ν = Poisson ratio for material of silo wall;
σ_1 = major principal stress (kPa);
σ_3 = minor principal stress (kPa);
σ_h = horizontal principal stress or pressure (kPa); and
σ_v = vertical principal stress or pressure (kPa).

Prestressed Composite Girders. I: Experimental Study for Negative Moment

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Abstract: Limited experimental results were reported in the literature on the behavior of prestressed composite girders subjected to negative bending moment. This paper examines experimentally the behavior of prestressed composite steel-concrete girders. The steel beams were all welded plate girders. Five composite specimens were tested to study the various aspects of prestressed composite girders, including the effects of construction stages, prestressing sequence, tendon type, and compactness of the plate girders on their structural performance. Load versus midspan deflection, strains in the concrete deck and steel girder, and the force increase in the prestressing tendons due to the applied loads were measured and studied. The test results showed that prestressing a composite girder in the negative moment region increases its stiffness by preventing the cracking of concrete under service loads; enlarges the elastic range; and increases the ultimate capacity due to the effective use of high-strength tendons.

Introduction

According to 1986 statistics from the U.S. Federal Highway Administration (FHWA), there are 574,729 bridges on the highway systems, about half of which are structurally deficient or functionally obsolete ("Highway" 1986). By attacking bridge deterioration early on, rehabilitation is possible and a costly replacement can be avoided. An effective design concept that can be used for retrofitting existing bridges as well as designing new structures involves the use of prestressing in composite steel-concrete girders. Prestressing composite girders combines high-strength steel tendons with structural steel and a concrete deck, which are compositely connected, to collectively resist dead and live loads. Adding high-strength tendons to a composite girder and providing prestressing enlarge the elastic range and increase the yield load and ultimate capacity of the girder (Saadatmanesh et al. 1989). Prestressing can be applied to simple-span or continuous-girder bridges. In negative moment regions of continuous-span bridges, the composite section can be prestressed using tendons along the top (tension) flanges. This places the concrete in compression, prevents cracking at service loads, and increases the stiffness and strength of the girders. It also reduces the stresses in the bottom (compression) flange of the steel girders, resulting in a higher yield load for compact sections and a higher flange buckling capacity for noncompact sections.

In this study, five prestressed composite steel-concrete girders (girders A, B, C, D, and E) were tested under negative bending moment. The objective of the experimental program was to study the effects of deck

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prestressing, in addition to steel beam prestressing, tendon type, steel compactness, concrete slab type, epoxy coating of tendons, construction stages, prestressing sequences, and the bond between the tendons and concrete, on the structural performance of the girders. Measurement of load, deflection, end rotations, strains in concrete and steel, and tendon force increase were recorded and compared with predicted values.

Previous Work

Composite construction has long been recognized as an economical means of resisting positive moments. However, a composite girder in a negative moment region is not as structurally effective as that in a positive moment region, due to concrete cracking. In 1975, Sarnes tested two composite steel-concrete beams with prestressed slabs in a negative moment region. The test results showed that slab prestressing in the negative moment region eliminated slab cracking and reduced stresses in the tension flange of the steel beams under service loads (Sarnes 1975). Kennedy and Grace (1982) studied the effect of the interaction between prestressing a portion of the deck slab and shear connectors in a negative moment region analytically as well as experimentally through the test of two one-eighth scale, two-span continuous bridges of compositive construction. The results showed that prestressing the deck increased the cracking load and stiffness of the girders. Saadatmanesh et al. (1989) studied the behavior of prestressed composite steel-concrete beams by testing two prestressed composite beams. One of the beams was tested under positive bending moment and the other under a negative bending moment. Both specimens were constructed of rolled beams compositely connected to nonprestressed concrete decks. Basu et al. (1987) studied experimentally and analytically the behavior of two-span partially prestressed composite beams. They concluded that partial prestressing increased the load capacity of the beam by about 20% and eliminated the problem of concrete deck cracking in the negative moment region. Dunker et al. (1986) studied posttensioning distribution in composite bridges, and Dunker et al. (1990) and Wiley et al. (1989) studied posttensioning methods for strengthening continuous bridges. They conducted model and field tests, and compared the results with predicted structural responses based on finite element analyses. The concepts of prestressed steel structures were also studied by Tochacek (1971) and Troitsky (1990).

In comparison to these previous studies, the contributions of the present study are to: (1) Study the effects of prestressing noncompact welded steel plate girders, compared with that of rolled steel beams, on their structural behavior; (2) examine the structural effects of prestressing the deck in addition to the steel beams; (3) compare the structural behavior of prestressed composite girders with uncoated and coated tendons for the deck; and (4) study the different prestressing sequences and schemes for the concrete deck and welded plate girders.

Experimental Program

Five composite girders, A, B, C, D, and E, were tested to their ultimate capacity under negative bending moment. Three other beams were tested to their ultimate capacity under positive bending moment. These three beams are not discussed in this paper. They are described in detail by Ayyub et al. (1990). The design details, including prestressing forces for the five

TABLE 1. Steel Girders

Girder (1)	Compression Flange: Thickness mm (in.) (2)	Compression Flange: Width mm (in.) (3)	Tension Flange: Thickness mm (in.) (4)	Tension Flange: Width mm (in.) (5)	Web: Thickness mm (in.) (6)	Web: Width mm (in.) (7)
A	12.7 (0.5)	147.6 (5.8)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)
B	9.5 (0.38)	196.9 (7.75)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)
C	9.5 (0.38)	196.9 (7.75)	7.9 (0.31)	133.4 (5.25)	9.5 (0.38)	508 (20)
D	12.7 (0.5)	147.6 (5.8)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)
E	9.5 (0.38)	196.9 (7.75)	7.9 (0.31)	133.4 (5.25)	6.4 (0.25)	685.8 (27)

TABLE 2. Concrete Slabs and Prestressing Tendons

Girder (1)	Concrete Slab[1]: Slab type (2)	Concrete Slab[1]: Tendon area mm² (sq in.) (3)	Concrete Slab[1]: Tendon type (4)	Concrete Slab[1]: Tendon bond (5)	Prestressing Tendon in Steel Girder[2]: Tendon area mm² (sq in.) (6)	Prestressing Tendon in Steel Girder[2]: Tendon force kN (kips) (7)
A	precast	592 (0.92)	uncoated	bonded	197 (0.31)	196 (44.0)
B	precast	595 (0.92)	uncoated	bonded	197 (0.31)	205 (46.0)
C	precast	592 (0.92)	uncoated	bonded	197 (0.31)	222 (50.0)
D	precast	595 (0.92)	epoxy coated	bonded	197 (0.31)	213 (48.0)
E	cast in place	592 (0.92)	uncoated	non-bonded	0 (0)	0 (0)

[1]Concrete slab size: width = 1,067 mm (42 in.); thickness = 102 mm (3.5 in.); prestressing force = 744 kN (167.3 kips); and prestressing tendons in the slab were positioned at its mid-depth.

