Testing of Segmental Concrete Girders With External Tendons

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Construction of Interstate 526 over the Cooper and Wando Rivers, near Charleston, South Carolina includes 8 miles (13 km) of bridges and viaducts. In the preliminary design stage of this project some innovative concepts for segmental concrete construction were proposed. Because of the large investment involved, it was imperative to evaluate the structural performance and economy of the proposed systems.

The owner of the bridge, South Carolina Department of Highways and Public Transportation, in cooperation with the Federal Highway Administration, recommended that tests be conducted to compare the performance of two innovative segmental construction systems with a conventional system, under short term static loading. This was accomplished through fabrication and testing of three match cast segmental concrete girders.

SIGNIFICANCE OF RESEARCH

The conventional system for post-tensioning segmental concrete bridges uses internal tendons.[1] In this system, ducts that house the post-tensioning tendons are located within the main concrete cross section. At all joints, the ducts of adjacent segments have to be carefully positioned to meet precisely. Securing these ducts in place inside the forms requires extra care and time. In addition, the ducts interfere with casting and consolidation of concrete. Special precautions are required to prevent concrete intrusion into the ducts at both ends of each segment.

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On the other hand, the external tendon system facilitates construction and maintenance. Advantages of the external prestressing system have been discussed by Virlogeux.[2] They include the following:

1. Manufacturing time, a major economic concern for all contractors, is reduced by eliminating duct laying and adjustment operations.
2. A more streamlined tendon layout can be achieved, imparting a reduction in frictional losses and thereby improving the efficiency of the prestressing system.
3. Placing and consolidation of concrete are simplified.
4. Web thickness and therefore the weight of the superstructure may be reduced.
5. Potential problem of concrete intrusion in the ducts during casting is eliminated.
6. Grout injection is simplified. It is easier to check the grouting process and to assess and repair inadequately grouted areas.
7. With proper design, replacement of broken or corroded tendons is possible.

Construction speed and economy associated with external tendon systems make them more attractive than internal tendon systems. In external prestressing systems, although the ducts may be grouted, the tendons act as if unbonded. As a result, when strength considerations govern, a slightly larger tendon area may be needed to obtain a nominal flexural strength equal to that of internal bonded systems.

Synopsis

Three 30 ft long (9.14 m) match cast segmental concrete girders were loaded statically to destruction. The test variable was the location of the tendon ducts. In the first girder, the ducts were embedded within the girder cross section. The ducts of the second girder were external to the concrete cross section except at pier segments and intermediate deviation blocks. The third girder was similar to the second except that portions of the external ducts were embedded in a second stage concrete cast. The segments included multiple shear keys and were dry jointed. All ducts were grouted.

The first and third girders attained the flexural strength predicted by the classic bending theory for bonded tendons. The flexural strength of the second girder (external tendons) was lower than that of the first and third girders but considerably higher than predicted by the AASHTO equation for flexural strength of members with unbonded tendons.

Until more refined analytical procedures become available, it is recommended that the tendons of segmental members with external grouted post-tensioning systems be designed using the procedures for unbonded tendons given in the 1983 AASHTO Specifications for Highway Bridges.

OBJECTIVES

The objectives of this experimental investigation were to confirm theoretical analyses and to compare the behavior of partially bonded external tendon systems with a completely bonded internal tendon system. The effects of potential damage to anchorages at girder ends due to a severe earthquake were to be evaluated through releasing the anchor wedges of selected tendons prior to loading the girders to destruction. Loss of anchorages in the event of a severe earthquake relates to structural redundancy.

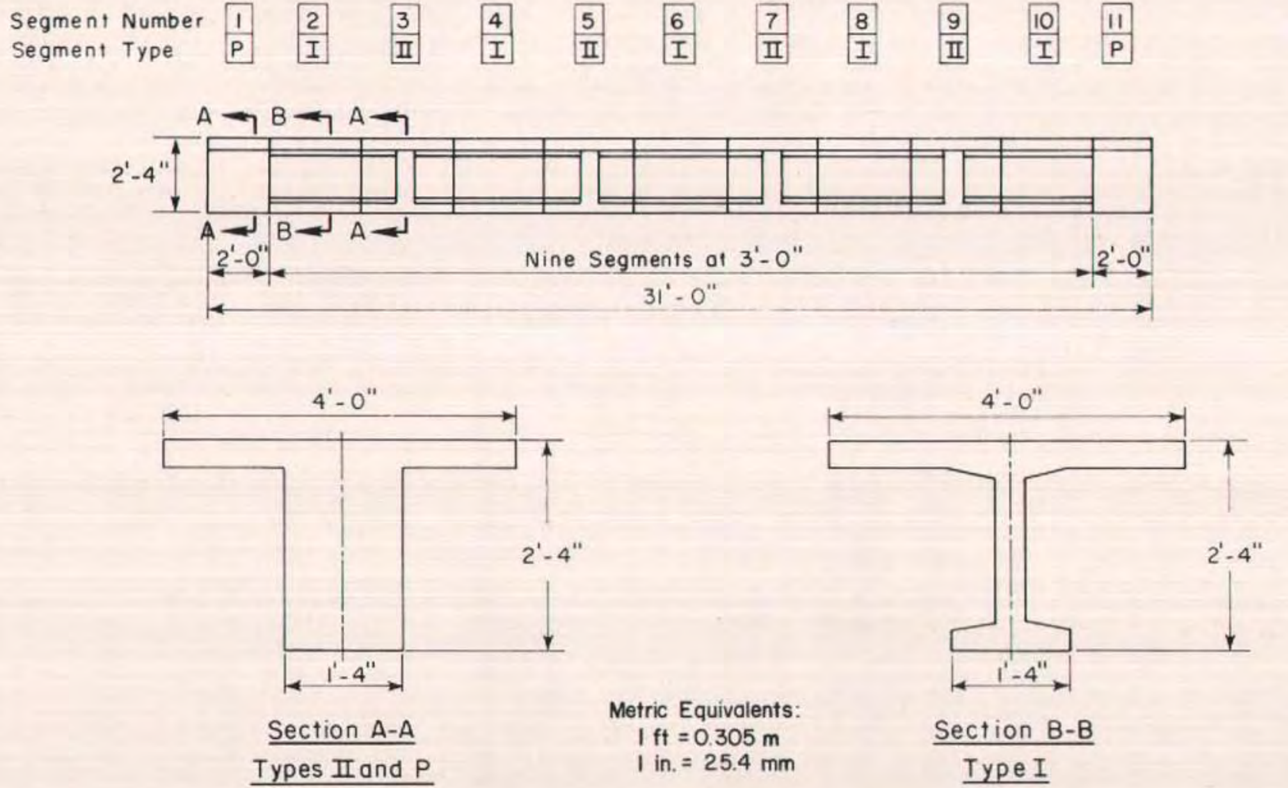

Fig. 1. Principal dimensions and details of girders tested.

TEST SPECIMENS

Three segmental girders were fabricated and tested. The first girder, called the Bonded Tendon Girder, was post-tensioned with internal tendons. The second girder, called the Unbonded Tendon Girder, was post-tensioned with external tendons. The third girder, called the Modified Unbonded Tendon Girder, was similar to the second girder, except that a second stage concrete cast was placed on top of the bottom flange, covering portions of the external ducts, for bond development.

The tested girders represented an approximate one-fifth scale model of the bridge prototype. The bridge prototype consists of a single cell trapezoidal box. However, for simplicity and economy, a decked bulb T section was selected for the test specimens. Shear keys between adjacent segments were not fully scaled down in the test specimens.

Test Variable

The controlled test variable was the location of the strand ducts in the test girders. This variation can be seen in some of the illustrations.

Geometry

Each girder included eleven match cast segments of three types as shown in Fig. 1. Type I segment was a prismatic section. Type II segment was similar to Type I but included a full depth diaphragm in the middle of the segment. Strands were deflected at these diaphragms.

