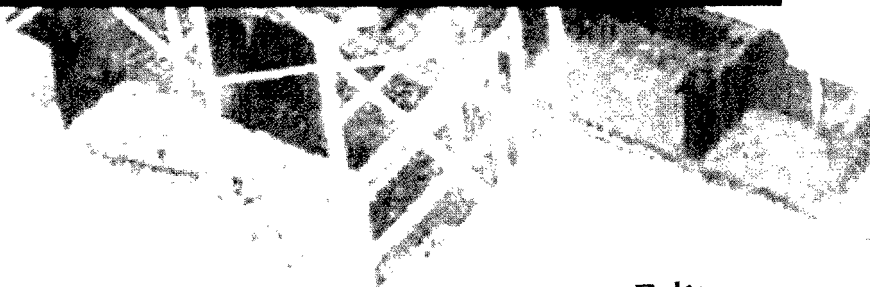


LARGE-SCALE STRUCTURAL TESTING



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Large-Scale Structural Testing

Editors

Mohsen A. Issa

Y. L. Mo



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Printed in the United States of America

Editorial production: Bonnie L. Schmick

Library of Congress catalog card number: 2003101441
ISBN: 0-87031-108-5

PREFACE

To date, most structural tests are based on small scale tests to verify the accuracy of analytical models. Small-scale tests can allow the mechanics (modes) of failure to be examined carefully at a fractional cost of full-scale testing. Full-scale tests render the realistic behavior of structures; however, they require large-scale testing facilities and enormous amount of manpower in addition to being very expensive to set up and run. Whether large or small in scale, testing of structures and structural components is deemed vital in predicting field performance.

Realizing the need for more experimental testing of concrete structures, Committee 444, Experimental Analysis of Concrete Structures, took the initiative to sponsor two technical sessions on "Large-Scale Structural Testing." This is part of the mission of Committee 444 to promote and facilitate application of experimental analysis for the evaluation and design of concrete structures. The two Technical Sessions were held at the ACI Spring Convention in Vancouver, British Columbia, Canada, in March-April 2003. This volume includes papers presented at the two technical sessions. Additional papers, which were not presented, are also published herein. All papers were peer reviewed, per ACI requirements, by experts within and outside the committee. These original reviewed and revised papers constitute this ACI Special Publication.

The two technical sessions were noted for their international flavor and the variety of topics presented. The topics included the following full-scale tests: fatigue testing of two pre-cracked AASHTO bridge girders, pseudo-dynamic and forced vibration tests of a two-story reinforced high-performance concrete building, lateral load tests of unbonded post-tensioned precast concrete walls, testing of Wacker drive viaduct, seismic jacketing of RC columns for enhanced axial load carrying performance, load tests of Canada Place pier, seismic performance of outrigger beam-wall connections, testing continuous reinforced concrete flat slabs, testing of seal slab/pile interface bond, experimental and analytical studies of RC framed shear walls, shear tests on beams cut from ASR affected bridge decks, flexural and shear strengthening of RC beams using carbon fiber-reinforced polymer laminates, and masonry infill structures. These papers should be of interest to practicing engineers, educators, and researchers in that they demonstrate the effective use of various facilities for concrete structures.

The editors and session organizers would like to thank the authors and presenters for submitting these papers and making this special publication possible. Their efforts are greatly appreciated. We would also like to thank the reviewers of the original manuscripts for their constructive comments and suggestions. Special thanks are due to the members of Committee 444 for their support and in particular to K. Nam Shiu, former Chair of Committee 444, for his support and encouragement when this venture was proposed. Special thanks are also due to the ACI staff for their great help in organizing the sessions and preparing this special publication.

Mohsen A. Issa
Y. L. Mo
Editors

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Fatigue Testing of Two Full-Size Pre-Cracked AASHTO Bridge Girders

by P. Zia, M. J. Kowalsky, G. C. Ellen, and S. E. Longo

Synopsis:

This paper presents the results of an investigation on the fatigue performance of two full-size pre-cracked prestressed concrete bridge girders. One AASHTO Type III girder and one AASHTO Type V girder were tested under 1,000,000 cycles of repeated service load intermingled with 2,500 cycles of repeated overload. The behavior of the girders was monitored after each 200,000 cycles of service load as well as each 500 cycles of overload. At the end of the fatigue tests, the girders were tested to failure to determine their ultimate strengths.

The test results demonstrated that the fatigue loadings had virtually no effect on the girder behavior. The girders showed no degradation in stiffness or strength after 1,002,500 cycles of fatigue loading. Both girders showed considerable ductility, and their ultimate loads and maximum deflections exceeded the predicted values.

Keywords: bridge; cracking; cyclic load; fatigue; fatigue strength; fatigue testing; girder; prestressed concrete; repeated load

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INTRODUCTION

During the production of large long-span prestressed concrete bridge girders in the pre-tensioning plant, there is an inherent production problem that often causes two or more transverse cracks within the middle third of the girder. The cracks would start at the top flange of the girder and extend downward through the web, and often into the bottom flange of the girder.

In a previous study by Zia and Caner (5), (6), it was determined that just before de-tensioning the large tensile force in the prestressing strands restricts the contraction of the concrete girders in their initial cooling period. This restrained thermal contraction causes tensile stresses in the concrete girders, large enough to exceed the tensile strength of the concrete at its early age, thus causing the transverse cracks. Upon de-tensioning, the cracks would be closed and become virtually invisible. Zia and Caner also observed that given time and moisture, the cracks would heal and the concrete would regain nearly its full compressive strength and most of its tensile strength.

However, the degree of concrete healing is uncertain, and bridge engineers have been concerned about the long-term behavior and strength of the girders impaired by the transverse cracks. Accordingly, this investigation was undertaken to assess the fatigue performance of two full-size pre-cracked AASHTO prestressed concrete girders.

RESEARCH SIGNIFICANCE

This research provides much needed data on fatigue performance of full-size prestressed concrete bridge girders, especially with pre-existing cracks. Such information is valuable to bridge designers when they consider the fatigue resistance of such girders. The research also demonstrates the validity of the AASHTO design requirement for stress limit under fatigue loading.

EXPERIMENTAL PROGRAM

Test Specimens

Two AASHTO girders were tested under fatigue loadings. One was a Type III girder of 19.95 m (65 ft. 5 ½ in.) long and the other was a Type V girder of 19.8 m (65 ft.) long. The Type III girder (Fig. 1) was prestressed with thirty-four 12.7 mm (1/2 in.) low-relaxation strands, each tensioned initially to 138 kN (31 kips). Twenty-two of the strands were straight, and the remaining 12 strands were draped with hold-down points at 1.85 m (6 ft. 1 in.) on each side of the mid-span. In the top flange of the girder, there were two additional straight strands initially tensioned to 13.3 kN (3 kips) each. These two strands were used primarily to support the stirrups and help control cracks, and were not considered as active prestressing elements.

The Type V girder (Fig. 2) was prestressed with thirty-six 12.7 mm (1/2 in.) low-relaxation strands, each with initial tension of 138 kN (31 kips). All the strands were straight with 32 strands located in the bottom flange and 4 strands located in the top flange ("B" Strands). Also located in the top flange were two additional straight strands ("A" Strands) initially tensioned to 4.45 kN (1 kip) to support stirrups and help control cracks. Again, these two strands were not considered as active prestressing elements.

Both girders showed transverse cracks when they were delivered to the laboratory. Figs. 1 and 2 show the cross-sections of the two girders. Other details on the cross-sectional properties and material characteristics of the two test specimens are described elsewhere by Zia, et al. (7). The compressive strengths of the concrete for the two girders are given in Table 1.

Test Procedure

Both girders were simply supported by elastomeric bearing pads and a single point load was applied at mid-span by a MTS actuator with a 1,984 kN (446 kips) fatigue capacity and 1,016 mm (40 in.) stroke. The span was 19.6 m (64 ft. 4 in.) for the Type III girder and 19.4 m (63 ft. 8 in.) for the Type V girder. The girders were tested without a cast-in-place deck.

Initially, each girder was loaded statically until flexural cracks were observed in the bottom flange of the girder, and the load-deflection data were recorded. Then the fatigue service load was applied to the girder for a total of 1,000,000 cycles. After each 200,000 cycles of service loading, a fatigue overload of 500 cycles was applied to the girder, for a total of 2,500 cycles of overload. Following these fatigue loadings, the girder was tested to failure for its ultimate strength. At the end of each segment of these fatigue loadings (service load as well as overload), a static test was conducted to obtain the load-deflection data in order to assess the behavior of the girder. In addition, the crack development at the various loading stages was also monitored.

The limits of each load cycle of the fatigue loadings are given in Table 2. For the Type III girder, the lower load limit would produce the same moment at the mid-span by a composite deck of 2.44 m (8 ft.) wide and 203 mm (8 in.) thick. Similarly, the lower load limit for the Type V girder would produce the same moment at the mid-span by a composite deck of 2.7 m (9 ft.) wide and 213 mm (8 1/2 in.) thick.

For both girders, the upper load limit of the fatigue service load would produce a nominal bottom fiber stress of $0.25\sqrt{f'_c}$ MPa ($3\sqrt{f'_c}$ psi) at the mid-span of the composite girder if a concrete deck had been cast on the test girder. The stress is the design value under service load used by NCDOT. Since the girders were tested without a cast-in-place deck, the actual nominal bottom fiber stress was $0.27\sqrt{f'_c}$ MPa ($3.24\sqrt{f'_c}$ psi) for the Type III test girder and $0.26\sqrt{f'_c}$ MPa ($3.15\sqrt{f'_c}$ psi) for the Type V test girder. So the actual nominal stress for the two test girders was 8% and 5%, respectively, higher than the design value.

Also for both girders, the upper load limit of the fatigue overload would represent the effect of over-weight vehicles allowed to use a bridge with special permit issued by NCDOT, which is based on 75% of the ultimate strength of the girder with a composite deck. Under this fatigue overload, the actual nominal bottom fiber stress was $1.6\sqrt{f'_c}$ MPa ($19.23\sqrt{f'_c}$ psi) for the Type III test girder and $0.94\sqrt{f'_c}$ MPa ($11.34\sqrt{f'_c}$ psi) for the Type V test girder. As expected, these stresses were significantly higher than the stresses under the fatigue service load.

ANALYTICAL STUDIES

Analytical studies were also conducted to model the behavior of the test girders by using two separate computer programs, one called *Cracked Beam* and the other *Response 2000*. The former was developed by Ellen (3) and Longo (4) using Microsoft Excel and the latter was developed by Bentz (2). The analytical results are presented below for comparisons with the test results. Other details on the analytical studies and the computer programs can be found in reference (7).