[2]Prestressing tendons for plate girders were positioned 25 mm (1 in.) from the extreme fiber of tension flange of the steel girder.

girders, are summarized in Tables 1 and 2. The mechanical properties of concrete, steel girders, and tendons are given in Table 3.

As a result of a thorough literature search and a preliminary parametric analysis (Ayyub et al. 1988), an experimental program was designed to test five girders under a negative bending moment. Girders A and D, with compact flanges and noncompact webs, were tested to study the effect of epoxy coating of the strands on the structural behavior of the girders. Girders B and E, with noncompact flanges and noncompact webs were identical, except that girder B had precast prestressed concrete slab and girder E had

TABLE 3. Mechanical Properties of Materials

Element (1)	Yield Strength: Mean value MPa (ksi) (2)	Yield Strength: Standard deviation MPa (ksi) (3)	Ultimate Strength: Mean value MPa (ksi) (4)	Ultimate Strength: Standard deviation MPa (ksi) (5)
Steel girders A, B, C, D and E (in tension)	376 (54.5)	5.5 (0.8)	524 (76)	3.4 (0.5)
13 mm (0.500 in.) strands (in tension)	1,601 (233)	N/A	1,942 (281)	N/A
15 mm (0.600 in) strands (in tension)	1,620 (235)	N/A	2,017 (292)	N/A
Concrete slab of girder A (in compression)	N/A	N/A	24.8 (3.6)	4.68 (0.68)
Concrete slab of girder A (in tension)	N/A	N/A	2.4 (0.35)	0.45 (0.07)
Concrete slab of girder B (in compression)	N/A	N/A	36.2 (5.26)	0.31 (0.05)
Concrete slab of girder B (in tension)	N/A	N/A	3.3 (0.47)	0.44 (0.06)
Concrete slab of girder C (in compression)	N/A	N/A	37.9 (5.5)	0.82 (0.12)
Concrete slab of girder C (in tension)	N/A	N/A	2.9 (0.42)	0.04 (0.01)
Concrete slab of girder D (in compression)	N/A	N/A	34.9 (5.06)	0.72 (0.10)
Concrete slab of girder D (in tension)	N/A	N/A	3.3 (0.47)	0.25 (0.04)
Concrete slab of girder E (in compression)	N/A	N/A	35.9 (5.2)	1.46 (0.21)
Concrete slab of girder E (in tension)	N/A	N/A	3.3 (0.47)	0.06 (0.01)

Note: N/A = not applicable or not available.

posttensioned, cast-in-place concrete slab. Girder C had a noncompact flange and compact web. All specimens were tested to failure by being subjected to negative moments applied over their entire spans, as simply supported beams loaded at midspan. This test setup approximately simulated a portion of a two-span continuous girder between the inflection points on the two sides of an intermediate support. The approximate simulation was due to the linear bending moment diagram for the tested girders, compared with the nonlinear moment diagram for a continuous girder.

Test Specimens

Girder A

The 4.83 m (15 ft 10 in.) long prestressed composite girder had a span of 4.57 m (15 ft) between the end supports, and consisted of a welded steel plate girder and a precast concrete slab. The steel plate girder and the concrete slab were separately prestressed, then compositely connected, us-

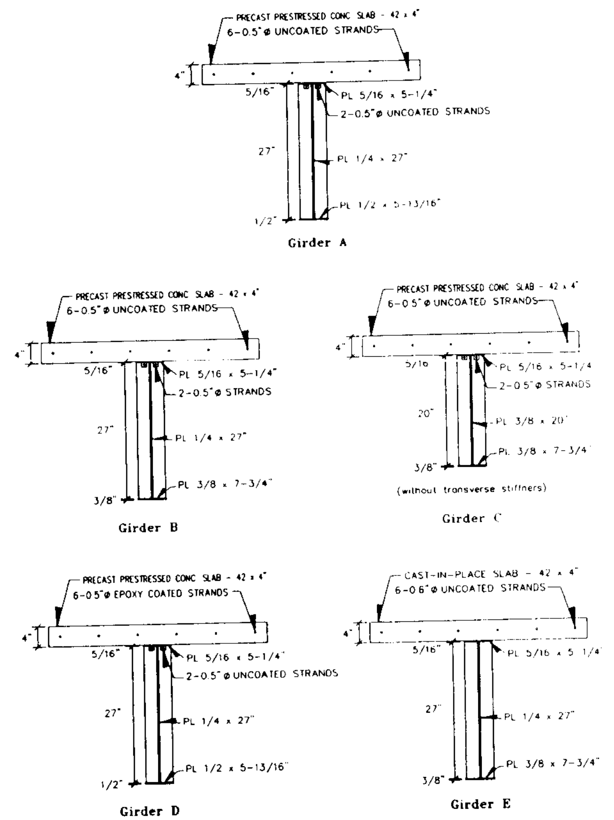

FIG. 1. Cross-Sections of Girders at Midspan (1 in. = 25.4 mm; 1 ft = 304.8 mm)

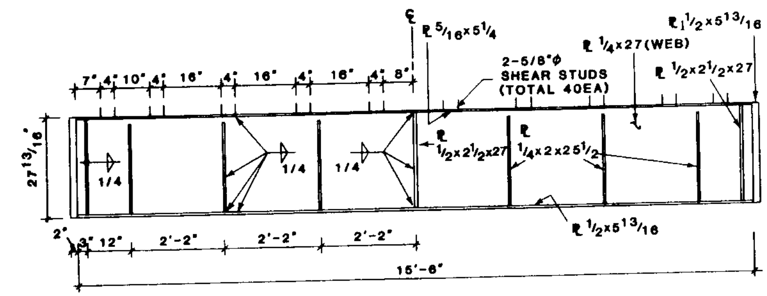

FIG. 2. Example Details of Steel Plate Girders A and D (1 in. = 25.4 mm; 1 ft = 304.8 mm)

ing grout to fill shear stud pockets provided in the slab. The cross section of girder A at midspan is shown in Fig. 1. The steel plate girder was made of the ASTM A588 steel, and consisted of a 7.5 mm (5/16 in.) thick by 133 mm (5.25 in.) wide steel plate as the tension flange, 6.4 mm (1/4 in.) thick by 686 mm (27 in.) deep web, and a compression flange of 12.7 mm (0.5 in.) thick by 148 mm (5.81 in.) wide plate. Three pairs of 6 mm (0.25 in.)