The diaphragms incorporated saddles as used in deviation blocks of segmental box girders. The pier segment, Type P, was a solid T section. Size and location of shear and alignment keys are shown in Fig. 2. A midspan cross section for each girder is given in Fig. 3. Tendon profiles of the three girders are plotted in Fig. 4.

Material Properties

The concrete mix was designed for a target one-day compressive strength of 2500 psi (17.2 MPa), for stripping of forms, and a target 14-day compressive strength of 5500 psi (37.9 MPa), for post-tensioning. Maximum aggregate size was ⅜ in. (9.5 mm) pea gravel. The mix used for grouting the tendon ducts consisted of Type I portland cement, water, and a water-reducing admixture.

At girder testing time, concrete compressive strength of each segment was determined from 6x12 in. (152x305 m) standard cylinders. Average concrete compressive strength ranged between 5810 and 7660 psi (40.0 and 52.8 MPa). Average concrete compressive strength of the secondary cast of the Modified Unbonded Tendon Girder was 5250 psi (36.2 MPa) at test time.

Low-relaxation seven-wire strands conforming to ASTM Designation: A 416, Grade 270 (1862 MPa) were used. Two 0.5 in. (12.7 mm) and four 0.6 in. (15.2 mm) diameter strands were used in each girder as shown in Fig. 3. Measured tensile strength of 0.5 and 0.6 in. (12.7 and 15.2 mm) diameter strands were 276 and 278 ksi (1903 and 1917 MPa), respectively.

Reinforcing steel conformed to ASTM Designation: A 615, Grade 60 (414 MPa). Shear reinforcement in each segment consisted of four vertical No. 4 (12.7 mm) reinforcing bars at each face of the web.

Specimen Fabrication

Techniques used in fabrication of the test girders simulated actual construction procedures for full scale match cast segmental concrete bridge girders. A short line match casting system was used. Appropriate steps were taken to ensure geometry control. Casting started from both ends with pier segments. They were cast against a bulkhead. Subsequent segments were match cast

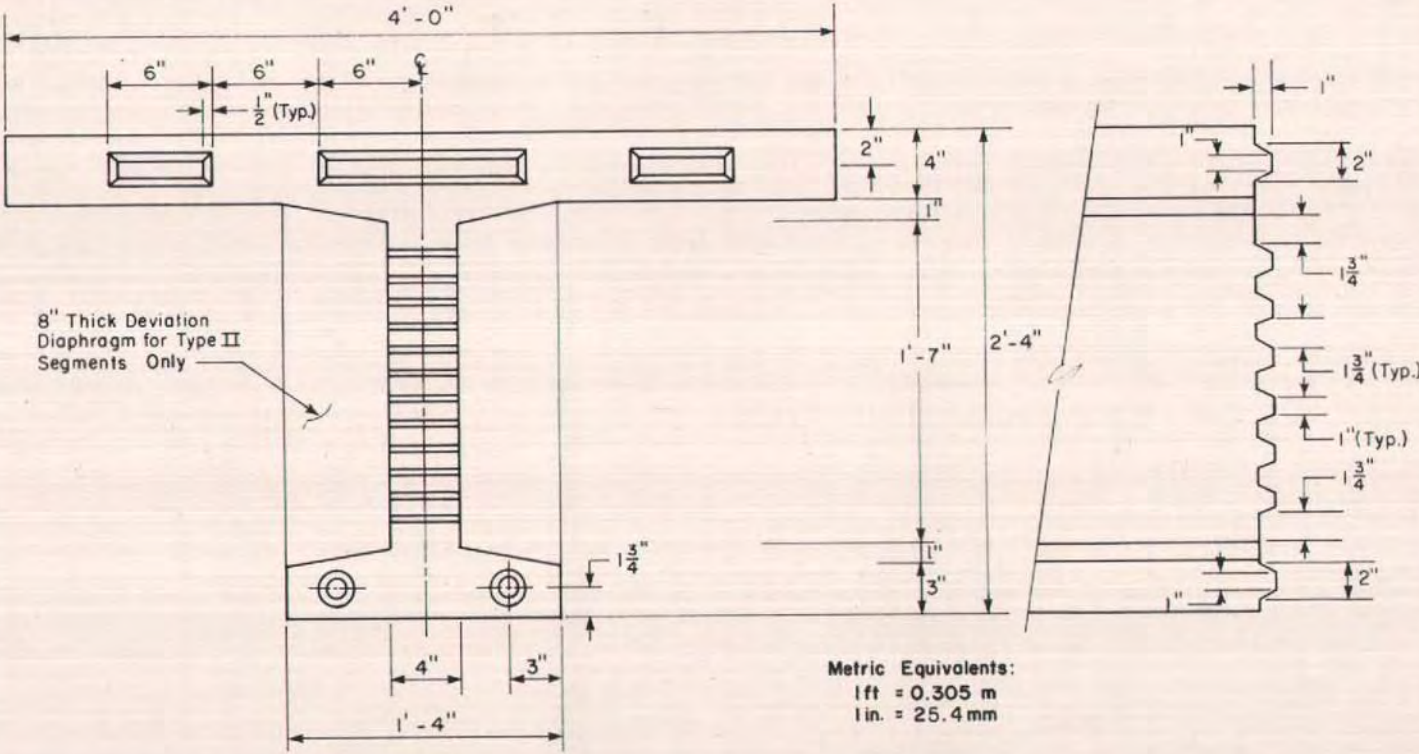

Fig. 2. Cross-sectional dimensions including shear and alignment keys.

