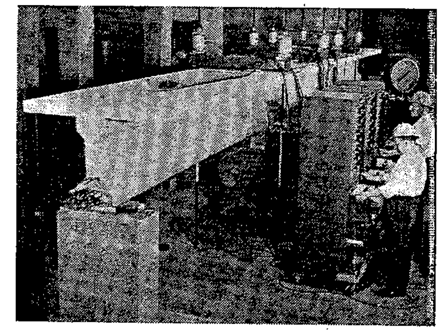


Fig. 5—A Test Girder Ready for Static Loading

1 million, 2½ million and 5 million cycles. At the completion of the 5 million cycles, the girder was moved and tested statically to destruction.

All five girders were loaded to failure with the test arrangement shown in Fig. 5. The loading apparatus was made up of steel cross-heads and rods, and the loads were applied by hydraulic rams reacting against the underside of the test floor[5]. The four equal concentrated loads were applied in 1000-lb. increments with readings of strains, deflections, and end slip taken at each load level.

TEST RESULTS—GIRDERS 1, 2, AND 3

Strength

The measured and calculated ultimate moments and other characteristics of the three girders are given in Table 4. The calculated ultimate moment was computed using the equivalent rectangular stress distribution of the 1963 ACI Building Code. Since the properties of the concrete and steel were carefully determined, and the loading precisely applied, a ϕ-value of unity was used in the expression:

$$M_u = \phi A_s f_{su} d (1 - 0.59q) \quad (2)$$

in which

M_u = ultimate resisting moment
ϕ = capacity reduction factor
A_s = area of prestressed tendons
f_{su} = calculated stress in prestressing steel at ultimate load
d = distance from extreme compression fiber to centroid of the prestressing force
q = $A_s f_{su}/bdf'_c$ = reinforcement index
b = width of compression face of flexural member
f'_c = compressive strength of concrete

The calculation of ultimate strength was based on a value for f_{su} equal to the measured ultimate strength

Table 4—Characteristics of Girders

Girder No.	Ultimate Moment Test ft-kips	Ultimate Moment Calc ft-kips	Ratio $\frac{M_{test}}{M_{calc}}$	Maximum Concrete Strain (midspan)	q	No. of Major Flexural Cracks at Ultimate
1 Non Blanketed	459	471	0.98	.0038	0.058	18
2 Partially Blanketed	454	472	0.96	.0032	0.054	20
3 Fully Blanketed	395	471	0.84	.0009	0.055	13

of the seven wire strand used.

In the case of Girders 1 and 2, all strand ruptured nearly simultaneously at ultimate load after extensive yielding. The strengths of these two girders were nearly the same, the ratio of M_{test} to M_{calc} being 0.98 and 0.96.

Girder 3, with fully blanketed strands, showed a much greater strength deficiency. The girder did not fracture near the point of maximum moment, but at an outside load point 7.5 ft. from the midspan. Two of the strand blankets extended from the girder end to 6 ft.-1 in. from midspan, so only 10 strands were acting at the fracture location. Figs. 6, 7 and 8 show the fracture areas of the three girders. The blanketed strand can be seen protruding from the concrete of the fully blanketed strand girder. While the ultimate test moment was 84 percent of the theoretical moment at the girder center, the test moment at the point of fracture was only 76 percent of the theoretical ultimate moment at that point. However, the girder did carry 84 percent of the theoretical load, and for this reason the moment-ratio in Table 4 is listed as 0.84.

Load-Deflection Relationships

Load-deflection observations were made in the static tests to detect, if possible, any bond slip in the strands in the working load range during the 5 million load cycles. The results of these observations showed no evidence of bond failure at these load levels. The load-deflection relation was linear and in none of the girders tested was there a significant difference between the relation at the first loading and that after 5 million load cycles.