RESULTS

Tests for Initial Cracking

Figs. 3 and 4 show the selected load-deflection curves of the two girders after different loading cycles. It is noted in both figures that the ultimate load test included loading and unloading several times. From both figures, it can be seen that for the initial static test (zero cycle) that was conducted to determine the flexural cracking load, both girders behaved elastically before cracking. After the flexural cracking load was determined, the girders were unloaded and reloaded in order to determine the load at which the flexural crack reopens. Based on the slope of the load-deflection curve, the initial cracking load, and the load at crack reopening, the girder stiffness $E_c I$, the elastic modulus E_c and the flexural modulus f_r of the concrete, as well as the prestress loss for each girder were calculated. These results are summarized in Tables 3 and 4.

It can be seen from Table 4, the elastic modulus E_c determined from the slope of the load-deflection curve is quite close to the value predicted by the ACI Code using the 28-day concrete compressive strength. On the other hand, the flexural modulus f_r obtained from the test differs much more from the predicted value by the ACI Code.

Regarding the loss of prestress, the magnitude is quite small. Since the girders were tested not too long after they were cast, so the losses due to creep and shrinkage would be negligible. The values given in Table 4 would represent mostly the loss due to elastic shortening.

Fatigue Test

During the fatigue test under service loading, crack development was very limited since the upper limit of the service load cycle was lower than the maximum load applied to the girder during the initial static test. However, the first 500 cycles of overloading caused much more significant cracking because the upper limit of the overload cycle was substantially higher than the maximum load applied to the girder during the initial static test. For all subsequent service load and overload cycles, the effect on crack development was gradual and minimal.

At the end of the fatigue test of the Type III girder, cracks were observed in the lower part of the girder within the central region of 5.8 m (19 ft.). The maximum crack length was 546 mm (21.5 in.), extending well into the web of the girder. The maximum crack width under a load of 667 kN (150 kips) was 0.33 mm (0.013 in.) and the average crack spacing was 140 mm (5.5 in.) on the side of the girder.

For the Type V girder, cracks were observed within the central part of 4.12 m (13 ft.) of the girder at the end of the fatigue test. The maximum crack length was 1194 mm (47 in.) near the midspan. The maximum crack width was 0.64 mm (0.025 in.) under a load of 1,130 kN (254 kips) and the crack spacing ranged from 203 to 254 mm (8 to 10 in.)

As seen from Figs. 3 and 4, all the load-deflection curves for each test girder are virtually parallel in the elastic range, which indicates that there was no stiffness degradation after the girder was subjected to 1,000,000 cycles of service load and 2,500 cycles of overload. In addition, in each figure, the continuous shift of the load-deflection curves from the origin suggests that there was a gradual, but permanent reduction in the camber of the girder.

Ultimate Load Test

Under the ultimate load test, the load-deflection curves for both girders as shown in Figs. 3 and 4 indicate that initially each girder maintained its original stiffness. However, as more extensive cracking developed in the girder under increasing load, there was a gradual reduction in the girder stiffness.

For the Type III girder, as the ultimate load of 907 kN (204 kips) was approached, the girder deflection was 132 mm (5.19 in.). Since the girder was tested without a composite deck slab, its failure mode was sudden and explosive. Failure occurred primarily from crushing of concrete in the compression zone on one side of the steel loading plate. As soon as the compression zone was lost, the large force in the prestressing strands put the remaining concrete area in the web and the bottom flange under a very high compressive stress which, in turn, caused concrete crushing in those areas as shown in Fig. 5.

A similar behavior was observed for the Type V girder. As the ultimate load was reached at 1,680 kN (377 kips), the top flange failed in compression at about 0.91 m (3 ft.) from the midspan. Immediately following the crushing of the top flange, the web also crushed explosively. A view of the girder after failure is shown in Fig. 6.

The behavior and the strength of the two girders were analyzed by the two computer programs, *Cracked Beam* and *Response 2000*. The comparisons between the experimental and the analytical results are shown in Tables 5 to 7. Similarly, the experimental and predicted load-deflection curves are compared as shown in Figs. 7 and 8.

It is worth noting that, as shown in Table 6, *Cracked Beam* predicted slightly higher stress in the bottom layer of strands than *Response 2000*. This is because the tensile stress in concrete was neglected in *Cracked Beam* analysis, whereas it was considered effective in the analysis by *Response 2000*. Therefore using *Cracked Beam* would be more conservative.

Also from Table 6, it can be seen that based on *Cracked Beam* the stress range in the bottom layer of strands for the Type III girder was 134 MPa (208.2 ksi – 188.7 ksi = 19.5 ksi) under the fatigue service load, and 202 MPa (218 ksi – 188.7 ksi = 29.3 ksi) under the fatigue overload. Similarly, the stress range in the bottom layer of strands for the Type V girder was 97 MPa (197 ksi – 183 ksi = 14 ksi) under the fatigue service load, and 269 MPa (222 ksi – 183 ksi = 39 ksi) under the fatigue overload.

According to the provision (Section 5.5.3.3) of the AASHTO LRFD Bridge Design Specifications (1), the limit on the stress range in straight strands subjected to fatigue loading is 124 MPa (18 ksi). Therefore, the two test girders were subjected to similar stress ranges under the fatigue service load, but with substantially higher stress ranges under the fatigue overload.

CONCLUSIONS

1. After 1,000,000 cycles of fatigue service load and 2,500 cycles of fatigue overload, both girders showed no degradation of stiffness and strength. No prestressing strand in either girder showed any signs of fatigue or failure. The measured and calculated deflections demonstrated that the ductility of the girders was not affected by the fatigue loadings.
2. For both girders, cracking and permanent deflection progressively increased with each segment of 500 cycles of fatigue overloading.

3. Based on the initial cracking load from test, the calculated values of prestress loss, the modulus of elasticity of concrete, and the flexural modulus of concrete were reasonably accurate.

4. The analytical results from both computer programs were sufficiently accurate in predicting the structural performance of the girders. In general, the predictions made by *Cracked Beam* were more conservative than the predictions made by *Response 2000*.

5. The research demonstrated that the limit of 124 MPa (18 ksi) on the stress range in strands subjected to fatigue loading as specified by AASHTO is a suitable design criterion.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the support of the North Carolina Department of Transportation and the Federal Highway Administration for sponsoring this investigation. They are also indebted to Bayshore Concrete Products Corporation, Cape Charles, Virginia who donated the prestressed concrete test girders.

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Table 1 Compressive Strength of Concrete

Test Specimen	Compressive Strength (psi)		
	1 day	7 day	28 day
Type III Girder	5,476	6,962	7,698
Type V Girder	4,572	7,293	9,439

Note: 1 MPa = 145 psi or 1 ksi = 6.895 MPa

Table 2 Load Limits of Fatigue Loadings

Test Specimen	Fatigue Service Load (kips)	Fatigue Overload (kips)
Type III Girder	26 to 107	26 to 152
Type V Girder	30 to 186	30 to 254

Note: 1 kip = 4.448 kN

Table 3 Girder Stiffness, Cracking Load, and Load at Crack Reopening

Test Specimen	Girder Stiffness $E_c I$ (in 10^6 kips-in ²)	Cracking Load (kips)	Load at Crack Reopening (kips)
Type III	731	134	114
Type V	2,695	208	140

Note: 1 kips-in² = 2,870 kN-mm² 1 kip = 4.448 kN

Table 4 E_c , f_r , and Prestress Loss

Test Specimen	E_c (in 10^3 ksi)		f_r (in psi)		Prestress Loss
	Test	ACI	Test	ACI	
Type III	5.83	5.39	842	658	9%
Type V	5.18	5.93	526	729	10.4%

Note: 1,000 ksi = 6.895 GPa or 1 MPa = 145 ksi

Table 5 Comparisons for Cracking and Ultimate Loads

Method	Type III		Type V	
	Cracking (kips)	Ultimate (kips)	Cracking (kips)	Ultimate (kips)
Experimental	125	204	204	377
<i>Cracked Beam Analysis</i>	121	185	224	346
<i>Response 2000 – Sectional</i>	129	189	212	307
<i>Response 2000 – Full Member</i>	149	191	204	333

Note: 1 kip = 4.448 kN

Table 6 Comparisons of Stresses in Strands

Method	Stress in Bottom Layer of Strands (ksi)			
	Minimum Fatigue Load	Fatigue Service Load	Fatigue Overload	Ultimate Load
Type III Girder				
<i>Cracked Beam</i>	188.7	208.2*	218.0	258.0
<i>Response 2000</i>	171.0	183.9	186.3	240.6
Type V Girder				
<i>Cracked Beam</i>	183	197	222	--
<i>Response 2000</i>	181	190	204	--

Note: 1 ksi = 6.895 MPa

* Based on cracked section analysis. For uncracked section, the stress is 202.4 ksi.

Table 7 Comparisons of Camber and Deflection at Failure for Type III Girder

Method	Initial Camber (in.)	Deflection at Failure (in.)
<i>Experimental</i>	1.16	5.19
<i>Response – Full Member</i>	1.11	4.43