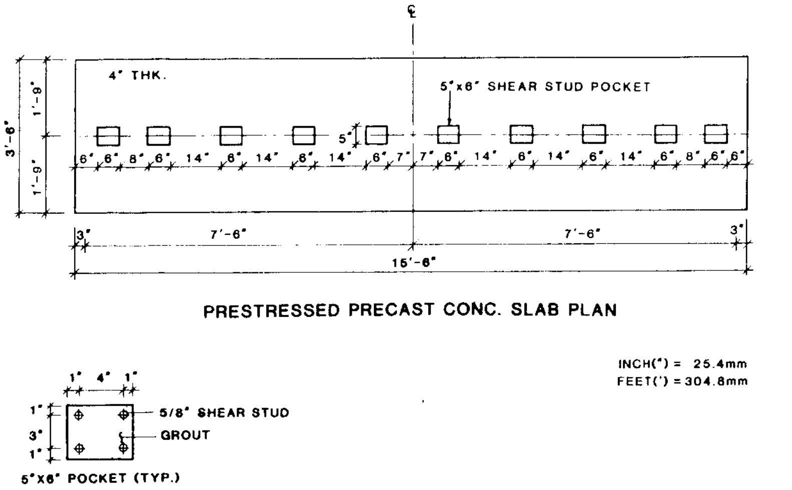

FIG. 3. Details of Precast Prestressed Concrete Slab (1 in. = 25.4 mm; 1 ft = 304.8 mm)

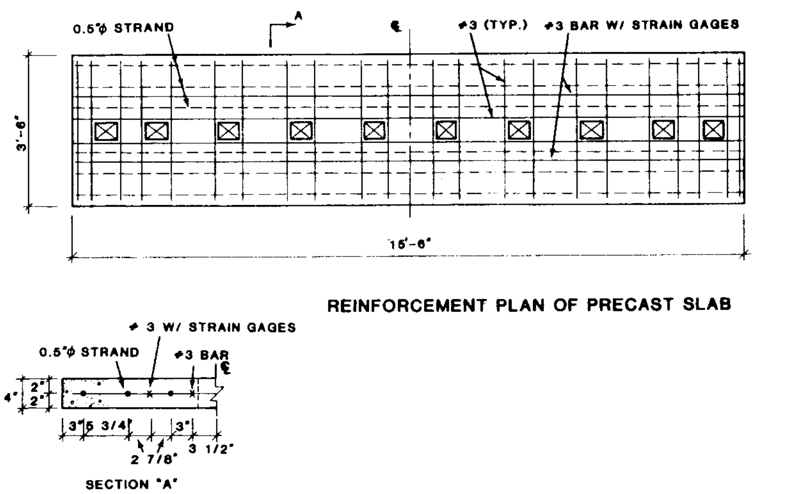

FIG. 4. Reinforcing Details of Precast Prestressed Concrete Slab (1 in. = 25.4 mm; 1 ft = 304.8 mm)

bearing stiffeners were welded to the web of the plate girder at the end supports and midspan under the loading point. Six pairs of 6 mm (0.25 in.) thick transverse stiffeners were welded at 660 mm (2 ft 2 in.) spacing to the web of the plate girder, as shown in Fig. 2. Two 38 mm (1.5 in.) thick end plates were welded at both ends of the beam to facilitate the anchorage of the tendons. Forty 16 mm (5/8 in.) diameter, 63 mm (2.5 in.) long shear studs (connectors) were welded to the top (tension) flange in groups of four. The plate girder was prestressed by two 12.7 mm (0.500 in.) diameter, grade 270 low-relaxation seven-wire strand (ASTM A416), running the full girder length 25 mm (1 in.) below the top (tension) flange. The precast prestressed

concrete slab was 1,070 mm (3 ft 6 in.) wide by 100 mm (4 in.) thick by 4,720 mm (15 ft 6 in.) long as shown in Fig. 3. Six 12.7 mm (0.500 in.) diameter grade 270 low-relaxation seven-wire strands (ASTM A416) were prestressed in the mid-depth of the concrete slab. Pretensioning and pre-casting techniques were used to construct the slab. Instrumented deformed reinforcing bars, 10 mm (3/8 in.) in diameter, were placed longitudinally and transversely around the 127 mm (5 in.) by 152 mm (6 in.) shear pockets to prevent cracking and facilitate measurement of concrete strains. The specified yield strength of the reinforcing bars was 414 MPa (60 ksi). The details of the precast concrete slab and its reinforcements are provided in Figs. 3 and 4. The precast prestressed concrete slab was compositely attached to the prestressed plate girder by grouting the shear-stud pockets of the slabs.

Girder B

The design details of this girder were similar to those of girder A except for the compression-flange dimensions. The compression flange of girder B was 9.5 mm (3/8 in.) thick by 197 mm (7.75 in.) wide, which is a noncompact section, whereas the compact compression flange of girder A was 12.7 mm (0.5 in.) thick by 148 mm (5.8 in.) wide plate. The cross section of girder B at midspan is shown in Fig. 1. The prestressing force of the tendons for the steel girder was slightly larger than the corresponding tendons in girder A, as shown in Table 2.

Girder C

This girder was similar to girder B except for its web dimensions and the absence of transverse stiffeners. The compact web of girder C was 9.5 mm (3/8 in.) thick by 508 mm (20 in.) deep. The cross section of girder C at midspan is shown in Fig. 1.

Girder D

Girder C was geometrically identical to girder A. All the components of the plate girder had the same dimensions as those of girder A. The pre-stressing of the plate girder was performed by two 12.7 mm (0.500 in.) diameter grade 270 low-relaxation seven-wire strands (ASTM A416), running the full girder length 25 mm (1.0 in.) below the top (tension) flange. The dimensions of the concrete slab and the method of construction were similar to those of girder A except that the prestressing strands, placed at the mid-depth of the concrete slab, were epoxy coated. The cross section of girder D at midspan is shown in Fig. 1.

Girder E

The steel plate girder of specimen E was identical to the steel plate girder of specimen B. The concrete slab, 1,070 mm (3 ft 6 in.) wide by 100 mm (4 in.) thick by 4,760 mm (15 ft 6 in.) long, was cast in place and compositely connected to the plate girder by means of shear studs. Six 25.4 mm (1 in.) diameter plastic conduits for posttensioning tendons were embedded in the concrete slab. Two 10 mm (3/8 in.) diameter deformed bars, with three strain gages mounted on each bar surface, were embedded longitudinally to measure the concrete slab strain. Two additional 10 mm (3/8 in.) diameter longitudinal reinforcing bars were placed in the slab. A total of 24 transverse 10 mm (3/8 in.) diameter bars were also provided in the slab. This rein-forcement was provided as temperature and shrinkage steel. After the concrete had sufficiently hardened, the composite girder was posttensioned by six 15.2 mm (0.600 in.) diameter, Grade 270 low-relaxation seven-wire strands (ASTM A416). The cross section of girder E at midspan is shown in Fig. 1.