The load-deflection curves shown

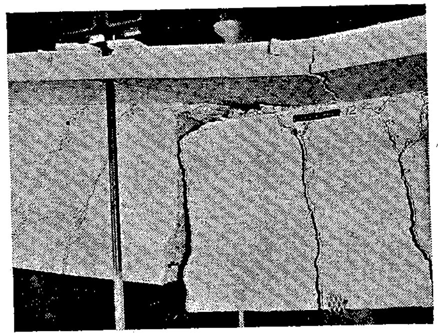


Fig. 6—Fracture Areas—Girder 1

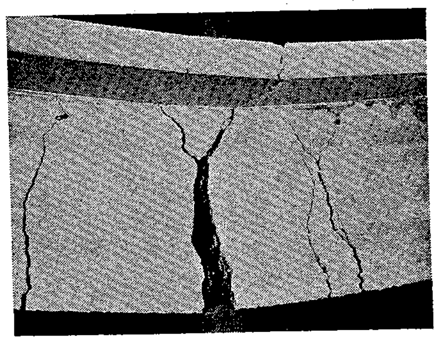


Fig. 7—Fracture Areas—Girder 2

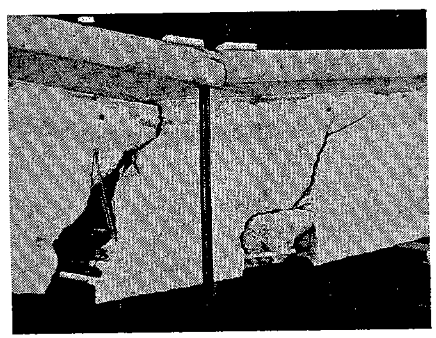


Fig. 8—Fracture Areas—Girder 3

in Fig. 9, plotted from data obtained in the static tests to destruction, illustrate the different performance of the three specimens. Load-deflection characteristics of the three girders were nearly identical from initial load to cracking. In the range beyond cracking, how-

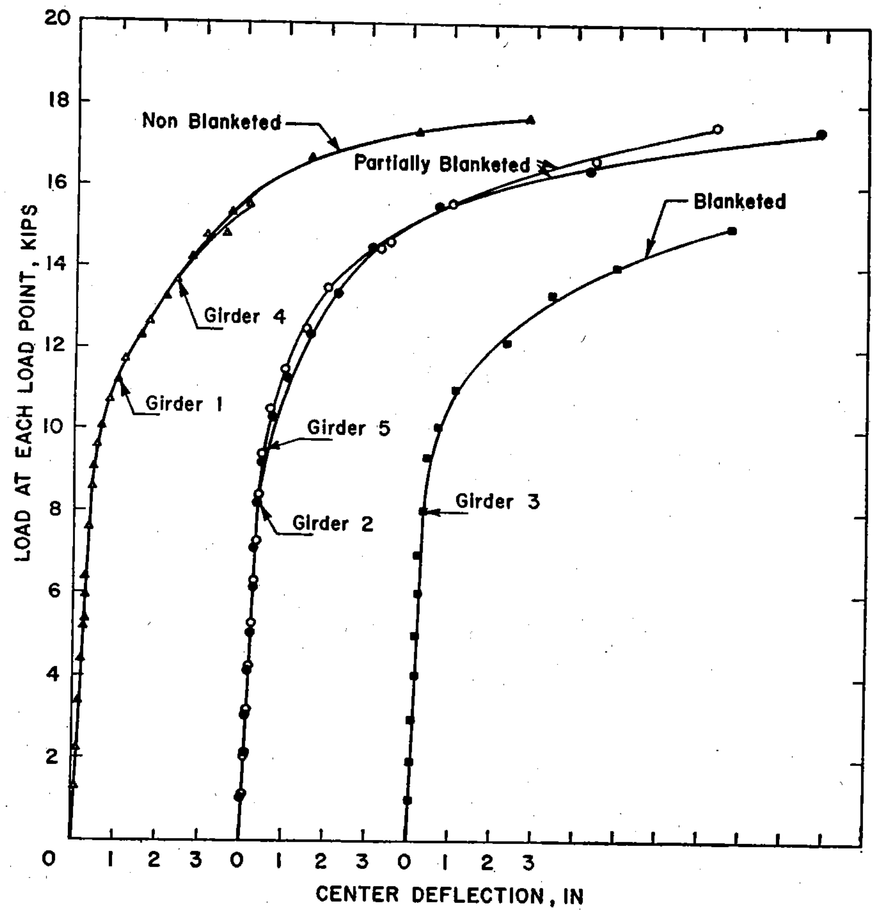


Fig. 9—Load-Deflection Curves

ever, the deflection characteristics of the girders depended on the length of strand blankets. Girder 1 deflected less at a given load after cracking than did the blanketed strand girders.

Flexural Cracking

The number of major flexural cracks at ultimate load in the girders also shows an effect of the blankets. As shown in Table 4, there were 18, 20, and 13 major cracks in Girders 1, 2, and 3. The flexural cracks of the fully blanketed girder, Girder 3, were considerably wider for given loads than were flexural cracks in the other two girders. This indicates a bond deficiency.