Note: 1 in. = 25.4 mm

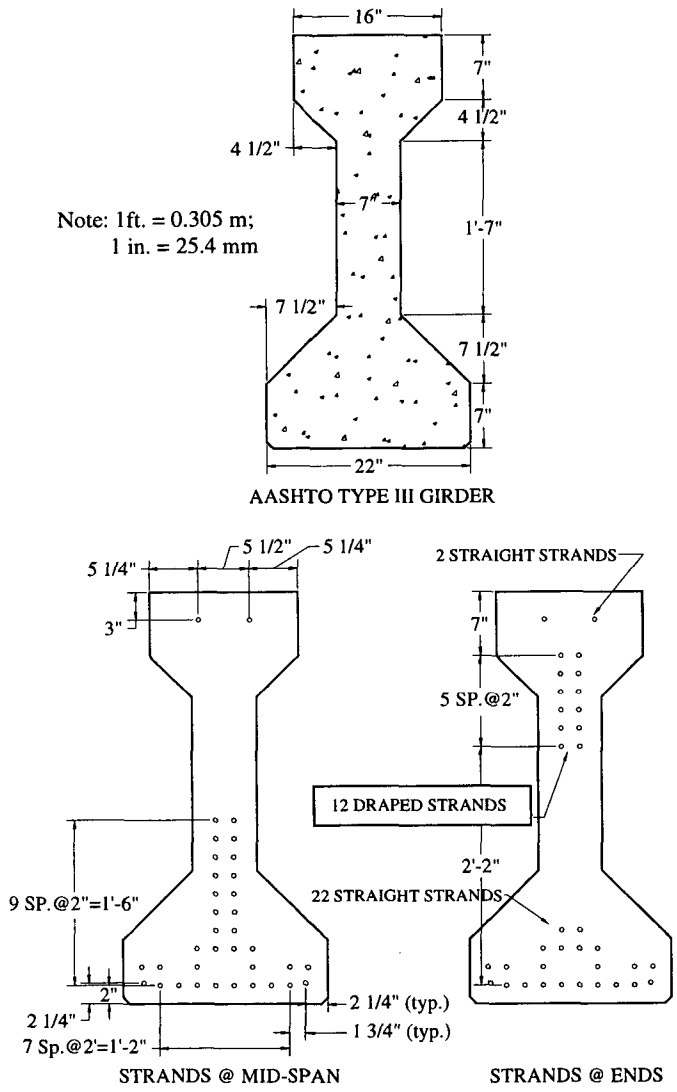


Figure 1 Cross-sections of AASHTO Type III Girder

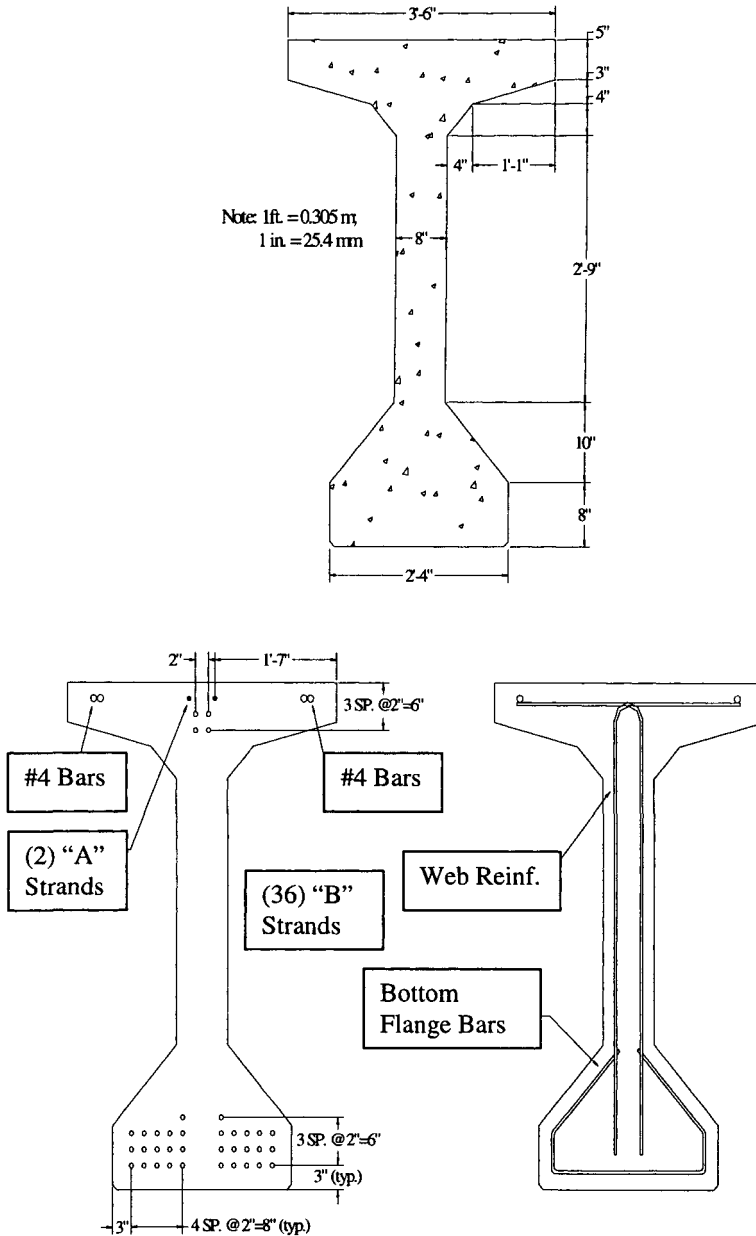


Figure 2 Cross-sections of AASHTO Type V Girder

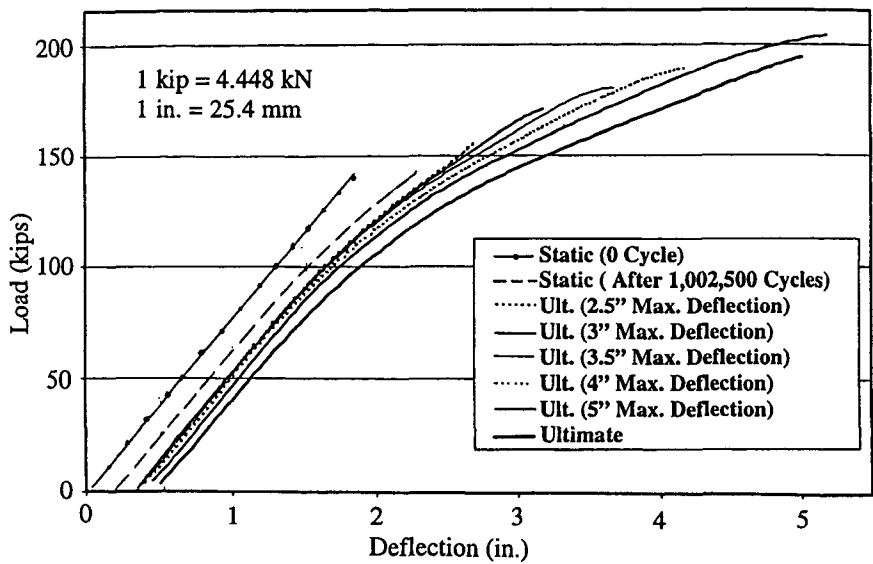


Figure 3 Load-Deflection Curves for Type III Girder

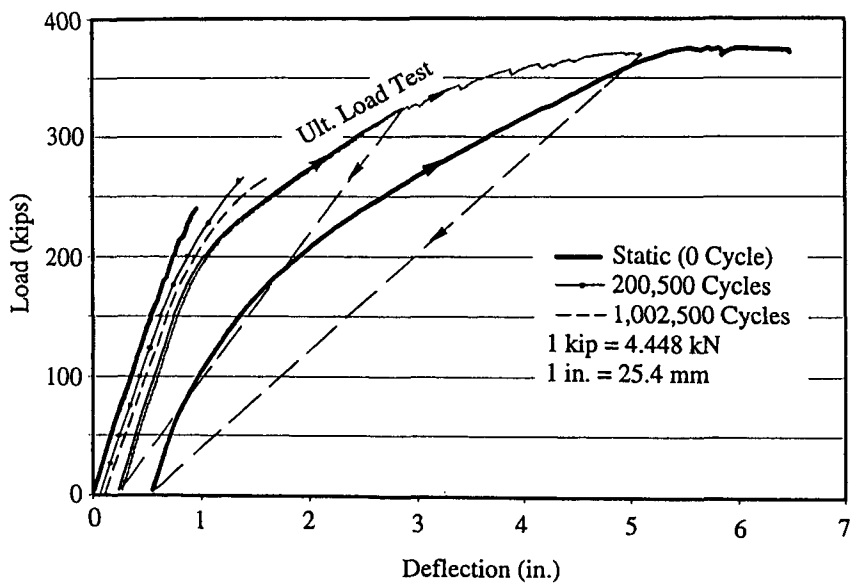


Figure 4 Load-Deflection Curves for Type V Girder

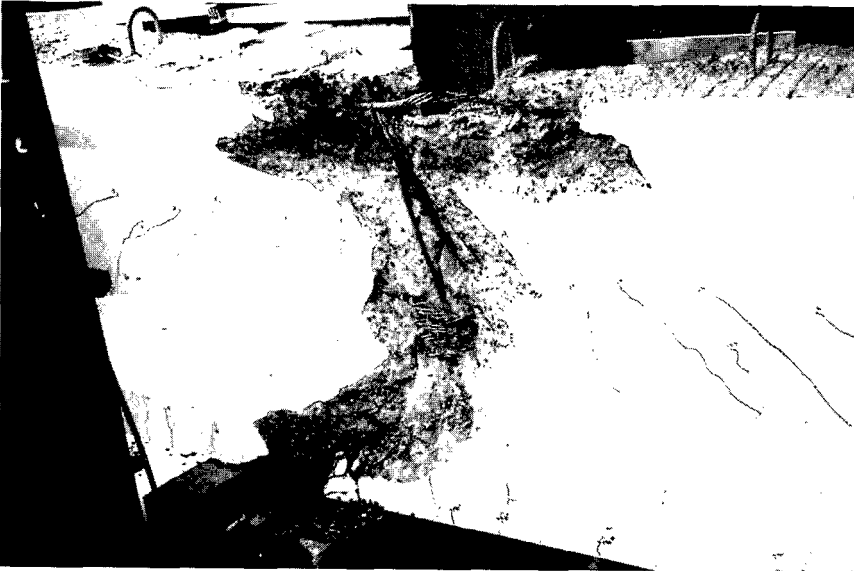


Figure 5 A View of Type III Girder after Failure



Figure 6 A View of Type V Girder after Failure

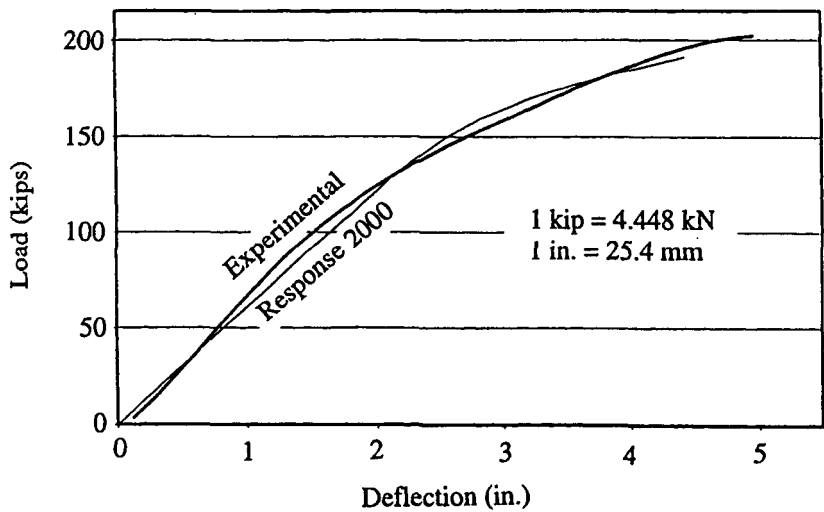


Figure 7 Experimental and Predicted Load-Deflection Curves for Type III Girder

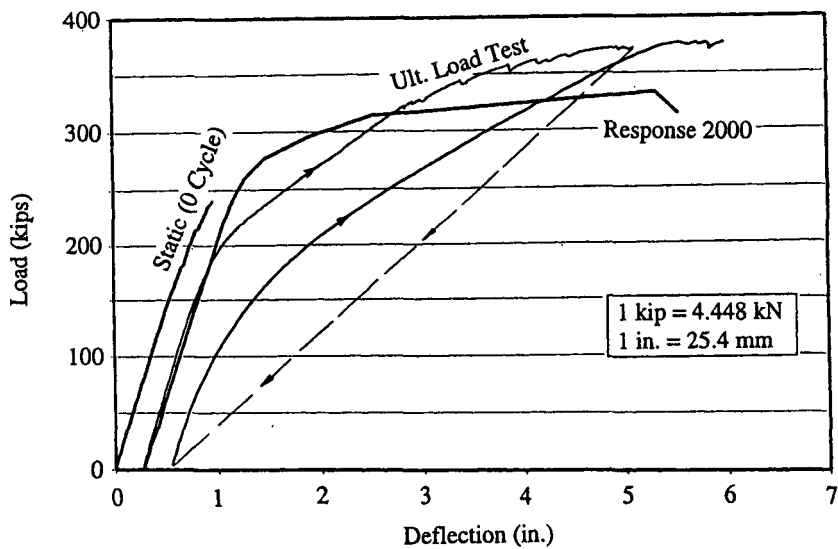


Figure 8 Experimental and Predicted Load-Deflection Curves for Type V Girder

Full-Scale Testing of Wacker Drive Viaduct Prototype

by J. Pearson, G. Klein, M. A. Issa, and S. Kaderbeck

Synopsis:

Wacker Drive in Chicago is a double-deck viaduct constructed along the Chicago River. Because of severe moisture-driven deterioration, the section of the elevated road between Randolph Street and Michigan Avenue are being reconstructed using a post-tensioned slab produced of high-performance concrete topped with a replaceable latex-modified concrete overlay.