Instrumentation and Testing Procedure

The girders were instrumented for the purpose of measuring deflections, end rotations, strains across the depth, prestressing force increase, applied load, and slip between the steel beam and concrete slab. All strain gages were placed 102 mm (4 in.) away from midspan, which is the point of load application. Five concrete strain gages were placed on the top surface of the concrete slab and four on the bottom surface of the slab. Two reinforcing bars with three strain gages each were embedded at the mid-depth of the concrete slab. Five steel strain gages were placed on the top (tension) flange and five under the bottom (compression) flange of the plate girders. Fourteen strain gages were placed along the depth of the web. One donut load cell was installed at the end of one strand for girders A, B, C, and D to measure the increase in the prestressing force due to load application. Another load cell was used to measure the applied load. The deflection at midspan was measured using two electronic deflection gages. The deflections at both ends were also recorded. End rotations were measured using electronic clinometers attached to the webs at the ends of the plate girders. A dial gage was placed at the end of the girder to measure any relative slip between the concrete slab and plate girder.

To simulate the negative moment region over the support, between the inflection points of a continuous bridge, the girders were inverted such that the concrete slab was at the bottom resting on the supporting edges. The girders were tested by applying a single concentrated load at midspan with roller (simple) end supports. The concentrated load was applied from the top at midspan of the specimen representing the support (pier) of an actual bridge. All girders, except girder B, had lateral supports for the compression flange at every quarter point along the span. The specimens were tested using a displacement control testing setup with 890 kN (200 kips) capacity test frame. The applied load was increased in specified increments in the elastic range using a manual hydraulic jack, pump, and double-acting ram. After initial yield of the specimen, the load was increased in equal increments of deflection. All girders were unloaded and reloaded twice before failure. The maximum applied load selected was smaller than the specimen's cracking load for the first cycle of loading, and larger than the cracking load for the second cycle. During the test, the specimens were checked for cracking of the concrete slab and buckling of the steel beam after each load increment. The test setup is shown in Fig. 5(*a*), and an example buckling failure for girder B is shown in Fig. 5(*b*).

Comparison of Test Results

The test results are summarized in Figs. 6–14. The figures include the experimental results and analytical values in solid and dashed lines, respectively. The analytical results were based on the assumption of complete composite action without any slip between the steel girder and concrete slab. The analytical curves were terminated at the yield loads, because the majority of the tested girders failed by buckling of the compression flange and the postbuckling analytical predictions were not stable. The analytical

FIG. 5(*a*). Test Setup, e.g., Girder C

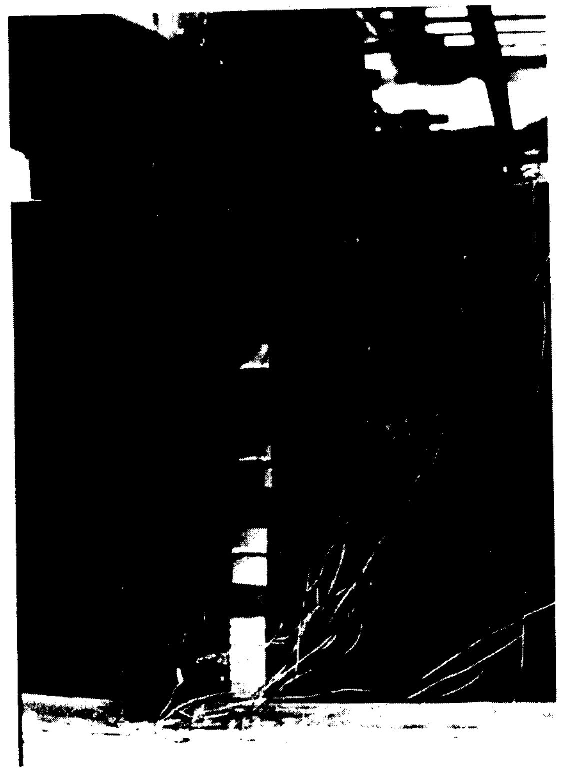

FIG. 5(*b*). Buckling Failure, e.g., Girder B

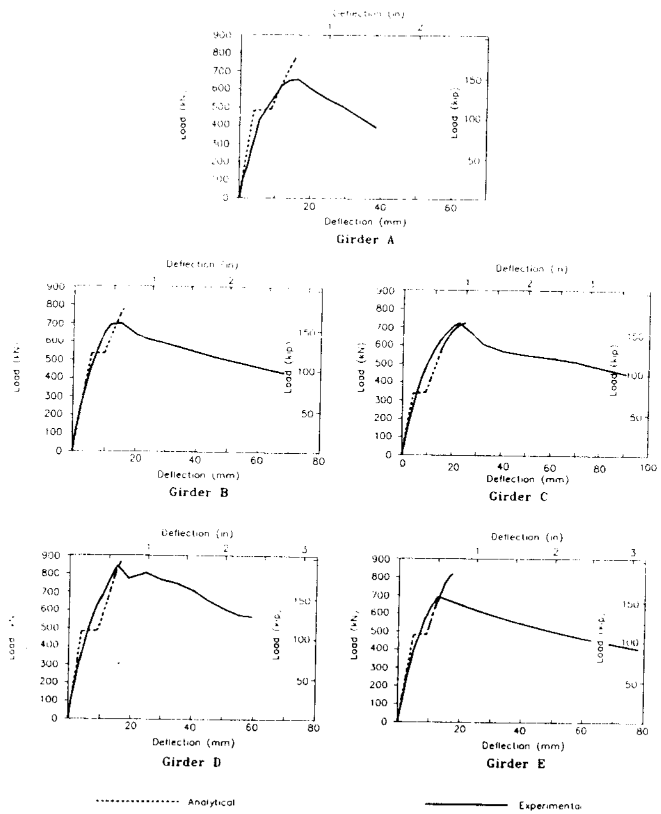

FIG. 6. Load-Deflection Behavior

curves in Figs. 6 and 7 were based on both flexural and shear deformations. They consist of an uncracked curve and cracked curve, which correspond respectively to the stages before and after the concrete in the slab reached the modulus of rupture. The details of the development of the analytical results are described in the companion paper (Ayyub et al. 1990).