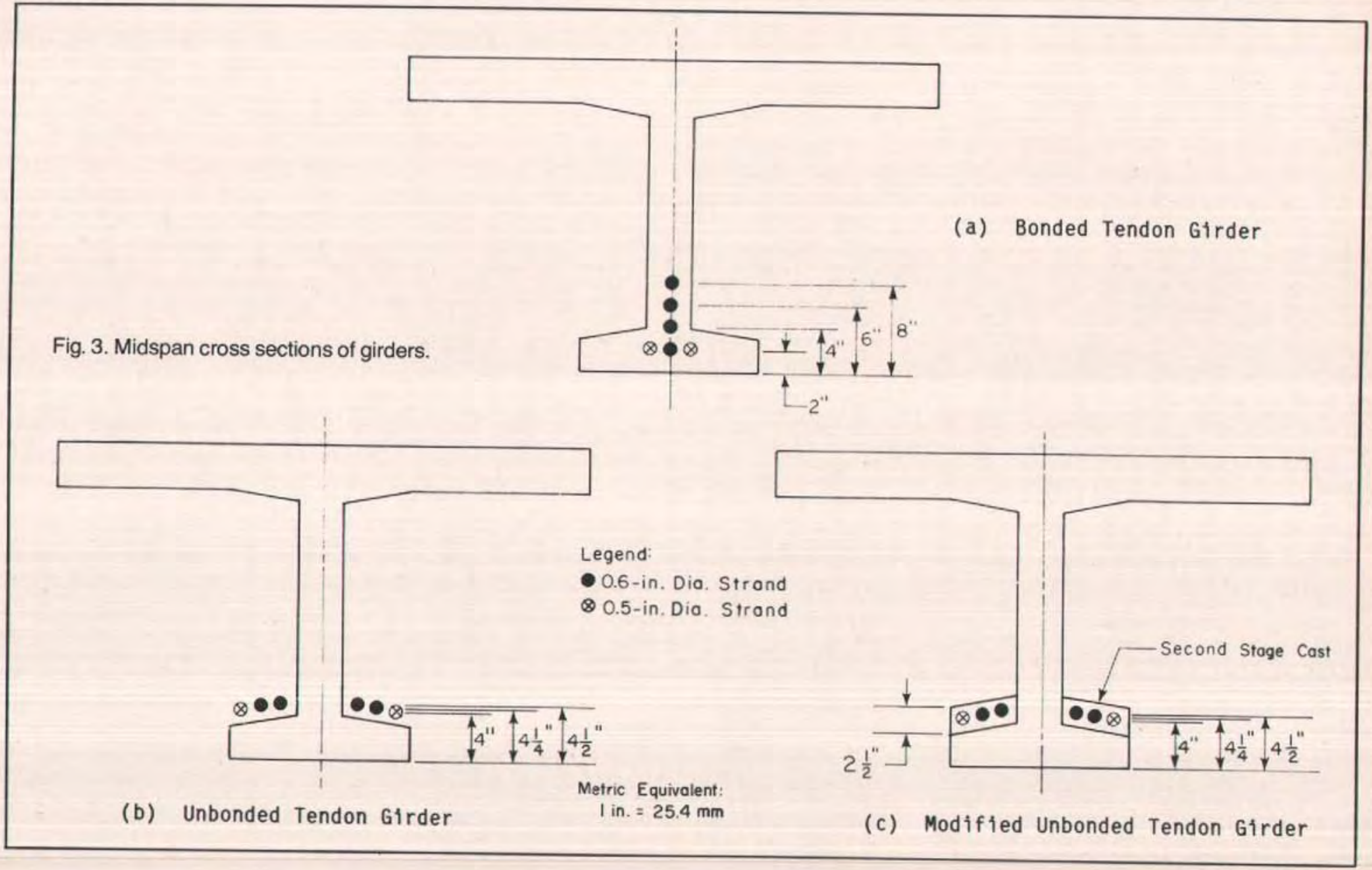

Fig. 3. Midspan cross sections of girders.

against an adjacent segment and a bulkhead. Finally, the middle segment was match cast against two adjacent segments.

After casting, the top surface of each segment was troweled and covered with polyethelene sheets. On the next day, the polyethelene sheets were removed, forms were stripped, and the two match cast segments were separated by pulling them apart carefully. All segments were stored under laboratory conditions of 73°F (23°C) and 50 percent relative humidity for about 5 weeks, before testing.

In the Modified Unbonded Tendon Girder, shear connectors were added to the bottom flange to ensure composite action between the second stage cast and the original concrete. G-shaped No. 4 (12.7 mm) reinforcing bars served as shear connectors to the bottom flange on either side of the web. They were welded to the web reinforcement at one end and to the bottom flange reinforcement at the other end. To accommodate welding of shear connectors, prior to casting, expanded polystyrene strips were secured to the reinforcement.

After stripping the forms, the polystyrene strips were removed. The detail of shear connectors proposed for the actual

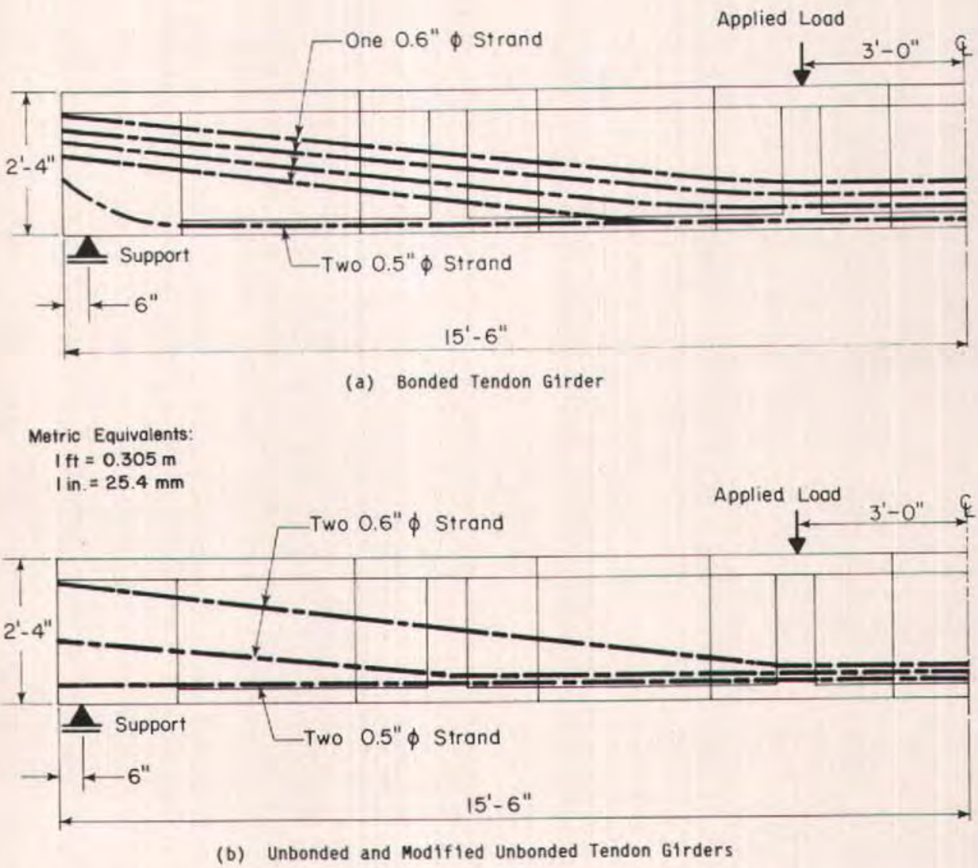

Fig. 4. Tendon profiles of Bonded Tendon Girder (top) and Unbonded and Modified Unbonded Tendon Girders (bottom).

bridge girders differed from the one used in the laboratory and was more suited for large scale production. After post-tensioning the Modified Unbonded Tendon Girder, a 2.5 in. (63.5 mm) thick second stage concrete cast was placed on top of the bottom flange to cover shear connectors and tendon ducts.

Prior to post-tensioning each girder, all segments were pulled together tightly, on a flat and level surface, using temporary connectors. At the joints of the Bonded Tendon Girder, special precautions were taken to prevent grout leakage. At the interface of adjacent segments, a thin bead of caulking compound was applied around each tendon duct, a distance of 1 in. (25.4 mm) away from the duct edge.

Each test girder included two 0.5 in. (12.7 mm) and four 0.6 in. (15.2 mm) diameter seven-wire low-relaxation strands. Each strand was housed inside a 1 in. (25.4 mm) inside diameter duct. Jacking force for 0.5 and 0.6 in. (12.7 and 15.2 mm) diameter strands was 28.9 and 41.0 kips (129 and 182 kN), respectively. The eccentricity of prestress was similar for all three girders.

Strain gauges installed on the tendons were monitored during prestressing. Average midspan strand stresses, based on measured strand strains, corresponded to 161 and 140 ksi (1110 and 965 MPa) for 0.5 and 0.6 in. (12.7 and 15.2 mm) diameter strands, respectively. The difference in effective prestress may be attributed to the difference in seating losses and the sequence of post-tensioning.

In each girder, tendon ducts were grouted after tensioning all six strands. The joints between segments were dry and free of grout or adhesives.

TEST PROGRAM

Test Setup

A photograph of the test setup is shown in Fig. 5. Each girder was simply supported over a 30 ft (9.14 m) span.