In order to verify the adequacy of the roadway design, a full scale prototype was built and a test program was executed to evaluate the structural performance of the superstructure and its elements. Specifically, the following structural performance characteristics were evaluated:

- Positive and negative flexural strength
- Shear strength
- Live load stress range in concrete and post tensioning tendons
- Cracking at service load, overload and ultimate load
- Deflection at service load, overload and ultimate load
- Overlay bond strength and interaction between overlay and substrate
- Dynamic response

This paper describes the prototype construction, analysis and testing.

Keywords: high-performance concrete; hydraulic ram; latex-modified concrete; load frame; post-tensioning strand

AUTHOR'S BIOGRAPHIES

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INTRODUCTION

The Wacker Drive viaduct is a double-deck viaduct constructed along the Chicago River. Due to severe moisture-driven deterioration, the section of the drive between Randolph Street and Michigan Avenue is being reconstructed, along with the South Water Street viaduct between Wabash and Michigan Avenues.

The original design called for segmental construction using match-cast precast segments, each measuring roughly 8 feet by 50 feet. Two sets of slabs consisting

of the precast segments were to be constructed, one on each side of the roadway with a central cast-in-place closure pour. Cast-in-place fascia and sidewalk sections would then be constructed on each side of the viaduct. Finally, the design called for a latex-modified concrete overlay on the roadway surface.

The low bidder for the project proposed an all cast-in-place high performance concrete superstructure. Except for the obvious details that apply only to precast systems, the testing described herein generally applies to a cast-in-place system.

The prototype test program was performed to evaluate structural performance. In addition, the prototype allowed for measurement of dynamic response, evaluation of fabrication procedures, and evaluation of overlay replacement. With respect to structural performance, the prototype-testing program allowed for the evaluation of the superstructure elements, including precast segments, segment joints, cast-in-place closure strip, overlay and walkway.

Specifically, the following structural performance characteristics were evaluated:

- Positive and negative flexural strength
- Shear strength
- Live load stress range in concrete and post tensioning tendons
- Cracking and deflection at service load, overload and ultimate load
- Overlay bond strength and interaction between overlay and segments
- Closure strip performance
- Joint separation and faulting
- Dynamic response

In order to accomplish these objectives, the following was performed:

- Prototype design, details and specifications
- Structural precast segment fabrication
- Prototype construction
- Design and construction of load frame
- Test instrumentation
- Pretest evaluations
- Measurement of dynamic response
- Service and ultimate load testing
- Structural analyses
- Data analysis and reporting
- Trial overlay removal and replacement

This paper deals mainly with prototype design and construction, load frame design and construction, test instrumentation and load test.

PROTOTYPE DESCRIPTION

The prototype model simulated a portion of the proposed Wacker Drive Viaduct that includes an end span from an expansion joint to the first interior bearing supports. This model included the sidewalk and the outer $2\frac{1}{2}$ lanes of the span. Figure 1 shows a plan view of the general layout of the prototype deck assemblage. The sidewalk extended 3 feet beyond the edge of the segments so that the total width of the prototype deck system was 57 feet 3 inches. The total length of the prototype deck was 44 feet 9 inches.

The prototype was comprised of six precast segments and one transverse closure strip. Although not needed for construction of the prototype, a closure strip was included to simulate planned closure strips in the actual structure. Typical segments had a depth of 13 inches and a rib depth of 24 inches, except for the two end segments, which had extra details. The width of each precast segment was 54 feet 3 inches, which included a typical 32 feet 6 inch span of the actual bridge, and a 14 feet 9 inch cantilever representing one half of the middle span, while the length of the segments in the longitudinal direction varied. In order to simulate the behavior of the bridge deck system, a segment cantilevered beyond the interior transverse bearings, and the segment width was extended 9 feet beyond the interior longitudinal bearings for negative bending moment tests.

As shown in Figure 1, the segments varied in length from 6 feet $7\frac{9}{16}$ inches to 8 feet $1\frac{1}{2}$ inches. The slab depth was 13 inches and the ribs were 24 inches deep and 48 inches wide, except at the end where greater depths were needed for post-tensioning anchorage.

Each segment was post-tensioned using four tendons in the transverse direction, of which two were post-tensioned for handling and erection prior to transferring the segment from the casting bed to the test frame. The longitudinal post-tensioning was performed at the transverse edge of Segment 6 and consisted of 38 tendons.

PROTOTYPE CONSTRUCTION

Forms. Polyurethane-coated plywood forms were used for casting the segments. A continuous form was used for the base on which three segments could be accommodated at one time. A shear key form was used on the match-cast side of the segment with the geometrical configuration of the keys along the width of the slab and ribs (Figure 2). The shear key form was stripped one day after casting. This form was moved to the required location along the bed for casting the adjacent segment. The adjacent segment was match cast against the hardened concrete surface.

Reinforcement. Grade 60 steel conforming to AASHTO M31 was used for flexural and bursting reinforcement. All reinforcing bars were epoxy coated, except the spiral bursting steel at the longitudinal post-tensioning anchorages. Epoxy-coated spiral reinforcing was not available at the time of construction. Transverse and longitudinal steel consisted of #4, #5, #6 and #7 bars. Straight and bent epoxy-coated bars were used throughout the deck (Figure 3).

Top and bottom reinforcement mats were assembled by tying transverse and longitudinal bars and by using spacers and reinforcement seats. Bent bars were used as ties in the rib cross-section and at anchorage zones for tendon confinement. Segments S2 through S5 had similar reinforcing plans, while segments S1 and S6 required different arrangements since these two segments accommodated the post-tensioning bulkheads. Plan and section views of Segments S1, S2 through S5, and S6 are shown in Figures 4 through 6.

Post-tensioning tendons and bars. Low relaxation Grade 270 ksi strands were used for the longitudinal and transverse post-tensioning tendons. All tendons consisted of 7-wire strands conforming to AASHTO M203.

Transverse post-tensioning was achieved using four 0.6-inch and five 0.6-inch tendons profiled to match the longitudinal bending moment deviations. Transverse tendons were designated T1 through T4. Tendons T1, T2, T3, and T4 of segments S2 through S5, and tendons T1 and T2 of segments S1 and S6 consisted of four 0.6-inch strands with the same profile. Tendons T4 and T5 of segments S1 and S6 were five 0.6-inch strands with a different profile. Tendons T1 and T2 of each segment were post-tensioned on the precast bed prior to transferring of the segment to the test frame, while the rest of the tendons were post-tensioned after assemblage and segment bonding. Jacking forces for these tendons were applied at the cantilevering end of the segments.

Longitudinal post-tensioning was achieved using five 0.6-inch and nine 0.6-inch tendons. A total of 26 0.6-inch tendons were along the slab thickness, and 12 0.6-inch tendons were contained in the rib profiles, six in each rib. Jacking forces were applied at the expansion joint along the outer edge of segment S6.

Post-tensioning for epoxy jointing process was achieved using Grade 150 deformed dywidag bars conforming to ASTM A722. Three 1½-inch dywidag bars were used in the longitudinal direction to provide the required confining pressure of 50 psi for the epoxy jointing (segment bonding) process of each match cast segment. These bars were retained in the segments to serve as a portion of the permanent post-tensioning for the deck system.

Tendon ducts. Metal sheath ducts were used for the longitudinal tendons and post-tensioning bars. These 3.15 and 2-inch diameter metal ducts were cut to the

length of the segment, and positioned precisely at the appropriate location for each tendon prior to casting concrete. Care was taken while placing these ducts to eliminate misalignments with adjacent segments. The transverse tendon profiles were accomplished using 1-inch x 4-inch flat polyethylene tubes curved to match the tendon deviations (Figure 7).

The post-tensioning ducts were tied to the bottom and top mat of steel every 2 feet to maintain the appropriate design profile. The post-tensioning strands were inserted into the ducts after the concrete was cast and cured.

Concrete material and placement. High performance concrete (HPC) was used for the precast segments and closure strip. The HPC concrete mixture consisted of Type III Portland cement, class C fly ash, silica fume, ground granulated blast furnace slag, $\frac{3}{4}$ -inch maximum aggregate size, natural siliceous sand, and an air entraining agent. Although the project specification called for Type I or II portland cement, Type III was permitted for the prototype because it was the only cement available in the batching facility where the prototype was constructed.

Segments were cast starting at segment S6 and proceeding to segments S5, and S4 in the first casting cycle. The second casting cycle consisted of casting segment S1 and proceeding to segments S2 and S3. Reinforcing steel and tendon ducts were placed in the form and the shear key fixture was attached to the sides of the form to constitute the geometry of each segment. Concrete was then cast and vibrated to assure uniform distribution between the reinforcing steel and ducts, and the surface was then finished and broomed to produce rough grooves (Figure 8). A total of 110 cubic yards of concrete was cast for the segment construction.

The match-cast surfaces of the concrete segments were coated with a form-release agent consisting mainly of soap to facilitate separation of the segments after concrete hardening. This agent has the potential to act as a bond-breaker and hence it had to be thoroughly cleaned. The match-cast ends were sandblasted to remove any coating material and debris and to remove minor surface irregularities. Air pressure was used to blow clinging dust particles to assure clean surfaces prior to application of epoxy for segment joining.

Once the concrete achieved the desired strength, post-tensioning tendons were inserted into the tendon ducts. Each segment was post-tensioned for handling prior to transferring them to the storage yard. One transverse duct was damaged during construction and discovered during tendon installation. The damage prevented the tendon from being installed. The duct was exposed by chipping the concrete cover at the blockage. It was discovered that the duct had deflected down; as if stepped on, causing a drastic change that the tendons could not

navigate. This blockage was corrected by removing enough surrounding concrete so that the tendons could pass. The concrete removal area was filled with grout after the tendons were tensioned.

LOADING SYSTEM

Load frame. The load frame was fabricated during the time the segments were being constructed. The load frame underlying the structure consisted of two main transverse plate girders perpendicular to a series of 33- and 36-inch wide-flange beams. The plate girders were designed by WJE and built especially for the load test. The beams were borrowed from CDOT and the Chicago Transit Authority. A sketch of the load frame is shown in Figure 9. The top of the plate girders was approximately 12 feet above the ground.