The measured and analytical load-deflection curves for girder A are shown in Fig. 6. The measured load-deflection curve is well contained between the uncracked and cracked lines of the analytical curve. At a measured load of 455 kN (100 kips), the concrete slab started to crack due to tension, and the stiffness of the girder consequently reduced. Since the girder was not laterally supported, the girder started lateral torsional buckling at a load of 618 kN (139 kips), resulting in a progressively nonlinear load-deflection curve. The maximum resisting load was 665 kN (144 kips), and the load-deflection curve declined as buckling extended from the compression flange to the web. The test showed that the forces of the prestressing tendons in the slab deterred the propagation of concrete cracking. The load-versus-end-rotation curves were similar to the load-deflection curves, and are shown in Fig. 7. The amount of slip between the steel girder and the concrete slab was measured as shown in Fig. 8. The slip resulted in a reduction in the composite action and the girder's stiffness. The analytical study was based

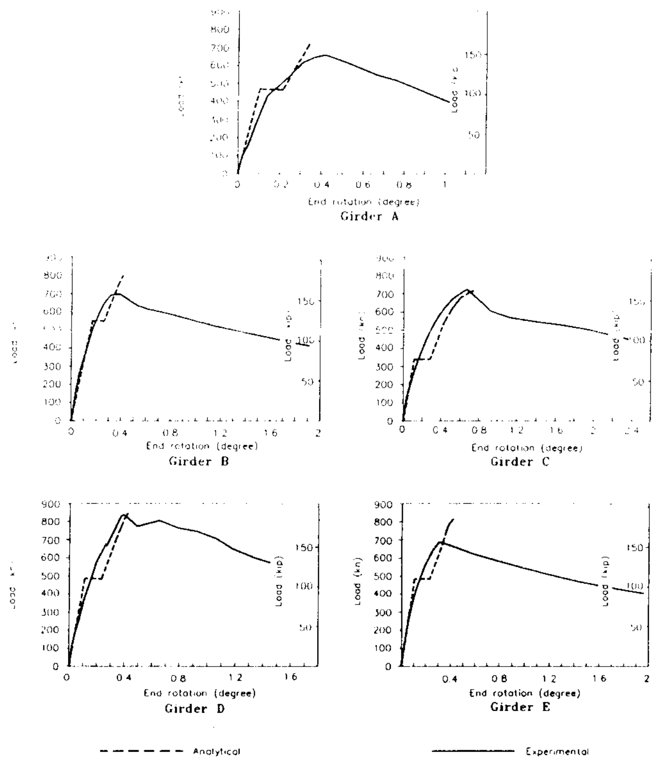

FIG. 7. Load-End Rotation for Girders

on the assumption of complete composite action (Ayyub et al. 1990). The load versus the increase in strand force of the steel girder is shown in Fig. 9. Since the analytical curve, shown with the dashed lines, was based on the assumption of complete composite action between the concrete slab and steel girder, the predicted force increase in the tendons due to the application of the load was larger than the measured one. The load versus strain in mid-depth of the concrete slab, the top steel flange, and bottom steel flange are shown in Figs. 10, 11, and 12, respectively. The measured strains across the composite girder depth are shown in Fig. 13. As the slip between the concrete slab and the steel girder increased, the neutral axis moved toward the bottom flange. Additionally, the strain distribution across the depth became nonlinear, as shown in Fig. 13, resulting in reduced strains in the concrete slab due to the applied loads. The lateral torsional buckling of the girder occurred at a loading level slightly lower than the predicted yield load.

For girder B, the amount of slip between the steel girder and the concrete slab was slightly smaller than that of girder A. The load-deflection and load-rotation curves for girder B are shown in Figs. 6 and 7, respectively. The load versus force increase for the prestressing strands in the steel girder is shown in Fig. 9. The measured increase in the force was smaller than the predicted one based on complete composite behavior due to the slip between the steel girder and concrete slab. The strains in the concrete and the two

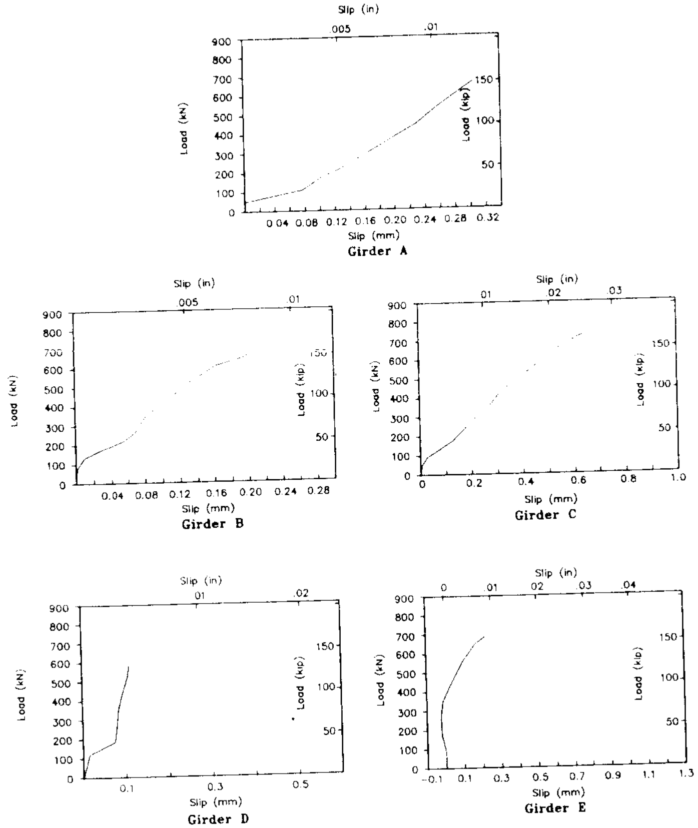

FIG. 8. Load Slip for Girders

steel flanges due to the applied loads are shown in Figs. 10, 11, and 12, respectively. This girder failed by crippling (local buckling) of the noncompact compression (bottom) flange, that resulted in limiting the load-carrying capacity of the girder. As the buckling extended from the compression flange to the web, the load-deflection and load-rotation curves started to decline. The predicted load-deflection and load-rotation curves correlated well with the measured ones at all load levels until failure by buckling.

Girder C was the only girder with a compact web. Due to the small initial deformation in its web, compared with that of the other girders, it was able to withstand the highest prestressing force, and it reached its ultimate strength without local buckling. Its load-deflection curve, as shown in Fig. 6, became progressively nonlinear once the compression flange started to yield. The maximum resisting load was 735 kN (165 kips), which is close to the analytical ultimate load. The load-rotation curve is shown in Fig. 7. The load versus increase in strand force was similar to that of girder B and is shown in Fig. 9. The load-strain curves for the concrete slab and steel flanges are shown in Figs. 10, 11, and 12, respectively.