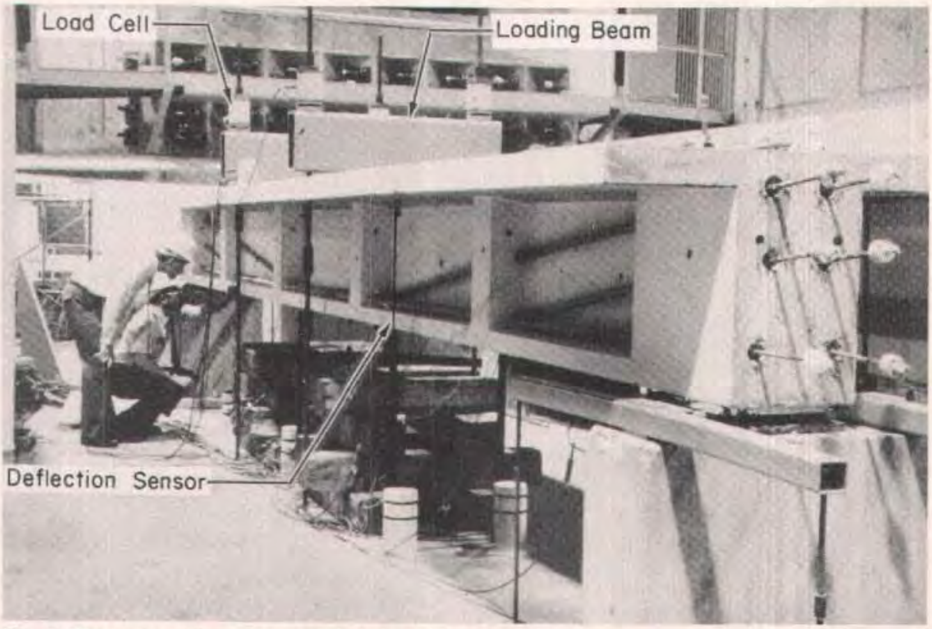

Fig. 5. Test setup of experimental girder with external tendons.

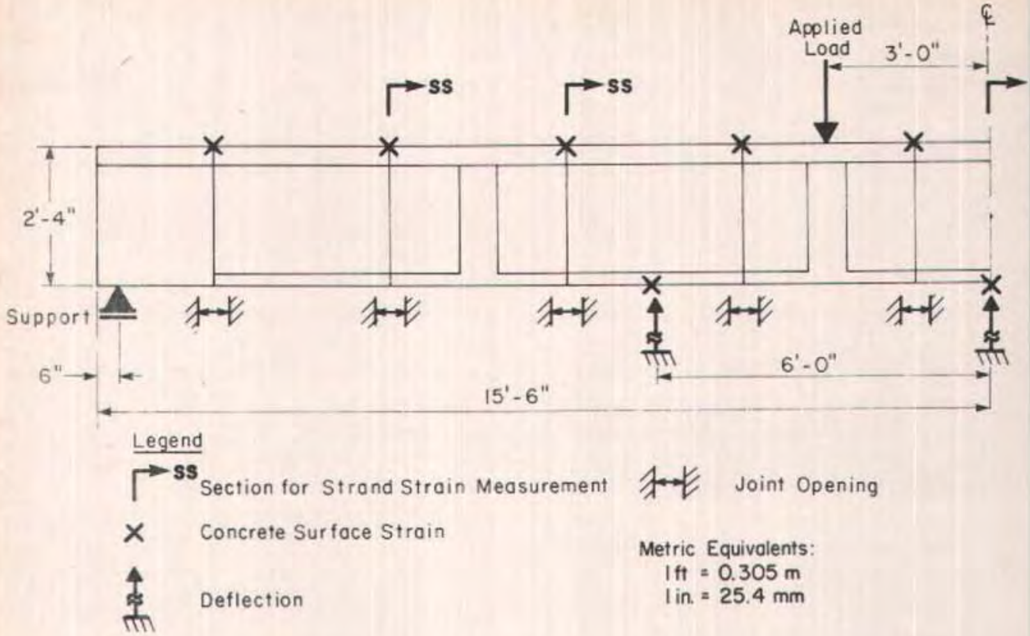

Fig. 6. Instrumentation plan for experimental girder.

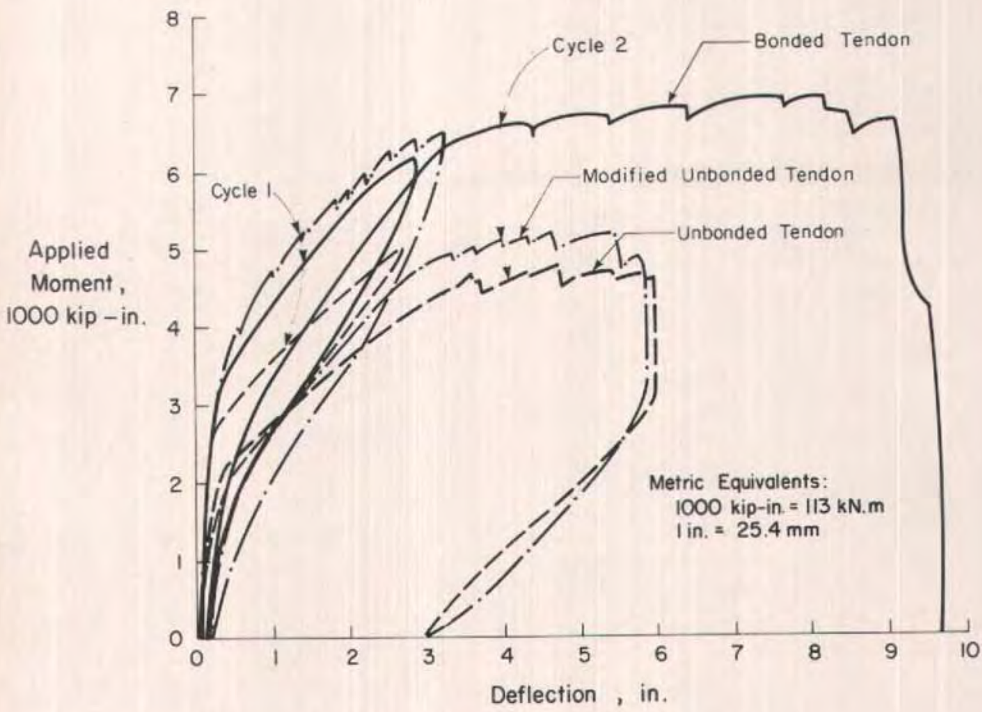

Fig. 7. Midspan applied moment versus deflection relationships.

Loads were applied along two sections located 6 ft (1.83 m) apart and equidistant from midspan.

Instrumentation

Measurements recorded during each test included applied loads, deflections, joint openings, concrete surface strains, and strand strains. The instrumentation plan was identical for all three girders, and is schematically shown in Fig. 6.

Data Acquisition

All measurements were scanned through an electronic data acquisition system and transmitted to a microcomputer. During the test, all measurements were reduced to engineering units and displayed continuously on a CRT screen. This allowed monitoring of the test progress. At selected load increments, reduced data were printed on a hard copy and stored on a computer disk. A continuous plot of midspan applied moment versus deflection was also recorded on an X-Y plotter.

Test Procedure

Each girder was subjected to two loading cycles. In the first cycle, each girder was loaded in small increments until an inelastic behavior was observed and a midspan deflection of approximately 3 in. (76 mm) was reached. The girder was then completely unloaded. Subsequently, anchor wedges for the top two strands were burned at both ends to simulate anchorage loss due to a severe earthquake. In the second loading cycle, each girder was loaded in increments up to destruction. Each test was concluded when a significant drop in load carrying capacity of the girder was observed.

Simulation of anchorage loss was a last minute addition to the test program. It was an attempt to "hit two birds with one stone." Production of segments was in progress when the decision was made to burn the anchor wedges of the top two strands. Had the decision been made earlier to release the anchor wedges, pier segments would have been designed differently. Further, the test specimens had monostrand anchors. The actual bridge includes "trumpet" anchors for multiple strands. The conical shape of a trumpet anchor provides better confinement and slip resistance for the strands than a straight anchor, once the grout hardens. Enhanced behavior would have been anticipated had the pier segments been longer and/or trumpet anchors been used.