The wide flange beams were welded to the underside and spanned between the plate girders. The wide flange beams were grouped in pairs. Steel channels bolted back-to-back were positioned perpendicular to the paired wide flange beams. A gap between the wide flange beams and channels allowed the passage of dywidag bars, which were used for loading the prototype.

Loading System. Load was applied using 40, 60-ton hollow core hydraulic jacks, divided into eight zones. The division of jacks into zones allowed for variation in load patterns. The loading zones are shown in Figure 10. Each of these loading zones was controlled using one pump connected to the hydraulic jacks with hoses, and several manifolds. Load cells and pressure gages were used in each zone to monitor the application of loads.

The load was applied using the hydraulic jacks and dywidag bars. Cast into the segments were PVC pipes slightly larger in diameter than the dywidag bars. The PVC pipes were located during segment fabrication at the desired loading points. The dywidag bars passed through the hydraulic rams, PVC pipe, wide flange beams, and steel channels. The dywidag bars were held in position at the top and bottom ends with plate washers and nuts.

The test loads were applied by operating the hydraulic rams. As the rams extended, they would react against the dywidag bars which in turn reacted against the steel frame work and prototype. This system essentially squeezed the prototype and steel frame together.

SEGMENT ERECTION

Epoxy jointing. Segments were joined by bonding the match cast segments with epoxy. After the desired compressive strength was reached, the segments were transferred to the test frame where they were assembled. Segments S6 and S2 were lowered onto the bearing pad supports along the plate girders using a handling crane, while segments S5, S4, S3, and S1 were placed on temporary hydraulic jack supports (Figure 11). Segment S6 was supported by bearing supports A2 and B2. Segment S2 was supported by bearing supports A1 and B1.

The jointing procedure involved applying epoxy in an approximate $\frac{1}{16}$ -inch thick layer on the adjacent sides of segments S6 and S5. Care was taken while applying the epoxy layer leaving a half inch clear boundary around tendon ducts to prevent blockage.

Segment S5 was then moved toward S6 by the handling crane. Segment S5 was post-tensioned against segment S6 using dywidag bars (Figure 12). Segment S4 was then post-tensioned against segment S5. Segments S1, S2 and S3 were bonded in a similar manner.

A space between segments S3 and S4 remained that measured 1-ft. 1 1/2-in. wide. This space was filled with HPC and termed the closure strip. Once the closure strip concrete achieved the desired compressive strength, post-tensioning of the prototype in the longitudinal direction was begun.

Post-tensioning. Longitudinal and the unstressed transverse tendons were divided into several groups and post-tensioning was performed in a sequence to achieve the desired stress in the concrete.

Upon completing the post-tensioning of the deck and rib longitudinal tendons, lift-off tests were conducted. Lift-off tests were conducted to determine the load change in a particular group of tendons after all tendons were tensioned. Tendons that had a lift-off pressure that was lower than 5 percent of the design pressure or more were retensioned to within ± 5 percent of the design pressure.

Tendon duct grouting. After tendon tensioning was completed, the tendon ducts were grouted with a water/cement/expansive agent mixture (Figure 13). The grout was pumped into one end of a duct. Vent tubes were capped as the grout discharged until grout discharged at the opposite end of the duct. Grout was allowed to flow for a short time with all of the vent tubes capped and then the end was capped.

Sidewalk construction. The deck surface was prepared for the sidewalk and the latex-modified concrete by sandblasting and water blasting. The sandblasting removed any laitance and loose material to provide a good bonding surface. The water blasting was performed 24 hours before the placement of concrete to

remove loose sandblasting sand and create a saturated-surface dry condition of the deck. Forming for the sidewalk took place after post-tensioning was completed.

Overlay. A latex-modified concrete (LMC) overlay was cast on top of the segments after they were post-tensioned. The LMC is the wearing surface of the actual structure. The same mix design was used for the actual structure as was used for the prototype.

Following sandblasting, a water/cement/sand mixture was broomed into the blasted surface of the precast segments in preparation for applying the latex-modified concrete layer. The application of this layer is a standard practice that facilitates bonding of the overlay to the hardened concrete surface. The overlay mix was then placed and struck off to a thickness of 2¼ inches. The overlay was struck off using a vibrating screed. Wet burlap was dragged across the overlay to produce a roughened finished surface (Figure 14).

INSTRUMENTATION

Primary instrumentation. The primary instrumentation system included the following electronic transducers:

- 12 cable extension transducers for measurement of deflection
- 9 linearly varying displacement transformers (LVDT's) for measuring displacement at bearings
- 2 LVDT's for measuring joint segment openings in the negative moment regions
- 3 LVDT's for measuring joint segment openings in the positive moment regions
- 3 LVDT's for measurement of joint faulting (out of plane movement)
- 12 internal concrete strain gages
- 8 strain gages installed on the post-tensioning tendons
- 5 load cells for measurement of applied load

Figure 15 shows the primary instrumentation plan. A data acquisition system was used to collect all measured strains, deflections, and load cell readings. In addition, mechanical strain gage measurement points were installed on the top of the deck. Strain measurement using the mechanical gage is shown in Figure 16. These strains were recorded by hand.

Supplemental instrumentation. Supplemental instrumentation was installed and consisted of 39 surface strain gages and 3 LVDT's. A separate data acquisition system was used for these instruments and operated by personnel from the

University of Illinois-Chicago. The distribution of strain was measured at all critical locations using strain gages mounted along the transverse and longitudinal directions on the top and bottom surfaces of the deck slab.

TEST PROCEDURE AND PERFORMANCE CRITERIA

Procedure. A pretest evaluation was conducted prior to the load test. This evaluation was performed to determine the condition of the prototype before any load application. Any conditions found (i.e., shrinkage cracks, overlay debonding) were documented before the load test was performed. After the pretest evaluation, a modal analysis was performed to determine the mode shapes and frequencies. This was done to establish a baseline dynamic response for comparison to an analytical model. Following the pretest inspection and measurement of dynamic response, the prototype was tested for the following load conditions:

Service load tests

- Positive moment
- Negative moment
- Cyclic negative moment

Overload tests

- Positive moment
- Negative moment

Ultimate load tests

- Simulating moment envelopes
- Longitudinal positive moment
- Load at expansion joint

The service load pattern simulated the design load: (HS20 + 30 percent impact). The overload level was 150 percent of service load.

The ultimate load test was further subdivided into three tests. First, test loads were applied to simulate stresses in proportion to the longitudinal and transverse moment envelopes. After a negative moment failure occurred in Column B1, the second ultimate load test was carried to maximize longitudinal positive moment in the main span. Finally, the structure was loaded along the expansion joint by simultaneously loading the jacks in the two rows paralleling the joint. Table 1 lists the tests performed, the test methods, and the performance criteria.

Table 1 - Test methods and performance criteria

Phase	Test	Method	Performance Criteria
<i>Pretest evaluation</i>	Tendon force	Lift off	$\pm 5\%$ of design jacking force after adjustment for expected seating losses
	Crack survey	Visual with comparator	No cracking in segments, sidewalk and overlay
	Overlay delamination	Chain drag	No delamination
	Overlay bond	Tensile pullout after partial depth coring per ACI 503R	150 psi min.
<i>Dynamic response</i>		Measurement of mode shape and frequencies using accelerometers and an FFT analyzer	None
<i>Service-load</i>	Static load equivalent to (HS20 + 30") impact + sidewalk loads) in 2 patterns:	Static load applied hydraulically	- No cracking in segments - No joint separation - Sidewalk and overlay cracks less than 10 mils
	1. Maximum positive moment 2. Maximum negative moment		No overlay debonding
<i>Repeated negative service load</i>	± 50 cycles of service load in maximum longitudinal negative moment pattern	Hydraulically applied load - ± 50 cycles	- No cracking in segments - No joint separation - Overlay and sidewalk cracks less than 10 mils
			No overlay debonding
<i>Overload</i>	Static load equivalent to 1.5 (HS20 + 30") impact + sidewalk load) in 3 patterns:		- Cracks in segments less than 10 mils - Joint separation less than 10 mils - Overlay and sidewalk cracks less than 25 mils
	1. Maximum positive moment 2. Maximum negative moment	Static load applied hydraulically	- No overlay debonding - No signs of failure
<i>Ultimate load test</i>	Static load in a pattern that approximates design moment envelope.	Apply load to failure or test frame limit. If possible, continue testing until both a negative and positive moment failure develop	- First failure load at least 95% of factored loads - Flexural failure precedes shear or joint failure

PRETEST EVALUATION

Prior to load testing, the condition of the prototype was evaluated for cracking, overlay delamination, overlay bond and dynamic response. The crack and overlay delamination survey were conducted to establish the prototype condition before testing began. The overlay bond tests were performed to evaluate the surface preparation procedures of the segments. The dynamic response evaluation was performed to calibrate a finite element analysis model.

Crack Survey. A visual crack survey was performed on the top, bottom, and side surfaces of the prototype before the load test. Vertical hairline (less than 0.003 inches) shrinkage cracks were found on the north face of segment S6 and the south face of segment S1 (see Figure 17). Hairline shrinkage cracks were also found along the vertical edge of the sidewalk.

Overlay Delamination. A chain drag survey of the entire surface of the prototype was performed to determine if any areas of delamination existed before the load test. Delaminations of the latex-modified overlay were not found.

Overlay Bond Tests. In order to provide a good bonding surface for the overlay, the surface of each segment was roughened. Segments S4, S5, and S6 were roughened with a soft bristled broom. This finish did not have measurable amplitude and coarse aggregate was not showing. Segments S1, S2, and S3 were roughened with a stiff bristled broom. This finish had amplitudes of approximately 0.25 inches with some coarse aggregate showing. After the latex-modified concrete overlay had cured for 10 days, bond tests were performed on each segment. The average bond strength for the stiff bristle broom segments was 456 psi and for the soft bristle broom segments was 418 psi.

Dynamic Response. A modal analysis was performed on the prototype before the load test. The measured mode shapes and frequencies were for comparison to and calibration of analytical models. The purpose of these tests was to obtain a baseline dynamic response for use in comparison to and calibration of the analytical models.

STRUCTURAL TEST RESULTS

Service load tests for positive moment. Service loads causing maximum positive moments were applied. The load was applied in four steps, and strain gage readings as well as deflections and load cell readings were taken at the end of each load step. At maximum load, midbay deflection was about 0.11 inch. Deflections at the deck midspan, along with midspan deflections at the centerline of Rib A, Rib B, Column Line 1 and Column Line 2, are shown in Table 2. Also, the structure was inspected for cracks; none were observed.

The distribution of strains at the bottom surface showed low tensile strains across the middle span region. The distribution of strains at the bottom surface showed low tensile strains across the middle span region. The overall response of the deck system indicated the adequacy of the design under maximum service positive bending since no cracks were observed during this loading case.