Strand Strain

Table 5 shows the range of increase in strand stress due to working load during the cyclic loading. There was no evidence from these stresses that the girders had cracked, that the strand had slipped, or that the performance was other than normal within the working load range. Stress variations were quite small.

Typical strand strain observations

Table 5
Increase in Stress in Outside Strand, Lower Level, at Working Load During Cyclic Loading

CYCLES	GIRDER 1 PSI	GIRDER 2 PSI	GIRDER 3 PSI
1	6540	5700	6540
5	5820	5700	6960
1 million	6270	5700	7260
2½ million	6120	5400	7260
5 million	6120	5400	—

during the static load to destruction are shown in Fig. 10. This observation also clearly distinguishes the performance of the three girders. The strain data from the outside strands on the lower and intermediate level are chosen as typical. In the case of Girder 1, the strand strain increased linearly until the concrete cracked in the vicinity of the strain gage. After cracking the strain increased until the strand ruptured near midspan at ultimate load. This behavior was typical of all strand in this specimen.

The unblanketed strand at the outside intermediate level of Girder 2 behaved in a manner similar to that of the strand in Girder 1 in the static load test. Strain increased linearly to cracking and at increasing rates thereafter until ultimate. The blanketed outside strand at the lower level of Girder 2 showed normal behavior to a point beyond the cracking load. At a well-defined point, the strain began to decrease and continued to do so until all strands fractured near midspan at the ultimate girder load. It may be noted that the strain 1 ft.-6 in. from midspan decreased only slightly and

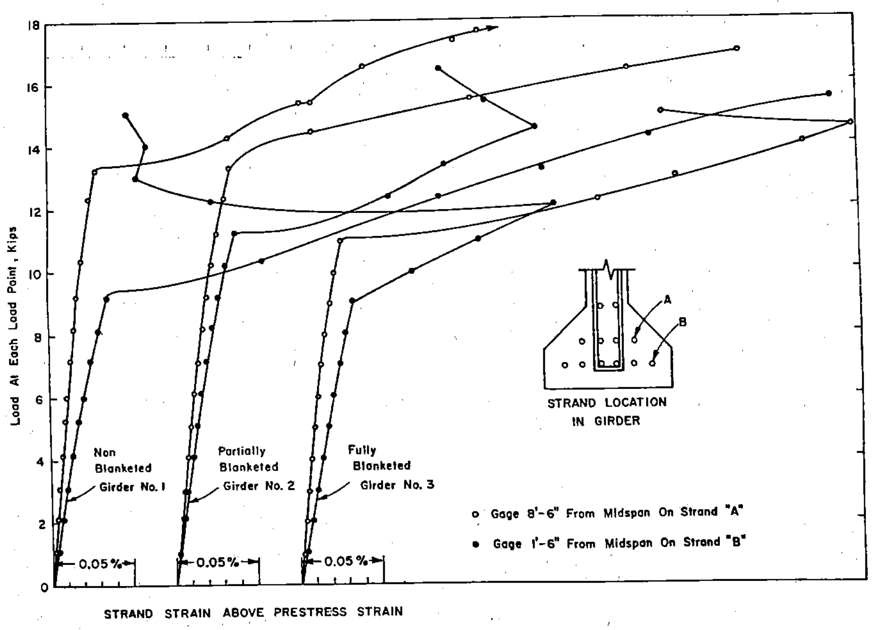


Fig. 10—Typical Load-Strain Relation

remained well above the original prestress strain.

In Girder 3 the outside strands at both the lower and the intermediate level were blanketed. The strain-load relationship clearly shows bond failure at the third increment after that producing concrete cracking in the vicinity of the strain gage. After bond slip the strain of the outside strand at the lower level decreased to well below the prestress strain.

In summary, all blanketed strands sustained some bond failure, while no bond failure occurred in unblanketed strands. As ultimate load was approached the strain level of some strands of the fully blanketed girder decreased to well below the initial prestress level, while the strands remained well above prestress in the case of the partially blanketed girder.

In all cases of sudden decrease in strain, such as illustrated in Fig. 10, the protruding strands at the girder ends began to "draw into" the girder as further evidence of bond failure soon after the records indicated a decreasing strain.