BEHAVIOR OF SPECIMENS

Relationships of midspan applied moment versus deflection for the three tested girders are illustrated in Fig. 7. Peak applied moments and corresponding deflections for first and second loading cycles are summarized in Table 1.

In the initial portion of the first loading cycle, the three girders exhibited similar stiffnesses. As loading progressed, the Bonded Tendon and the Modified Unbonded Tendon Girders behaved similarly while the Unbonded Tendon Girder exhibited a larger deflection for the same applied moment. Loads resisted by the test girders during the first loading cycle were compared at a midspan deflection of 2.65 in. (67 mm). This was the maximum deflection of the Unbonded Tendon Girder during the first loading cycle. The Bonded Tendon and the Modified Unbonded Tendon Girders required 22 and 28 percent more load than the Unbonded Tendon Girder, respectively, to reach this midspan deflection.

After the first loading cycle, each girder was completely unloaded and the anchor wedges for the top two strands were burned. In the Bonded Tendon Girder, because the ducts were em-

Table 1. Summary of test data.*

Test girder	Dead load moment (kip-in.)	Peak moment during first loading cycle		Residual deflection† (in.)	Peak moment during second loading cycle	
		Applied moment (kip-in.)	Deflection (in.)		Applied moment (kip-in.)	Deflection (in.)
Bonded Tendon Girder	690	6220	2.95	0.13	6910	7.68
Unbonded Tendon Girder	690	4980	2.65	0.05	4670	4.31
Modified Unbonded Tendon Girder	720	6530	3.26	0.18	5080	4.34

* All tabulated moments and deflections correspond to midspan. Applied moments exclude dead load moment due to weight of girder and loading hardware.
† After first loading cycle.
Metric Equivalents: 1000 kip-in. = 113 kN•m; 1 in. = 25.4 mm.

bedded in the concrete cross section and were grouted, the tendons were bonded and transfer of prestress occurred in the end regions. Although the anchorages were burned, the top two strands remained bonded. For the two external tendon girders, measurement of strand strains indicated a loss of bond for the top two strands at the end regions, immediately after burning of the anchorages. As a result, the effectiveness of about one-third of the tendons was lost. Applied load versus strand strain relationship for a top strand of the Unbonded Tendon Girder is illustrated in Fig. 8. This strand strain was measured at a section 10.5 ft (3.2 m) away from midspan.

Fig. 8 indicates a slight increase in the strain of the top two strands during the second loading cycle. The ducts of these strands were embedded in the pier segments and deviation diaphragms. The friction between the strands and the grout was probably adequate to stabilize that small tensile force in the strands. However, the remaining force in these strands was negligible. In the second loading cycle, loss of bond of the top two strands lead to considerable reduction in the load carrying capacity of the external tendon girders.

All three girders were assembled dry jointed. During the tests, opening of the joints between adjacent segments was monitored at the level of the bottom fibers. Opening of joints during the first and second loading cycles is plotted in Figs. 9 and 10, respectively. In Fig. 9, maximum opening of joints in the Modified Unbonded Tendon Girder was larger than that of the Unbonded Tendon Girder because each girder was loaded up to a different deflection during the first loading cycle.

With all three specimens, Joints 1-2 and 10-11, identified in Fig. 1, remained closed throughout testing. Similarly, Joints 2-3 and 9-10 of the Unbonded Tendon and Modified Unbonded Tendon Girders remained closed throughout testing. In the Bonded Tendon Girder, maximum opening of Joint 2-3 was 0.028 and 0.066 in. (0.7 and 1.7 mm) at the peak moments of the first and second loading cycles, respectively. At Joint 9-10, the corresponding openings were 0.018 and 0.049 in. (0.5 and 1.2

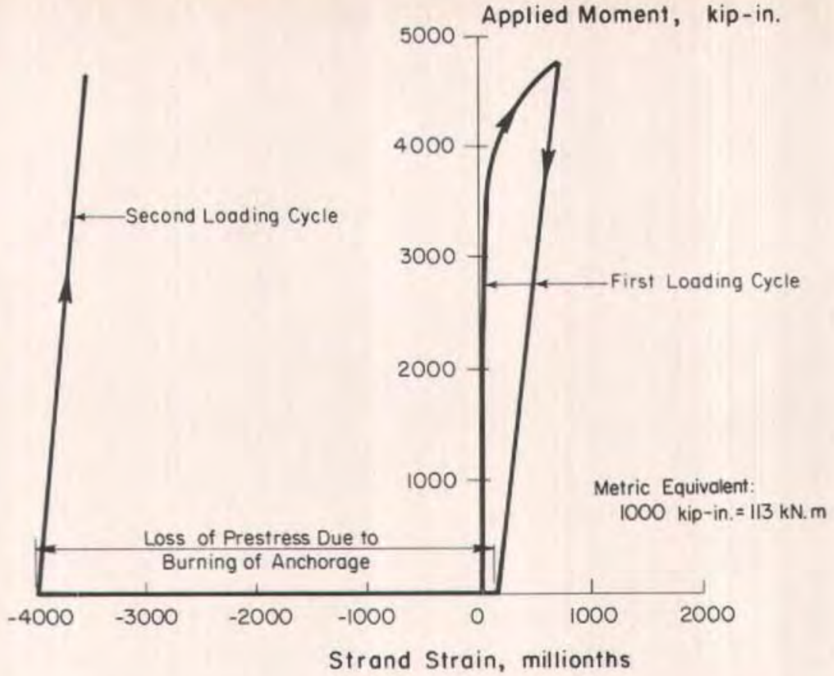

Fig. 8. Representative strand strain for a top strand in the external tendon girder.

mm), respectively.

The following sections include a description of the behavior of each test girder. Segment numbers are identified in Fig. 1. Each joint is identified by the numbers of adjacent segments.

Bonded Tendon Girder

First surface cracks appeared in the center portion of Segment 6 at an applied moment of 3420 kip-in. (386 kN•m). These were flexural cracks that extended up 13 in. (330 mm) from the bottom surface of the girder. As the applied load was increased, flexural cracks grew in size and number. In addition, diagonal web cracks appeared in the shear spans.

Throughout the first loading cycle, joint openings were largest at Joints 5-6 and 6-7. The bottom shear key on Segment 7 at Joint 6-7 broke off at an applied moment of 6070 kip-in. (686 kN•m). Upon further loading, a maximum applied moment of 6220 kip-in. (703 kN•m) was attained. At this point, an extensive and well distributed pattern of diagonal web cracks had formed in the shear spans, outside the constant moment region.

The specimen was completely unloaded. Then, anchor wedges for the top two strands were burned at both ends. Strand strains did not change during wedge burning.

Next, the second loading cycle was started. At an applied moment of 5800 kip-in. (655 kN•m), new diagonal web cracks were observed. Throughout the second loading cycle, joint openings were largest at Joints 5-6 and 6-7. The maximum applied moment leveled off at 6910 kip-in. (781 kN•m). Upon further loading, the load carrying capacity of the girder decreased as a flexural failure