Table 2 - Maximum deflections for service load and overload tests

Downward vertical deflection, in.			
LVD	Center Column Line B (CB)	Midbay (CC)	Center Column Line A (CA)
Service positive	0.084	0.111	0.048
Service negative	0.020	0.048	0.008
Overload positive	0.128	0.153	0.054
Overload negative	0.020	0.071	0.003

Service load test for negative moment. For maximum negative moments, service level loads were applied at zones 2, 3, 4, 5, 6, and 7 across the middle and the cantilever spans. This configuration resulted in tensile stresses along the top surface of Column Line B and Column Line 1. Longitudinal strain was measured on the sidewalk surface at the maximum service loading using mechanical gages. The strain measured over column support A1 was 44.2 microstrain and column support B1 was 101.2 microstrain. The first flexural crack, corresponding to load level 1, was observed on the sidewalk surface at column A1 at the end of this load step. The longitudinal strain at this location was 190 microstrain after which it abruptly increased to 467 microstrain. Strain levels at column B1 remained below the cracking strain and hence no cracking was observed at this loading level. The middle span experienced no major strain variations.

Low-cycle negative moment fatigue loading. A low cycle fatigue loading was performed while maintaining load levels at zones 2, 3, 4, and 8 at maximum service load values. Each hydraulic ram in zones 5, 6, and 7 were cycled between 0 kips and 13 kips for 50 cycles. During this cyclic loading, transverse cracks developed in the vicinity of Column B1, and the crack at Column A1 extended the full width of the sidewalk and widened to about 6 mils.

OVERLOAD TESTS

Overloads for positive bending. Overloads at 1.5 times the design service loads were applied to test the behavior of the deck system under positive bending.

Positive bending moment loads were applied at zones 2, 3, and 4. The response of the deck was monitored by observing strain gage readings, deflections, and crack development. At this loading, transverse as well as longitudinal cracks were observed on the overlay at Column B2. The recorded strains were lower than the expected tensile levels due to this crack formation. The middle span remained within the elastic limits on the compression side as well as on the tension side with no signs of any crack development. The displacement transducer located at the midspan closure pour detected a slight separation at the bottom of Beam B. The segment separated about 0.002 inches from the closure pour at maximum load.

Overloads for negative bending. For this case, 1.5 times the design service loads were applied at zones 2, 3, 4, 5, 6, and 7 to produce negative bending effects along the support line. Transverse and longitudinal cracks were noticed around columns B1 and A1 due to the high tensile strains reached during this loading sequence. These cracks were, in general, a continuation of the cracks that originally developed during the previous load stages. The mechanical strain gage also picked up cracks in the walkway scored joints on either side of Column Line 1. Strain measurements indicated the primary crack over support widened to about 6 mils, and the cracks at the scored joints were about 2 mils wide. The overlay cracks at Column B1 were of similar width. In addition, hairline cracks on either side of Column Line B developed in the overlay due to transverse negative moment.

ULTIMATE LOAD TESTS

Three ultimate load cases were applied to observe the post-cracking behavior and modes of failure. The first test simulated stresses in proportion to the longitudinal and transverse moment envelopes, followed by testing for ultimate positive bending moment. Finally, ultimate loads were applied along the expansion joint. This loading sequence secured the required integrity in the region of interest for subsequent tests.

Ultimate load test 1. Loads for this case were applied at zones 2, 3, 4, 5, 6, 7, and 8 in equal increments up to failure. The loading simulated stresses in proportion to the longitudinal and transverse moment envelopes. The following is a summary of the noteworthy events that occurred during this test.

Table 3 - Event summary for ultimate load test 1

Total Super-imposed Load (kips)	Multiple of Service Live Load	Event	Event No.
1001	2.6	Separation at closure pour	1
1519	4.4	Separation at joint between Segments 4 and 5 at Column Line B	2
1756	5.2	Primary crack width at Column A1 increased to 0.014 inches.	3
2413	7.5	Flexural tension cracking at midspan of Beam B	4
2800	8.8	Flexural compression failure at bottom of Beam B at Column Line 1. See Figure 7.3.	5

The strains and deflections were monitored throughout the loading procedure to note the load levels for imminent cracking. Excessive longitudinal and transverse cracks occurred along Column Line B and Column Line 1, respectively (Figure 18). Delamination of the overlay layer was noticed at Column B1 due to the bridging of cracks in the transverse and longitudinal directions. The middle span experienced high compressive strains on the top surface, but not exceeding the crushing strength of the latex-modified concrete layer. The tensile strains on the bottom surface of the middle span were, however, beyond the concrete tensile strength and some tension cracks were observed. Some of the strain gages in the tension regions were affected by the extent of cracks, and were either defected or their locations isolated so the readings at these high levels of load were not reflective of the actual strain distribution.

Loading was stopped at the 7th load stage when Beam B failed in flexural compression at the support (Figure 19). Maximum strain measured with an internal strain gage was about 1500 microstrain less than the crushing strain, indicating the gage malfunctioned or was located just outside of the crushing zone.

Ultimate load test 2. After the negative moment failure occurred at Column B1, a second ultimate load test was carried out to further evaluate the extreme loading response to the main span. The deck system was then loaded at zones 2, 3, and 4 for maximum positive bending effects. The test was stopped prior to failure when the jack capacity was reached at a service multiple of about 8.8. Wide transverse cracks developed in the main beams, but these cracks closed when the load was removed. Longitudinal cracks in the overlay also developed,

but did not close completely, indicating some yielding of the transverse negative moment reinforcement.

Compressive strains at the center of the middle span top surface attained a value of 2200 microstrain in the transverse direction and 1870 microstrain in the longitudinal direction at the end of this loading level. These strains were below the crushing level and hence no signs of material degradation were observed at that location.

Ultimate load test 3. Finally, the structure was loaded along the expansion joint by simultaneously loading the jacks in the two rows paralleling the joint. Deflections were measured at the midspan of Column Line 2 and the cantilever adjacent to Column Line B at Column Line 2. Although the overhang deflected as much as 2.4 inches and very wide (60 mils) longitudinal cracks developed in the overlay and expansion joint segment, jack capacity was reached before failure occurred. The main span recovered completely with residual deflection in the cantilever of about 0.6 inches indicating yielding of the transverse mild reinforcement.

CONCLUSIONS

In conclusion, the load test and analyses of the Wacker Drive prototype indicate outstanding strength and serviceability of the structural system. Also, the problems encountered during fabrication and erection of the segments was minor. Special attention is required to avoid interference between longitudinal and transverse post-tensioning ducts, and care must be exercised to avoid damage, misalignment and blockage of ducts during construction.

The number of strain gages installed and their location on the prototype was chosen for redundancy and to help analyze the data from the computer analysis. In hindsight, the number of gages could have been reduced to limit the amount of analysis that has to be performed to verify readings. The mechanical Whittemore gage instrumentation performed the best and most reliably. The gage was able to monitor strain and span cracks to measure crack width. Electronic gages would malfunction when trying to bridge cracks and ultimately give erroneous readings.

ACKNOWLEDGEMENTS

The University of Illinois at Chicago provided invaluable support to this project. The authors are particularly grateful for the assistance of Md. S. Islam, Hiba Abdalla, and Aduana Fanuel.

The support of WJE staff in conducting this research is also gratefully acknowledged. The authors would like to specifically thank Nick Hyatt, Roger Pelletier, Brian Waszak, Dave Nixon, and Mary Horency.

The prototype was fabricated and assembled by PBM Concrete in Rochelle, Illinois. Their performance in this difficult task is a credit to their talent as a precast producer. Special thanks are due to Bill Able and Greg Fisher.

The prototype was developed based on a design by Earth Tech and J. Muller International. In particular, we wish to acknowledge Frank Powers of Earth Tech for his assistance in coordinating the project, as well as Pat Cassity and Eddie He of JMI for their design assistance.

Finally, this project was funded by the Federal Highway Administration, the Illinois Department of Transportation, and the Chicago Department of Transportation. The authors wish to thank the administrators and contributors who made this project possible.

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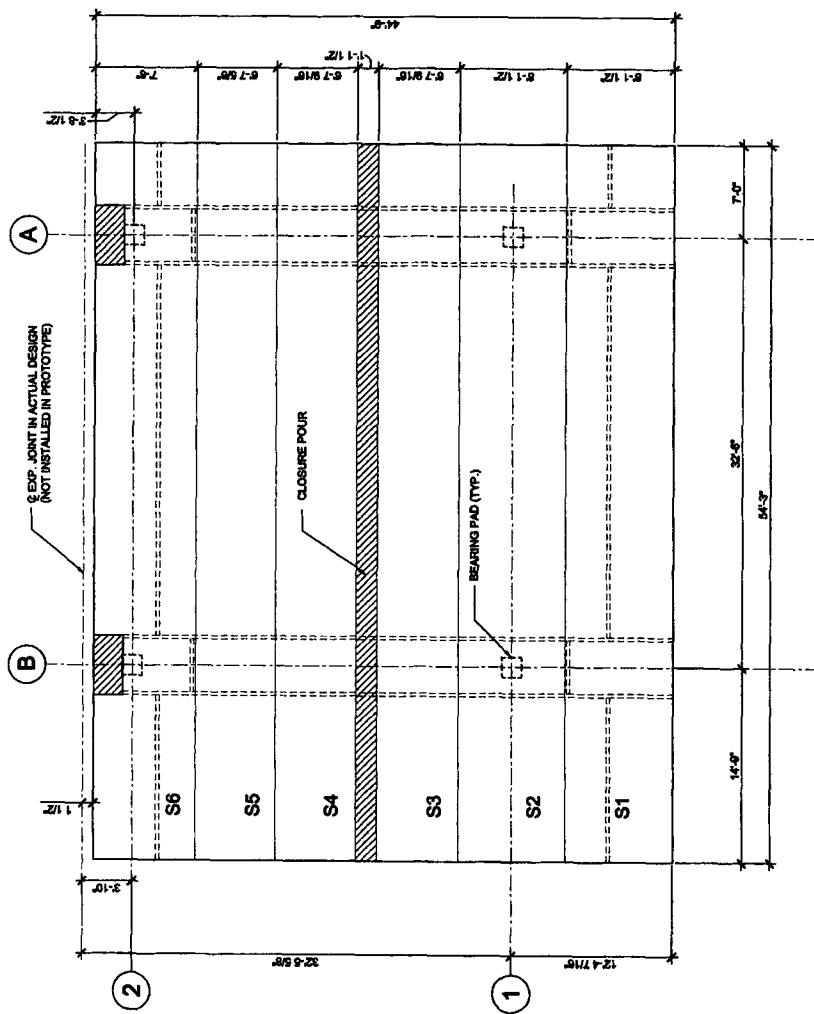