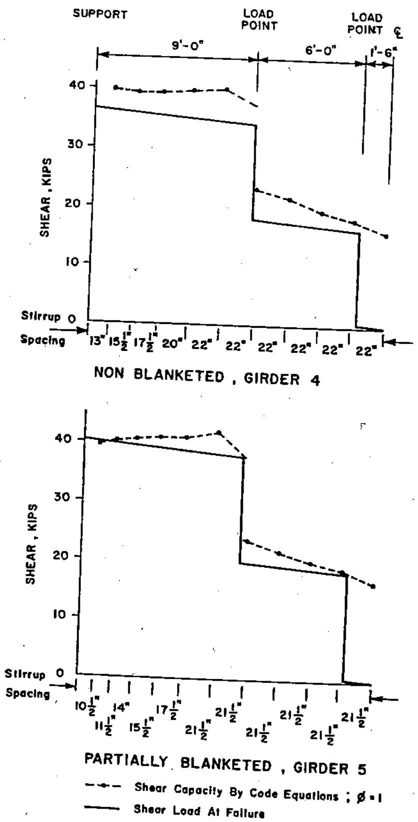


Fig. 11—Computed and Measured Shear Forces at Failure

TEST RESULTS—GIRDERS 4 AND 5

Strength

The computed and measured shear forces at failure for Girders 4 and 5 are shown in Fig. 11. Computed forces were determined from the actual properties of the materials used by Eq. (26-10) in Section 2610 of the 1963 ACI Code:

$$A_v = \frac{(V_u - \phi V_c)s}{\phi d f_y} \qquad (3)$$

in which

A_v = area of web reinforcement placed perpendicular to the axis of the member

ϕ = capacity reduction factor

V_u = shear due to specified ultimate load

V_c = shear carried by concrete

s = longitudinal spacing of web reinforcement

d = distance from extreme compression fiber to centroid of the prestressing force

f_y = yield strength of unprestressed reinforcement

The expression was rearranged and V_u calculated using ϕ equal to unity and f_y equal to 50.5 ksi. The diagonal cracking shear, V_c, in Eq. (3) was evaluated for each cross-section by the procedures given in Section 2610.

The ultimate moments of both

girders were calculated for the midspan section using Eq. (2) with ϕ equal to unity, the strand stress equal to the measured tensile strength, and concrete compressive strength equal to measured values.

Girder 4 failed in horizontal shear along the junction of the web and the sloping face of the lower flange as shown in Fig. 12. The failure plane extended from the region of the outermost load point to the end of the girder. The failure was instantaneous and occurred while the load was being increased to the next level. In the failure zone, the ratio of V_{test} to V_{calc} ranged from 0.88 to 0.92. The ratio at failure of M_{test} (420 ft.-kips) to M_{calc} (474 ft.-kips) was 0.89.

Girder 5 failed in flexure by fracture of all strands at a section near midspan as shown in Fig. 13. At ultimate the ratio of M_{test} (466 ft.-kips) to M_{calc} (461 ft.-kips) was 1.01. In the region from the inner load point to the end of the girder the ratio of V_{test} to V_{calc} ranged from 0.84 to 1.01, so that shear strength was practically exhausted.

Load-Deflection Relationship

The load-deflection curves for Girder 4 (non-blanketed) and Girder 5 (partially blanketed) are presented in Fig. 9 and are compared with the corresponding girders in the flexure series. It is seen that Girders 2 and 5 showed nearly identical characteristics, and both exhibited the typical ductility of a flexural failure. Girder 4, on the other hand, did not yield excessively. The flexural capacity of Girder 4 was not reached, and only a portion of the load-deflection curve of Girder 1 was duplicated.

Cracking

In both girders, cracking was confined mainly to the region between the outer load points. Diagonal cracks were of the type which progress from flexure cracks, and were observed in the area between the inner and outer load points. At the stage when the diagonal cracks had extended to the upper flange of the precast I-beam, horizontal cracks began to form at the junction of the web and the sloping face of the lower flange. In both Girders 4 and 5, just prior to failure, the horizontal cracks were continuous from the inner to the outer load points in both sides of midspan. The crack pattern of Girder 5 closely resembled that of Girder 4, indicating that the shear capacity was nearly exhausted when the strands fractured. No diagonal cracks were observed in either girder in the span near the supports; this was substantiated by the strain readings from the gaged stirrups.

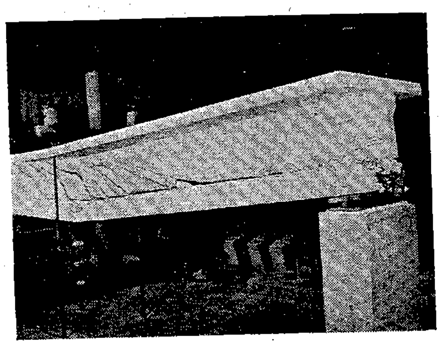


Fig. 12—Failure Areas—Girder 4

Fig. 13—Failure Areas—Girder 5

Strand Strain

The strand in Girder 4 showed no evidence of loss of bond at shear failure of the specimen. The strand strain increased to the end of the test in a manner similar to that shown in Fig. 10 for strand *A* of Girders 1 and 2. Readings from the dial gages at the girder ends indicated no differential movement between the strand end and the concrete.