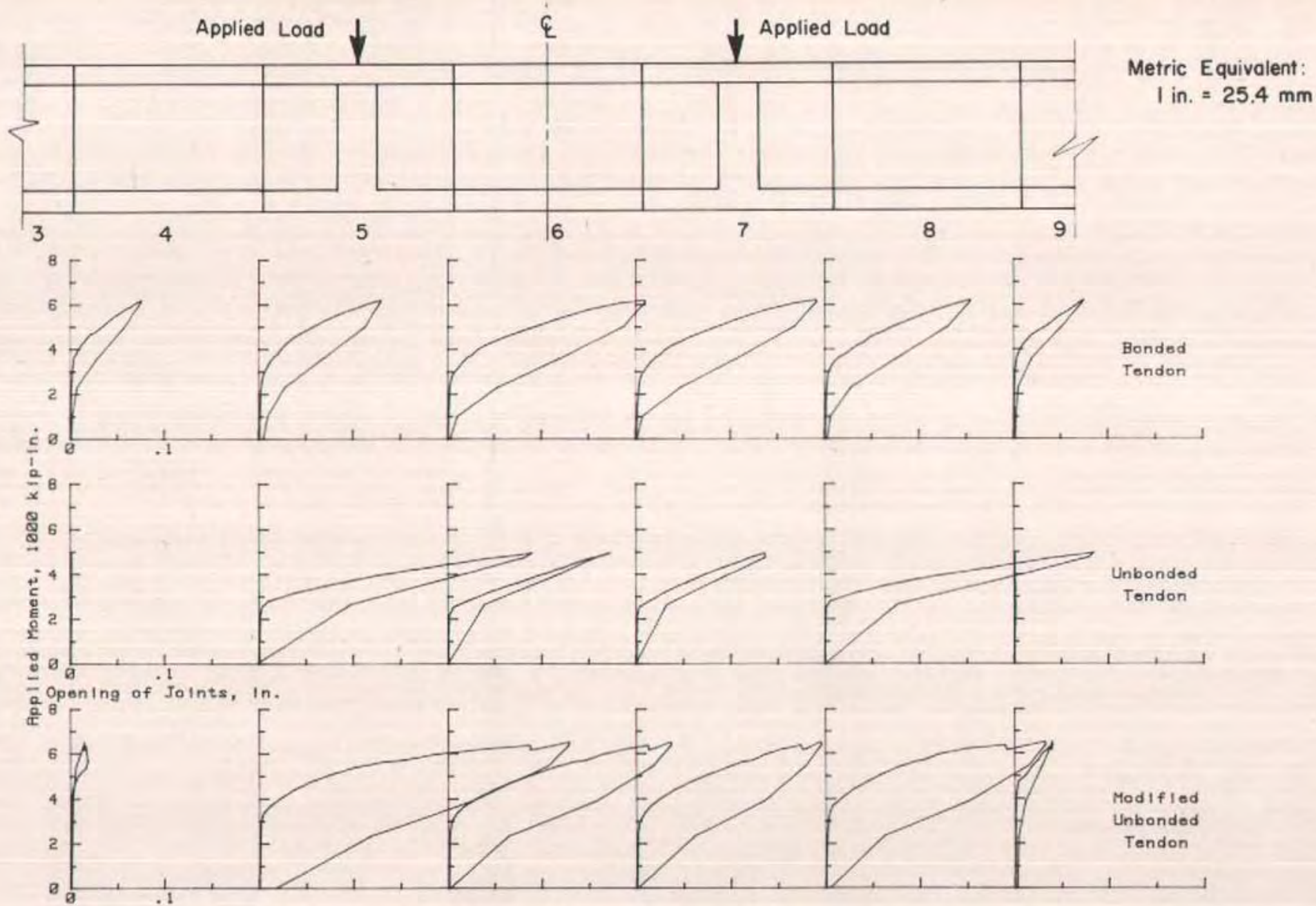

Fig. 9. Joint opening during first loading cycle.

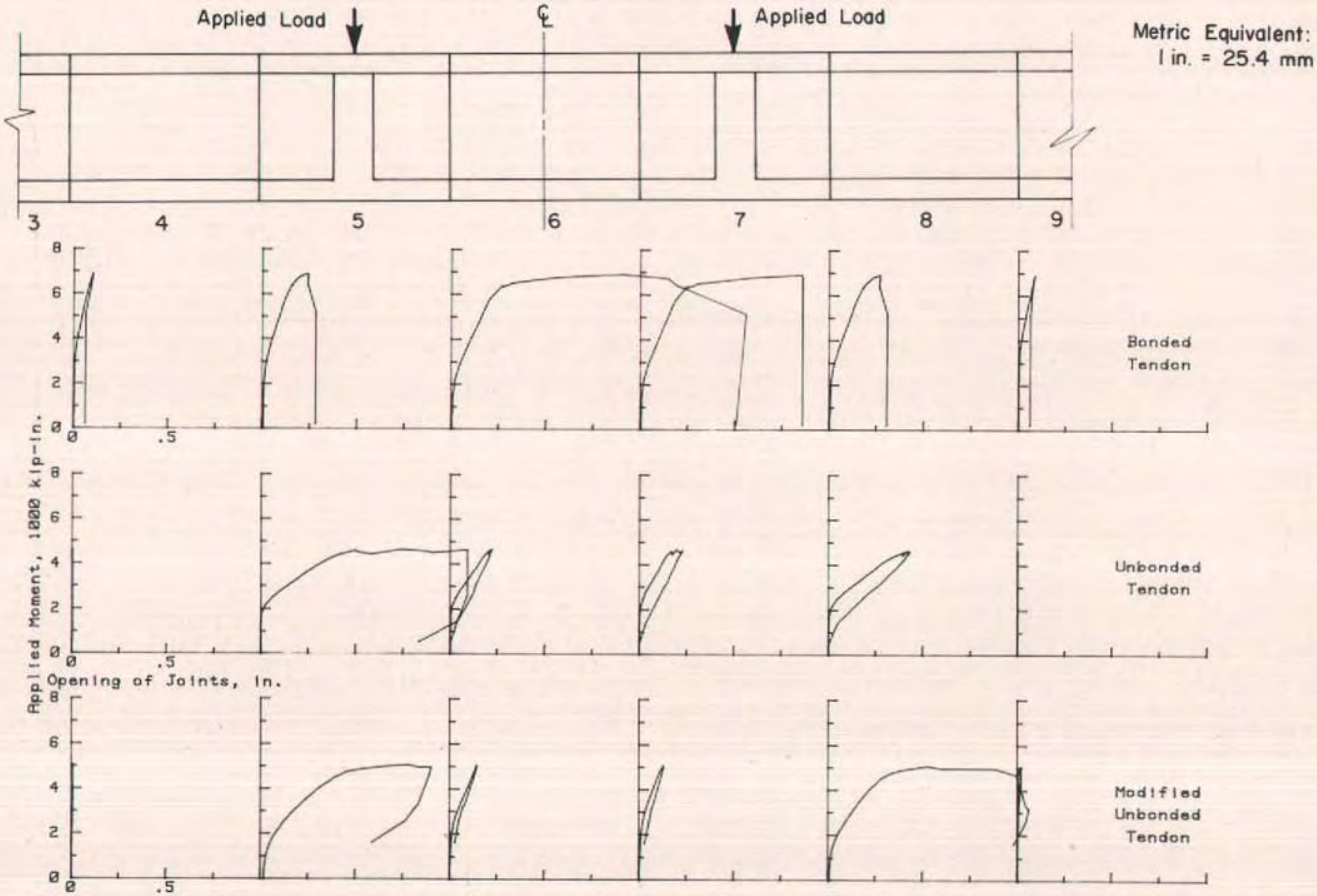

Fig. 10. Joint opening during second loading cycle.