Figure 1. General layout of prototype deck

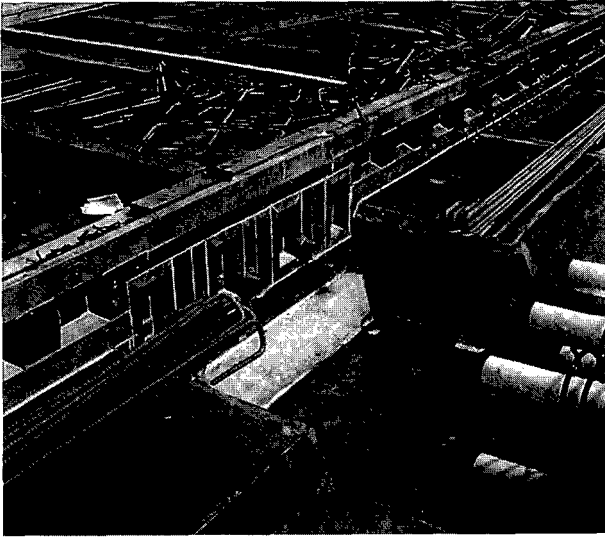


Figure 2. Shear key form set on casting bed

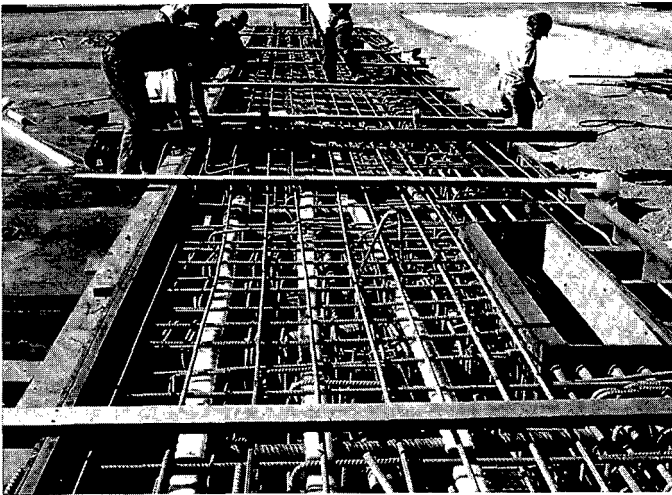


Figure 3. Reinforcing bars and post-tensioning ducts for Segment 6

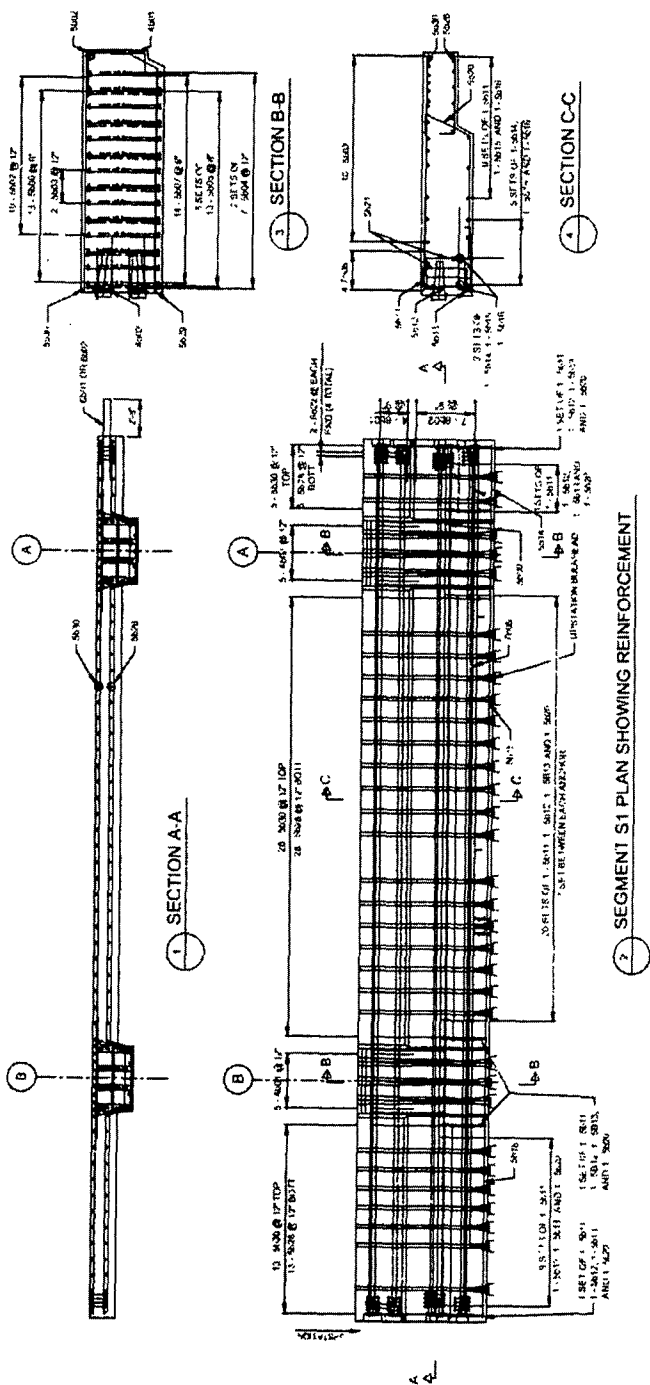


Figure 4. Plan and section view of segment S1

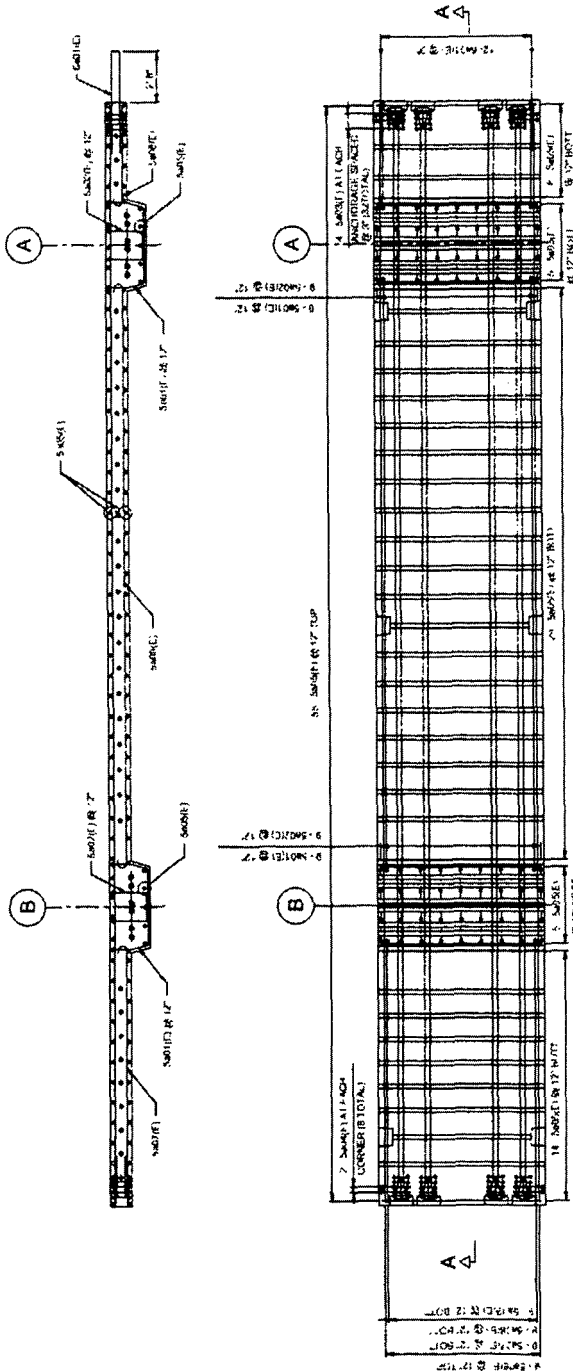


Figure 5. Plan and section view of segments S2 through S5 reinforcement

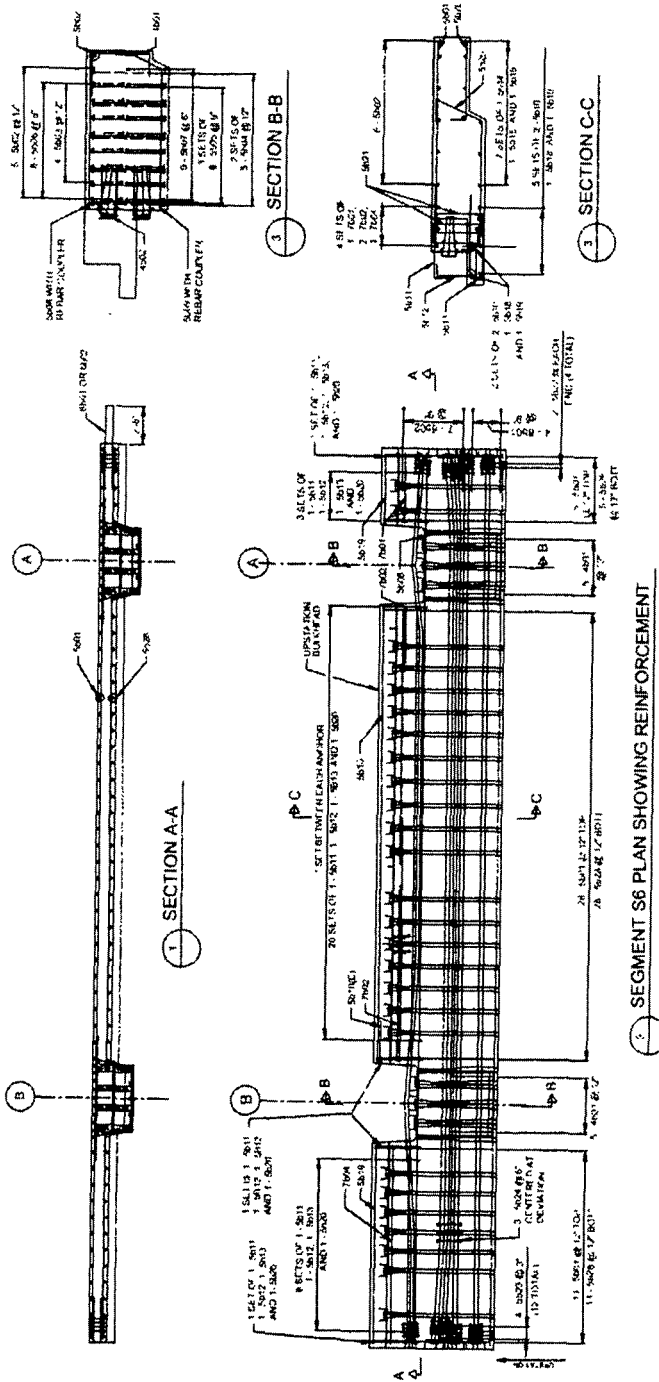


Figure 6. Plan and section view of Segment S6

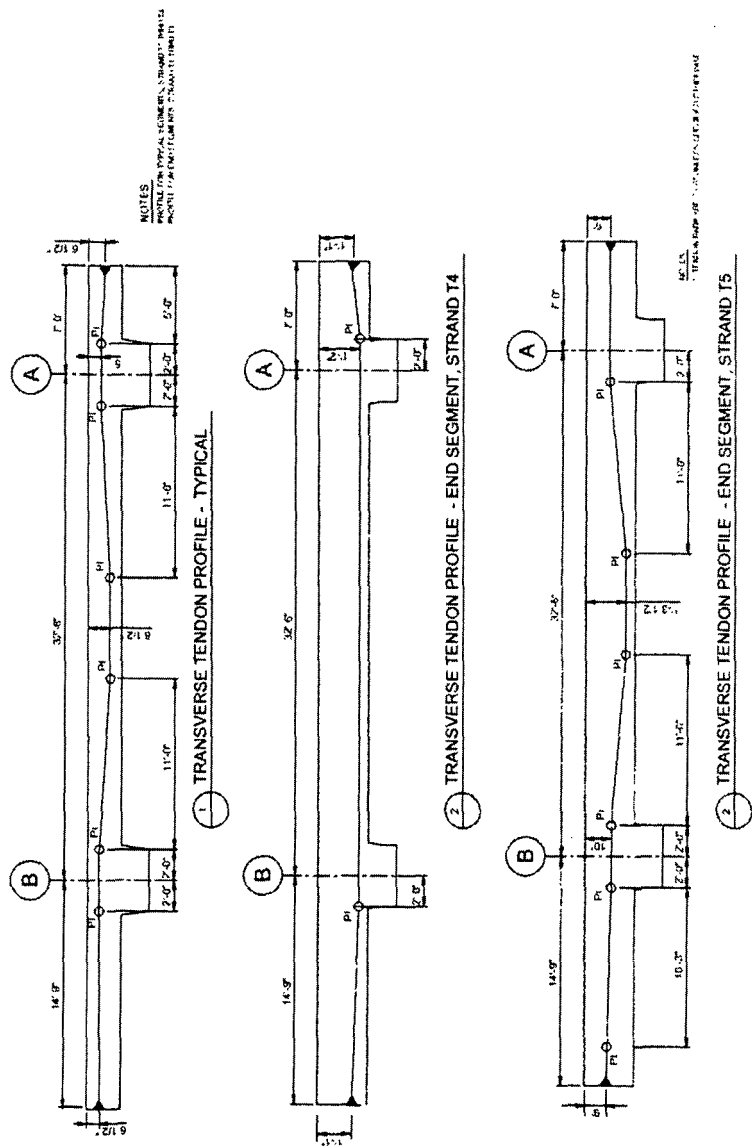


Figure 7. Transverse tendon profiles for segments

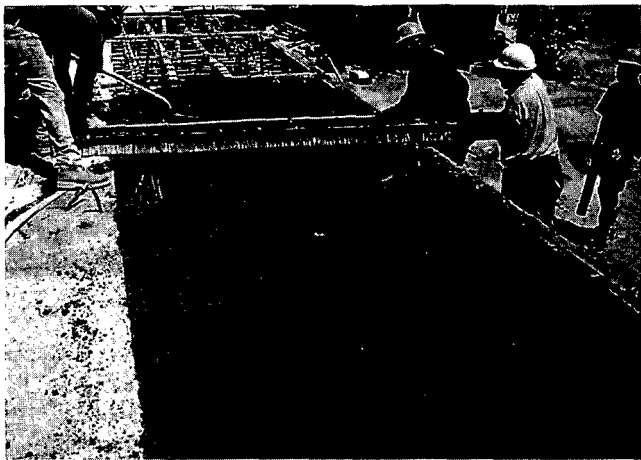


Figure 8. Broom finish on segment top surface

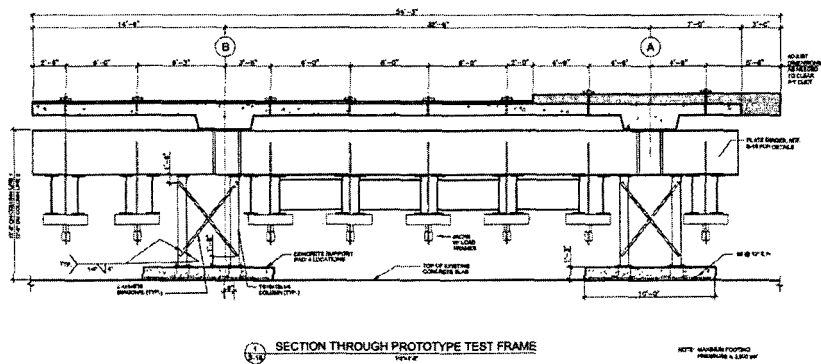