In Girder 5, all four blanketed strands exhibited some draw-in at the girder ends. The two strands at the outside of the bottom row of reinforcement showed considerable slip into the girder; the other two strands with longer blankets had substantially smaller movements. Except for the gaged location near the failure section, all strand strains increased continuously to the end of the test. Strain readings from the line of gages adjacent to the fracture plane became erratic, and both increasing and decreasing strand strains were recorded as the ultimate load in flexure was approached. Local effects due to the proximity of the failure plane apparently were responsible for the erratic gage output.

CONCLUSIONS

- The three tests of T-beams utilizing dynamic loads showed no detrimental effects of strand blanketing on pretensioned members subjected to 5 million repetitions in the working load range.
- Beyond the cracking load and under static loading, some bond slip occurred for all blanketed strand.
- The results from the two tests in which the girders had less than the required shear reinforcement indicate no detrimental effects of blanketing upon shear strength.
- There is evidence that the 1963 ACI Building Code (ACI 318-63)[3] requirement for bond embedment length of strand in Section 2611 cannot be directly applied to blanketed strand. However, the performance in these exploratory tests of blanketed strand girders with embedment lengths twice those required by Section 2611 closely matched the flexural performance of a similar pretensioned girder entirely without blankets.

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Sebastian Inlet Bridge

by W. E. Dean*

A new design in long span precast prestressed concrete highway bridges has resulted from particular topographic conditions at Florida's Sebastian Inlet. The crossing on State Road A-1-A is located on the east coast, about midway of the peninsula and only 200 yards from the Atlantic Ocean. The inlet connects the Ocean and Indian River through a jetty-protected channel 600 feet wide crossing the narrow coastal barrier island. It is the only natural opening for a reach of 110 miles. The normal tidal range of about 2½ ft. and the large reservoir of the 1 to 3 mile wide Indian River produce currents of 6 to 8 fps at every tide cycle.

Channel criteria prescribed by the Corps of Engineers required a main span 180 feet long for a square bridge to cross the 30° skewed channel. Vertical clearance is 40 ft. above mean sea level. With its proximity to the open ocean, the structure will be subject to constant exposure to wind-driven salt-laden spray. This condition presents a severe corrosion problem for a steel structure. Several similarly located structures along the Florida coast require almost constant maintenance to provide a questionable protection for steel surfaces.

The topography and criteria resulted in the following principal considerations for design:

Principal Engineer, Howard, Needles, Tammen & Bergendoff, Orlando, Florida.

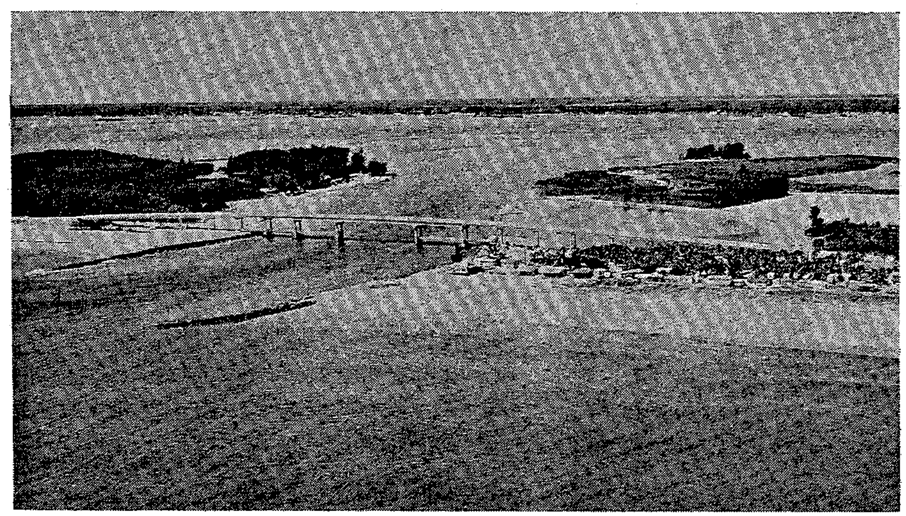

Fig. 1—Sebastian Inlet Bridge