Girder D was similar to girder A with the exception of six coated strands as opposed to six uncoated strands in the precast concrete slab of girder A. Girder D was laterally supported during the test. The measured and ana-

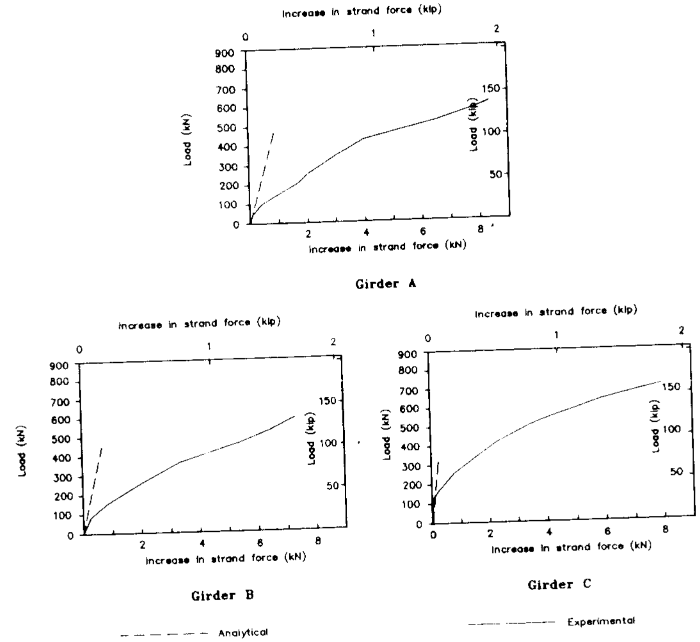

FIG. 9. Load Increase in Strand Force for Girders A, B, and C

lytical load-deflection and load-rotation curves for girder D are shown in Figs. 6 and 7, respectively. The two sets of curves correlated well at all levels of load. The load versus slip between the concrete slab and the steel girder is shown in Fig. 8. The increase in strand force could not be measured for girder D because the donut load cell used to measure the strand force failed during the experiment. The load-strain curves for the concrete slab and steel flanges are shown in Figs. 10, 11, and 12, respectively.

The steel plate girder of specimen E was similar to that of specimen B. However, the concrete slab of girder E was cast in place, unlike the precast prestressed slab of girder B. The prestressing of girder E was provided only in the concrete by means of posttensioning six unbonded tendons. Compared with the other girders, the load-induced slip between the concrete slab and the steel girder was smaller because the concrete slab was cast in place. Due to the unbonded nature of the prestressing strands in the concrete slab, only one big crack at about midspan was developed rather than many hairline cracks in the case of the bonded strands. As shown in the respective Figs. 6 and 7, the measured and the analytical load-deflection and load-rotation curves correlated well. The load versus slip between the concrete slab and the steel girder curve is shown in Fig. 8. The load-strain curves for the concrete slab and steel flanges are shown in Figs. 10, 11, and 12, respectively.

Based on Table 2 and Figs. 6 and 7 for girders C and E, the most effective construction sequence for prestressed composite girders in negative moment regions was to posttension the composite girder using tendons placed in the cast-in-place concrete slab. This is due to the relatively larger eccentricity for the prestressing force for girder E compared with girder C, resulting in a more efficient use of the prestressing steel. The same objectives could

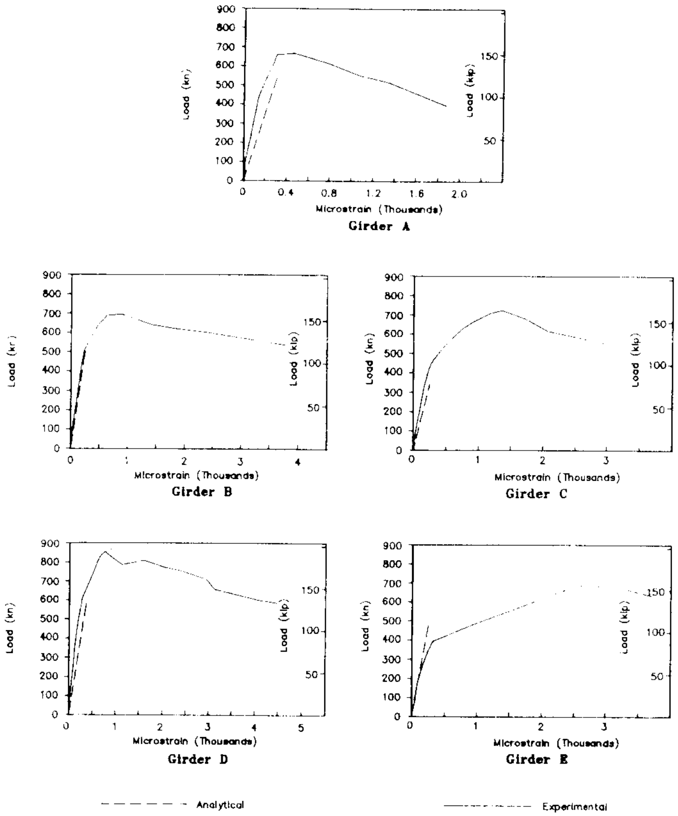

FIG. 10. Load Strain for Midthickness of Concrete Slab

have been achieved by posttensioning a precast slab instead of a cast-in-place slab.

Effect of Epoxy-Coated Strands on Girder Behavior

The coated strands are designed to offer corrosion resistance in combination with bond-transfer characteristics equal to the current uncoated strand capabilities. Girder D, with six epoxy-coated strands in the precast concrete slab, showed the same or better strength as girder A, which had uncoated strands. Girder D had a higher ultimate load capacity than girder A, as shown in Figs. 6 and 7. The higher strength was attributed to the lateral support provided for girder D rather than the effect of the coated tendons. Girder D with the coated tendons showed no sign of bond failure during the test. The test also established that coated strands have adequate flexural bond characteristics and better cracking characteristics in comparison to the uncoated strands.