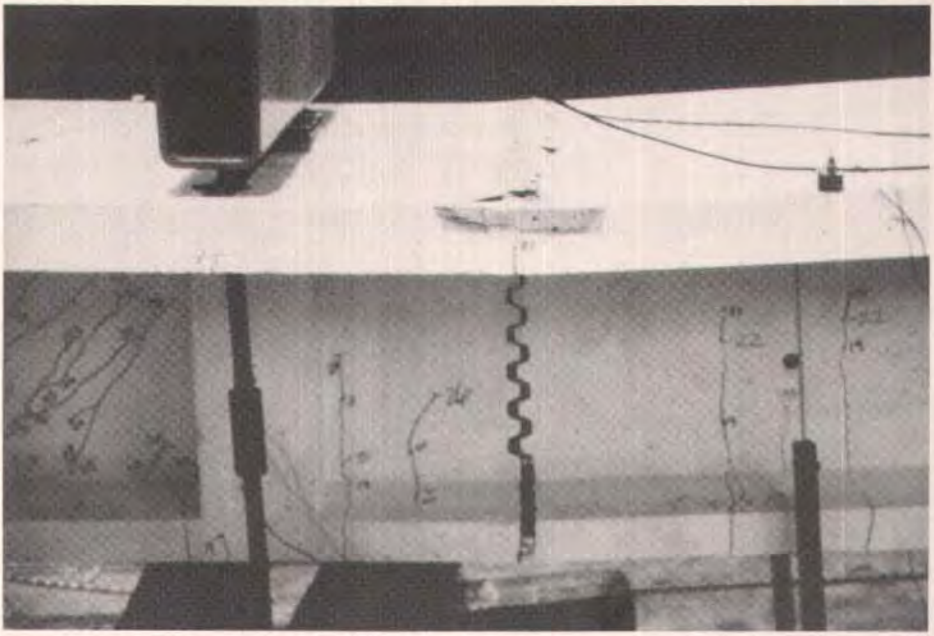

Fig. 11. Joint 5-6 of Bonded Tendon Girder after testing.

mechanism formed at Joint 5-6. Failure started with localized crushing of concrete along the top surface of the joint resulting in a load decrease. Then, several strands broke leading to a total loss of load carrying capacity. Fig. 11 is a photograph of the failure zone at Joint 5-6.

Unbonded Tendon Girder

First surface cracks appeared at an applied moment of 3170 kip-in. (358 kN•m). These were diagonal web cracks mainly concentrated at Joints 4-5 and 7-8. As the applied load was increased, diagonal web cracks grew in size and number. Throughout the first loading cycle, joint openings were largest at Joints 4-5 and 7-8. The maximum applied moment was 4980 kip-in. (563 kN•m). At this point, an extensive pattern of diagonal cracks had formed at Joints 4-5 and 7-8. At each of these two joints, cracks mainly formed on the side of the joint closer to the loading points. Few cracks occurred on the side of the joint closer to the support.

The specimen was completely unloaded. Then, anchor wedges for the top two strands were burned at both ends. Strand strains indicated a large drop in the stress of the two top strands in the draped portions of the strands. This stress drop was a result of releasing the anchors of the two top strands. Transfer length in the pier segments was inadequate for transfer of prestress. At midspan, no noticeable drop in strain due to burning of wedges was observed.

During the second loading cycle, new diagonal web cracks were observed at an applied moment of 2970 kip-in. (336 kN•m). Throughout the second loading cycle, joint openings were largest at Joints 4-5 and 7-8. At an applied moment of 4550 kip-in. (514 kN•m), two shear keys at Joint 4-5 broke off, causing a minor load drop.

Subsequently, load was increased gradually. The maximum applied moment attained was 4670 kip-in. (528

Fig. 12. Joint 4-5 of the Unbonded Tendon Girder after testing.

kN•m). Upon further loading, the load carrying capacity of the girder decreased as more individual shear keys broke off. Finally, a shear compression failure occurred in the top flange at Joint 4-5. The test was concluded. Fig. 12 is a photograph of the failure zone at Joint 4-5.

Modified Unbonded Tendon Girder

The first observed cracks were flexural and occurred in the second stage concrete cast. As the applied load was increased, flexural cracks in the second stage cast grew in size and number. In addition, diagonal web cracks appeared in the shear spans. Joint openings were largest at Joints 5-6 and 6-7 up to an applied moment of 5650 kip-in. (638 kN•m). Then, up to an applied moment of 6170 kip-in. (697 kN•m), joint openings were largest at Joints 4-5 and 6-7. Subsequently, joint openings were largest at Joints 4-5 and 7-8.

The maximum applied moment was 6530 kip-in. (738 kN•m). At this load level, an extensive pattern of diagonal cracks had formed in the shear spans. In one shear span, cracks were mainly concentrated in Segment 5 at Joint 4-5. Fewer cracks occurred in Segment 4. In the other shear span, cracks were well distributed and generally extended on both sides of Joint 7-8.

The specimen was completely unloaded. Then, anchor wedges for the top two strands were burned at both ends. Strand strains indicated a large drop in the stress of the top two strands. This stress drop was a result of releasing the anchors of the two top strands. Transfer length in the pier segments was inadequate for transfer of prestress. At midspan, no noticeable drop in strain due to burning of wedges was observed.

During the second loading cycle, at an applied moment of 3490 kip-in. (394 kN•m), new diagonal web cracks were observed. Throughout the second loading cycle, joint openings were largest at Joints 4-5 and 7-8. The maximum

Fig. 13. Joint 7-8 of the Modified Unbonded Tendon Girder after testing.

applied moment of 5080 kip-in. (574 kN•m) was 9 percent higher than that attained by the Unbonded Tendon Girder.

Following the peak load, two shear keys at Joint 4-5 broke off, causing a minor load drop. Upon further loading, more keys broke. Finally, shear keys broke off at Joint 7-8 and a shear compression failure occurred in the top flange at Joint 7-8 concluding the test. Fig. 13 is a photograph of the failure zone at Joint 7-8.

FLEXURAL STRENGTH ANALYSIS

Flexural strength analyses were performed for midspan sections of the test girders. Two analysis methods were used. In the first method, the classic bending theory of plane sections was used. The flexural strength was calculated through strain compatibility and equilibrium of internal forces, assuming tendons were bonded. In the second method, flexural strength was calculated from the equations given in the 1983 AASHTO Specifications[3] for bonded and unbonded tendons.

All analyses were based on measured material properties and effective prestress. The effective prestress was determined from measured strand strains. Since the segments were assembled with dry joints, concrete tensile strength was ignored. As the Unbonded Tendon and the Modified Unbonded Tendon Girders had identical strand layout, both analysis methods would predict similar flexural strength for the midspan sections. Results of the flexural strength analyses for the Bonded Tendon and Unbonded Tendon Girders are listed in Table 2.

Four cases were analyzed. Case 1 corresponds to the midspan section of the Bonded Tendon Girder with six strands. Cases 2 and 3 correspond to the midspan section of the Unbonded Tendon Girders with six strands. The difference between Cases 2 and 3 is the concrete compressive strength. This difference

Table 2. Calculated flexural strength.

Case number	Number of effective strands	Girder cross section	Concrete compressive strength (psi)	Calculated moment (kip-in.)	
				Computer analysis	AASHTO
1	All six strands	Bonded Tendon Girder	5810	7260	7320
2		Unbonded Tendon Girders	5810	7210*	4380
3			6400	7220*	4390
4	Four bottom strands only	Unbonded Tendon Girders	6400	4660*	2850

* Strands assumed bonded to concrete for analysis purposes.
Metric Equivalents: 1000 kip-in. = 113 kN•m; 1000 psi = 6.895 MPa.

allowed evaluation of the effect of minor variation in the concrete compressive strength on flexural strength. Concrete compressive strengths for Cases 2 and 3 corresponded to values measured on companion cylinders from the failure zone concrete of the Bonded Tendon Girder and Unbonded Tendon Girder, respectively. Case 4 corresponds to the Unbonded Tendon Girders with four bottom strands only.

After the first loading cycle, the anchorages for the top two strands were burned. Burning of the anchorages did not significantly affect the behavior of the Bonded Tendon Girder. For the Unbonded Tendon Girders, however, the contribution of the top two strands was drastically reduced after burning the anchorages. Therefore, Case 1 corresponds to the first and second loading cycles of the Bonded Tendon Girder. Case 3 corresponds to the first loading cycle of the Unbonded Tendon Girders, while Case 4 provides a lower bound flexural strength for the second loading cycle of the Unbonded Tendon and Modified Unbonded Tendon Girders.