Figure 9. Elevation view of load frame

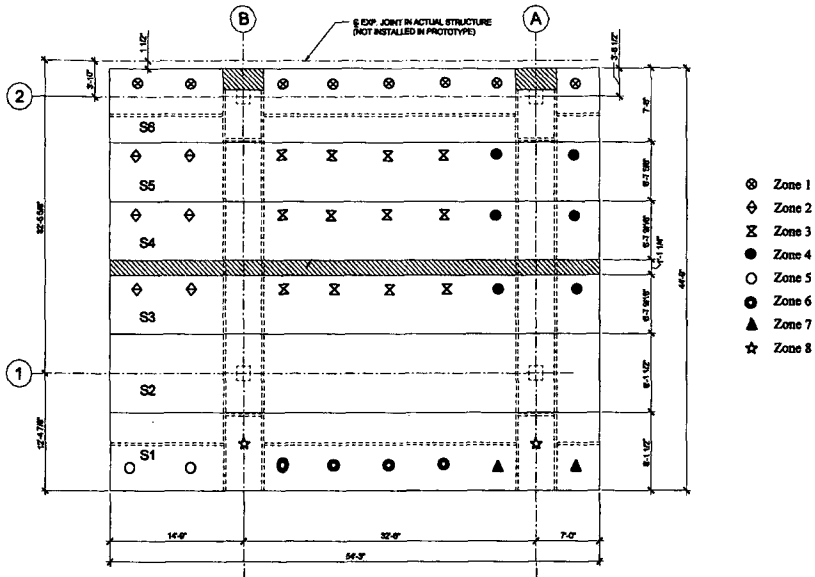


Figure 10. Plan view showing zones of load application



Figure 11. Positioning of segments by crane prior to bonding together

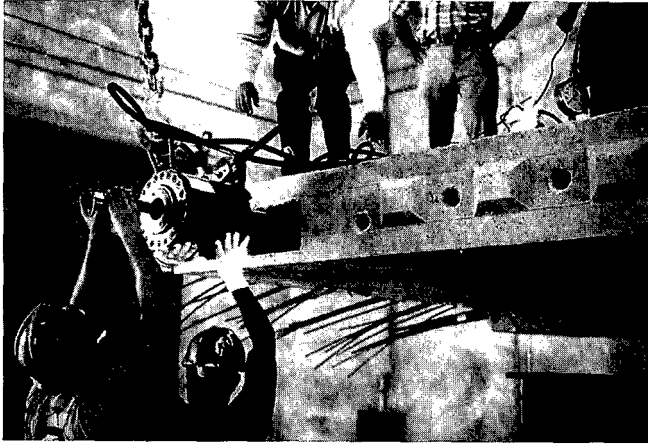


Figure 12. Bonding segments together using epoxy and post-tensioning dywidag bars



Figure 13. Grouting of post-tensioning ducts

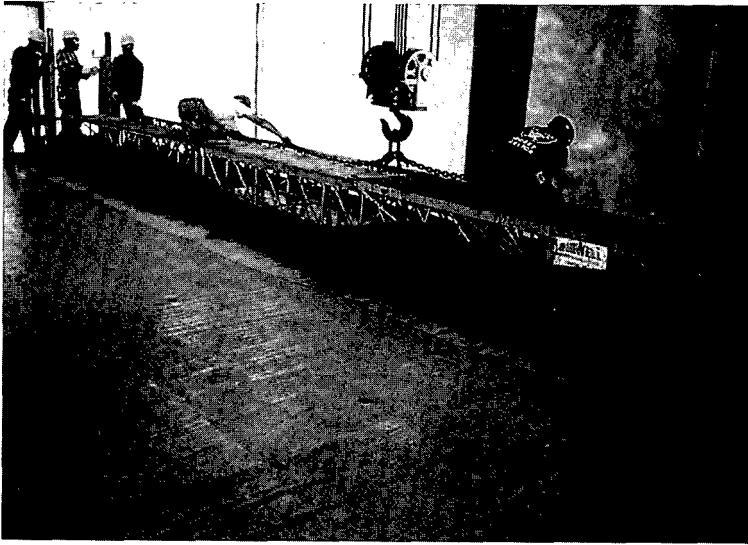


Figure 14. Wet burlap drag on latex-modified concrete overlay

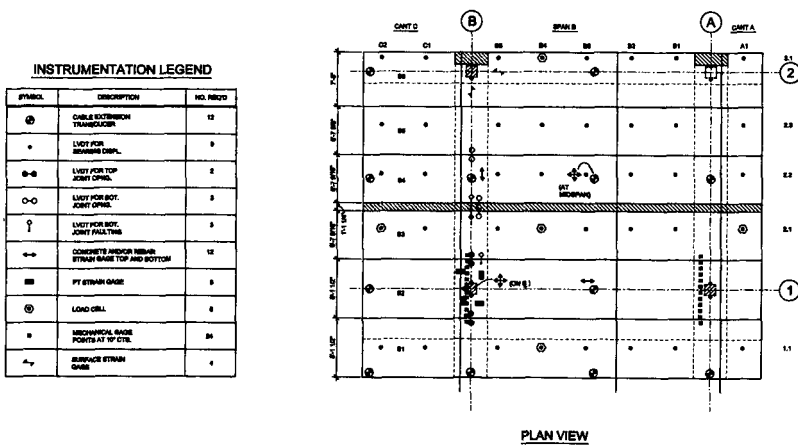


Figure 15. Instrumentation plan view



Figure 16. Whittemore gage locations for measuring strain by mechanical means

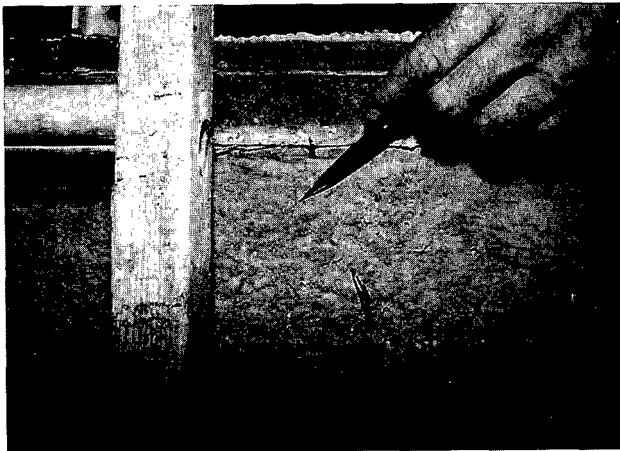


Figure 17. Typical hairline shrinkage cracks found before load testing