Effect of Lateral Support

For prestressed composite beams subjected to positive bending moment, the concrete slabs provide a continuous lateral support for the compression zone of the beam. In the case of girders under negative bending moments,

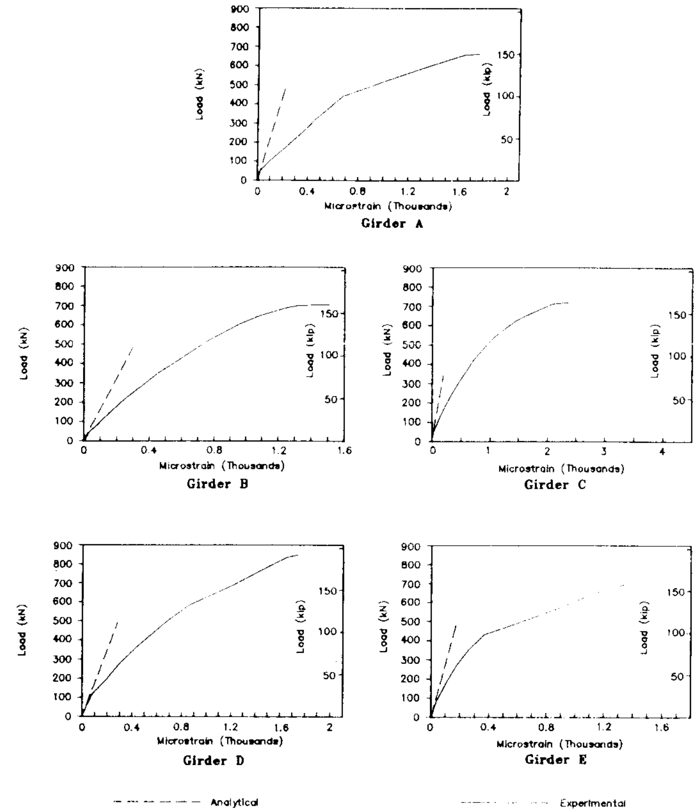

FIG. 11. Load Strain for Top Steel Flange

a lateral support system is required to prevent buckling of the compression flanges of the girders.

Girders A and D were tested to study the effect of lateral supports on the structural performance of the girders. Girder D was provided with a support system according to AASHTO bridge specifications (1983). Girder D reached a higher ultimate load than girder A, which was not laterally supported. The maximum resisting loads for girders D and A were 854 kN (192 kips) and 665 kN (150 kips), respectively, as shown in Figs. 6 and 7. Hence, it can be concluded that adequate lateral support is very important for prestressed girders subjected to a negative bending moment, and can be provided using the requirements for nonprestressed girders [e.g., *Standard* (1983)].

Compactness of Compression Flange and Web

To maximize the benefits of a plate girder, it should be built out of thin steel plates designed to avoid any instability (buckling) problems. Girders A and D had compact compression flanges, and girders B and E had noncompact compression flanges. Girder D, which had a compact compression flange, developed a postbuckling strength (tension field action) and a maximum resisting load that was larger than the yield load, as shown in Figs.

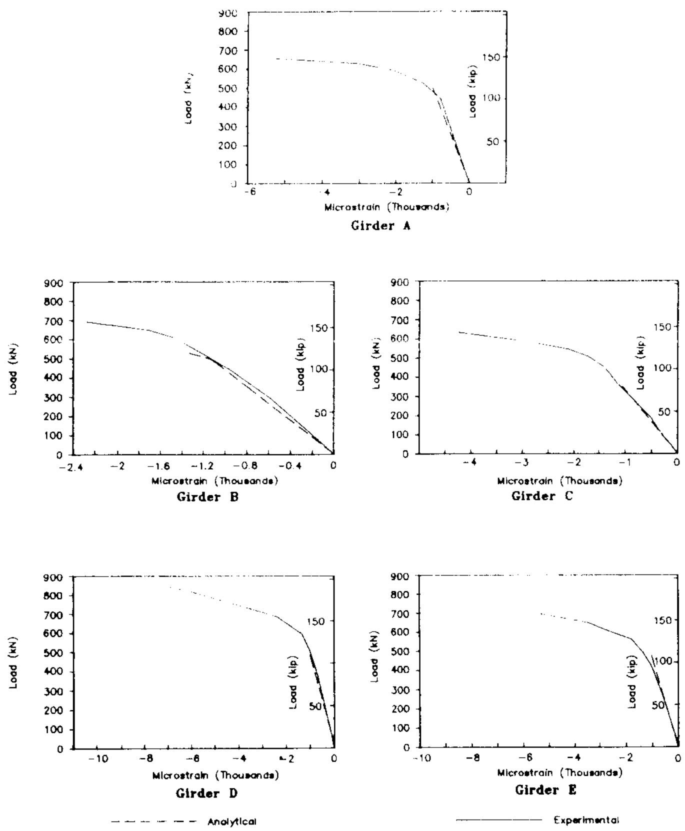

FIG. 12. Load Strain for Bottom Steel Flange

6 and 7. Girders B and E, which had noncompact compression flanges, did not have postbuckling strength, and their maximum resisting loads were almost the same as the corresponding predicted yield loads. Therefore, for the same cross-sectional area of a plate, it is recommended to make the compression flange plate thicker and the width narrower, i.e., a compact section. Girder C consisted of a compact web and a noncompact compression flange. The compact web sustained the girder far beyond the yield load to the ultimate load. Out of the tested specimens, only girder C reached its plastic ultimate load. However, it might not be economical to make a plate girder with a compact web because the depth-thickness ratio is very low for the compact section, resulting in higher structural steel weight. These findings and recommendations are similar to the corresponding properties of nonprestressed girders.

Precast and Cast-in-Place Concrete Slabs

Girder B with prestressed precast slab was compared with girder E, which had a cast-in-place slab. The overall performance of both girders was similar. Both girders reached the same maximum resisting load at about the predicted yield load. The prestressed precast concrete slabs had relatively better

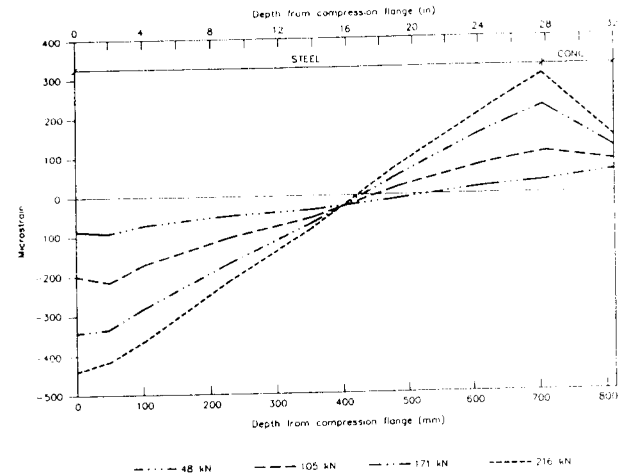

FIG. 13. Strain Distribution Across Depth of Girder A (1 in. = 25.4 mm)

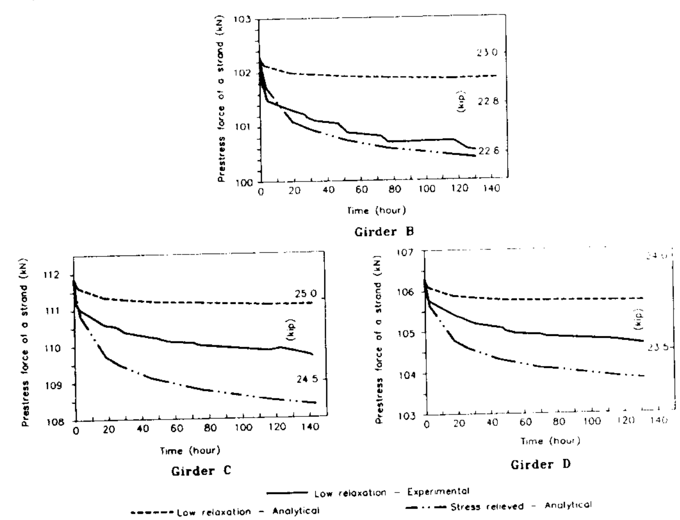

FIG. 14. Loss of Prestressing Force for Steel Girder Strands

mechanical properties as a result of better quality control in the pretensioning and precasting plant.