Computer Analysis

A computer program was used to predict the flexural strength of the test girders. For the Unbonded Tendon Girders, the strands were external to the concrete cross section at midspan. However, for analysis purposes, the strands were assumed fully bonded to the concrete.

Cases 1 and 2 were compared to evaluate the effect of the varying tendon configurations, assuming fully bonded strands. The analysis shows that tendon locations used in Bonded and Unbonded Tendon Girders yielded similar flexural strengths. Therefore, the prestress eccentricity was the same for all three girders.

Cases 2 and 3 were compared to evaluate the effect of concrete compressive strength on flexural strength. The analysis showed that flexural strength was not sensitive to small variations in concrete compressive strength.

For the Bonded Tendon Girder, the maximum total moment attained during the second loading cycle, 7600 kip-in. (859 kN•m), exceeded the calculated strength in Case 1 by 5 percent. The

Unbonded Tendon and Modified Unbonded Tendon Girders were not loaded to destruction during the first loading cycle while all strands were fully effective. Nevertheless, the maximum total moment attained by the Modified Unbonded Tendon Girder reached the computed strength for bonded tendons.

For the Unbonded and Modified Unbonded Tendon Girders, the maximum total moment attained during the second loading cycle, 5360 and 5800 kip-in. (606 and 655 kN•m), exceeded the calculated strength in Case 4 by 15 and 24 percent, respectively. This excess capacity indicates that loss of bond at girder ends for the top two strands did not totally eliminate their contribution to the flexural strength at midspan.

Analysis by AASHTO Specifications

The AASHTO Specifications[3] allow a higher strength for bonded tendons compared to unbonded tendons. The average strand strength at ultimate is specified as follows:

For bonded tendons:

$$f_{su}^* = f_s' \left(1 - 0.5 \, \frac{p^* f_s'}{f_c'}\right)$$

For unbonded tendons:

$$f_{su}^* = f_{se} + 15{,}000$$

where

- f_{su}^* = average stress in prestressing steel at ultimate load, psi
- f_s' = ultimate strength of prestressing steel, psi
- p^* = A_s^*/bd, ratio of prestressing steel
- A_s^* = area of prestressing steel, sq in.
- b = width of flange of flanged member or width of rectangular member, in.
- d = distance from extreme compressive fiber to centroid of prestressing force, in.
- f_c' = compressive strength of concrete at 28 days, psi
- f_{se} = effective steel prestress after losses, psi

Therefore, the AASHTO Specifications recognize that members with unbonded tendons have a lower strength than identical members with bonded tendons.

For the Bonded Tendon Girder, the maximum total moment attained during the second loading cycle, 7600 kip-in. (859 kN•m), exceeded the strength predicted by the AASHTO equation (Case 1) by 4 percent. For the Unbonded Tendon Girder, the maximum total moment attained in the first loading cycle, 5670 kip-in. (641 kN•m), exceeded the predicted strength for unbonded tendons (Case 3) by 29 percent.

It is concluded that the AASHTO provisions for unbonded tendons significantly underestimated the flexural strength of the Unbonded Tendon Girder. Despite the loss of bond of the top two strands, the maximum total moment attained during the second loading cycle, 5360 kip-in. (606 kN•m), exceeded the strength predicted by AASHTO for six unbonded tendons.

The Modified Unbonded Tendon post-tensioning system is not addressed by the AASHTO Specifications.[3] The "Modified Unbonded Tendon" system tested was effectively a combination of bonded tendons in the center portion of the girder where all ducts were embedded in concrete and external tendons where the ducts were outside the girder cross section. For the tested condition, the Modified Unbonded Tendon Girder reached during the first loading cycle the strength predicted by AASHTO for bonded tendons.

Analysis by ACI Code

The ACI 318-83 Code[4] equation for unbonded tendons yields a flexural strength higher than that predicted by the AASHTO Specifications.[3] For the Unbonded Tendon Girders with six effective strands, the flexural strength predicted by ACI 318-83 was 5600 kip-in. (633 kN•m). This strength was reached by the Unbonded Tendon Girder during the first loading cycle, where the maximum moment attained was 5670 kip-in. (641 kN•m). It is concluded that the ACI Code equation for unbonded tendons can also be used to predict the flexural strength of girders with external tendons.

CONCLUSIONS AND OBSERVATIONS

Based on the test results, the following conclusions were drawn and observations noted:

1. Although the segments were dry jointed, the Bonded Tendon Girder attained the strength predicted by the classic bending theory for bonded tendons.

2. During the first loading cycle, the Modified Unbonded Tendon Girder reached the strength predicted by the classic bending theory for bonded tendons.

3. Despite the loss of bond of the two top strands, the Unbonded Tendon and the Modified Unbonded Tendon Girders exceeded the flexural strength predicted by the AASHTO Specifications for members with unbonded tendons.

4. During the first loading cycle, the Unbonded Tendon Girder reached the flexural strength predicted by ACI 318 for unbonded tendons.

5. The Unbonded Tendon Girder was the easiest to construct. The Modified Unbonded Tendon Girder ranked second in ease of construction.

6. During the first loading cycle, the Bonded Tendon and the Modified Unbonded Tendon Girders behaved similarly. The Bonded Tendon Girder exhibited a slightly larger deflection at comparable applied moments.

7. In the Bonded Tendon Girder, releasing of the anchor wedges of the two top strands did not significantly affect the behavior of the girder. Adequate development length for the two top strands was available at the ends of the Bonded Tendon Girder.

8. Releasing the anchor wedges of the two top strands in the Unbonded Tendon and the Modified Unbonded Tendon girders affected the behavior of these girders. Strains measured on these strands indicated loss of prestress and therefore loss of bond in the pier segments and draped portions of the Unbonded Tendon and the Modified Unbonded Tendon Girders.

9. During the loading cycle to destruction, the Modified Unbonded Tendon Girder exhibited 8 percent higher strength compared to the Unbonded Tendon Girder. The Bonded Tendon Girder, with virtually 59 percent more prestressing, exhibited 42 percent higher strength compared to the Unbonded Tendon Girder.

10. The Bonded Tendon Girder failed in a flexural mode where simultaneously concrete crushed in the compression zone and strands fractured in the tensile zone.

11. In the Unbonded Tendon and the Modified Unbonded Tendon Girders, where there was loss of prestress in the two top strands, joint openings were larger in the shear span. As a result, the shear keys broke progressively leading to loss of shear carrying capacity. Ultimately, shear compression failure occurred in the top flange at a joint where shear keys had broken.

DESIGN RECOMMENDATIONS

Based on the test results reported in the paper and the conclusions noted above, the following is recommended:

1. The tendons of segmental members with external grouted post-tensioning system can be conservatively designed by the 1983 AASHTO Specifications or ACI 318-83 Code equations for members with unbonded tendons.

2. The tendons of segmental members with internal grouted post-tensioning systems can be reliably designed by the classic bending theory for bonded tendons.

CLOSING REMARKS

Based on the test results, the external tendon system with the secondary concrete cast was incorporated in the final design. An addendum to the final design allowed contractor options for the prestressing system. The lowest bid was for a concrete option that included a combination of external (unbonded) and internal (bonded) tendons. This system proved to be the most economical solution for 1.5 miles (2.4 km) of bridges for Phase I of this three-phase project. The lowest bid for the steel alternate was 37 percent higher than that of the selected concrete alternate.

Further research on analytical procedures verified by these and other experiments on segmental concrete girders with external tendons is currently underway in France under the direction of Professor Michel Virlogeux and at the University of Texas at Austin under the direction of Professor John E. Breen. When completed, such analytical tools should yield more accurate and economical designs. Until such tools become available, however, external tendons can be conservatively designed by the AASHTO equation for flexural strength of members with unbonded tendons.

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by December 1, 1987.