Prestress Losses

The steel plate girders of specimens A, B, C, and D were prestressed with two 12.7 mm (0.500 in.) diameter strands, then compositely connected to the precast prestressed slabs. To study the prestress loss of the strands, prior to connecting the girders to the slabs, the prestressing forces of girders B, C, and D were monitored. The prestress loss was caused primarily by the relaxation of the strands, anchorage set, and to a lesser extent, by the creep of the steel girders. Extensive research was done by Magura, Sozen, and Siess to study the relaxation properties of stress-relieved wires and strands (Magura et al. 1964). Based on this study, a relationship for stress in the prestressing strands as a function of the time t was provided as follows:

$$f_{ps}(t) = f_{pi}\left[1 - \frac{\log(t)}{K}\left(\frac{f_{pi}}{f_{py}} - 0.55\right)\right] \qquad (1)$$

where $f_{ps}(t)$ = stress in a prestressing strand at time t; f_{pi} = initial stress; t = time in hours (not less than one); f_{py} = yield stress of the prestressing steel; and K = constant. Eq. (1) was developed for prestressing strands. The constant K takes a value of 10 for stress-relieved strands and 45 for low-relaxation strands. The ratio of f_{pi} to f_{py} should always be greater than 0.55. Fig. 14 shows the measured prestressing force of the strands as a function of time for girders B, C, and D. In the first four hours, a rapid prestress loss occurred, then the curve flattened gradually starting at 20 hr. The calculated relaxations of the strands using (1) are shown in the same figure. Low-relaxation seven-wire strands were used in this study. The difference between the measured and calculated values is attributed to uncertainties in the coefficient K. The creep of the steel girder is believed to be minimal, based on the experimental results, the constant K was determined to be about 25–30 for low-relaxation strands.

Conclusions and Recommendations

Five girders were tested under a negative bending moment. The experimental program was developed to study various aspects of prestressed composite girders, including compactness of plate girders, concrete slab type, epoxy coating on tendons, lateral supports, construction sequence, prestressing stages, and prestress losses. Deflections, end-rotations, strains, increases in tendon forces, and slip between the concrete slab and steel girder due to the applied loads were measured during the tests. The analytical values agreed well with the experimental results. Based on the experimental results of the prestressed composite girders, the following conclusions and recommendations were drawn.

The slip between the concrete slab and the steel girder was small enough to justify the assumption of complete composite interaction between the slab and the steel girder in the elastic range. However, the slip became larger after the yield load. Therefore the measured deflection of the girder was slightly larger than the deflection based on the assumption of composite action.

Prestressing the slabs in composite girders that were subjected to negative bending moments prevented the cracking of the concrete slab under service loads; consequently, it increased the elastic strength of the girders, reduced deflections, and reduced the steel area required for the top (tension) flange of the steel girders.

The epoxy-coated strands showed flexural bond characteristics that were similar to the bond characteristics of currently used uncoated strands.

For the same cross-sectional area of a compression flange plate, it is recommended to make the plate thicker and the width narrower, which means a compact section. The girder with a compact compression flange had a reserve capacity, which was its postbuckling strength.

The prestressed precast concrete slab for the prestressed composite girder had the same or better structural performance and quality than the cast-in-place slab. Moreover, use of precast prestressed slabs is expected to reduce the construction cost and time and enhance the quality and strength of the materials.

The most effective construction sequence for prestressed composite girders in negative moment regions was determined to consist of a posttensioned composite girder with tendons in the concrete slab.

To use the proposed prestressing methods for strengthening existing bridges, the concrete decks of the bridges must be removed in order to use precast prestressed concrete slabs or to place tendons in the negative moment region prior to casting new concrete decks.

Acknowledgment

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APPENDIX II. NOTATIONS

The following symbols are used in this paper:

f = stress;
K = stress relaxation factor for prestressing strands; and
t = time.

Subscripts

pi = initial stress in prestressing steel;
ps = stress in prestressing steel; and
py = yield stress of prestressing steel.

PRESTRESSED COMPOSITE GIRDERS. II: ANALYTICAL STUDY FOR NEGATIVE MOMENT

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ABSTRACT: A method for the structural analysis of prestressed composite steel-concrete girders was studied in this paper. The deflections, forces in the prestressing tendons, and strains in the steel beam and concrete slab of composite girders were computed throughout the entire loading range up to failure. Equations are provided for the calculation of the yield and ultimate load capacities of the girders. The developed analytical models were based on the incremental deformation method. The results of the analytical study were compared with test results of several girders. Reasonably good correlations between analytical and experimental results were obtained. Also, the results showed that a substantial increase in the yield and an increase in the ultimate load capacities can be achieved by adding prestressing tendons to the composite girders and prestressing them. It was determined that the most effective construction sequence for prestressed composite girders in negative moment regions is to posttension the composite girders with tendons in the concrete slabs.

INTRODUCTION

The prestressing of girders in structures introduces initial stresses and strains in directions that are opposite to those induced by external loads. Prestressing composite girders in positive moment regions has long been recognized as an efficient structural system. However, prestressing composite girders in negative moment regions is just starting to receive a general acceptance by the engineering community. In the negative moment region of continuous nonprestressed composite girders, some conservative design practices ignore any contributions of the reinforced concrete slabs of the composite girders and consider only the contributions of the steel beam sections to the structural capacities of the girders. These practices are commonly attributed to the cracked behavior of the concrete slabs. Prestressing composite girders in negative moment regions can prevent cracking of concrete slabs in the service conditions, and reduces the deflection of the girders due to their increased stiffness. Fatigue strength can also be greatly improved by reducing the tensile stress ranges in the top flanges of the girders. The ultimate capacity and the elastic range of the structure can also be increased by adding tendons to the composite girders and prestressing them. Prestressing the critically stressed areas of the girder reduces stresses and deformations, increases the load-carrying capacity, and results in an economical design with efficient use of materials. Providing multiple load paths in the tension region of a girder consisting of the flange and the prestressing

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