DESIGN ULTIMATE LOAD TEST OF 1/10-SCALE MICRO-CONCRETE MODEL OF NEW POTOMAC RIVER CROSSING, I-266

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HIGHLIGHTS

Under contract with the designers of the proposed bridge, Howard, Needles, Tammen and Bergendoff, Consulting Engineers of New York City, a 1/10-scale micro-concrete model of a prestressed concrete cantilever box girder bridge was constructed and tested in the Structural Research Laboratory of the Portland Cement Association. An artist's rendering of the prototype, the proposed I-266 Potomac River crossing at Washington, D.C., is shown in Fig. 1. When completed, the bridge will be one of the largest cantilever prestressed concrete bridges in the world.

The prototype bridge on which the model was based has a 750-ft. (229 m) main span as shown in Fig. 2. Each side span will be 440 ft. (134 m) and the roadway deck will be 110 ft. (34 m) wide. Since the prototype is symmetrical about the center of the main span, only one-half of the bridge was modeled as can be seen in Fig. 3. At 1/10-scale, the model was 81 ft. 6 in. (24.8 m) long, 11 ft. (3.4 m) wide, about 6 ft. (1.8 m) deep at the pier and 1 ft. 3 in. (0.38 m) deep at midspan and at the abutment. Constructed of materials having properties similar to those of the prototype, the model represented the "direct" method of structural modeling as de-

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Described are the details of making and testing a 1/10-scale model of a proposed three-span bridge for Washington, D.C., that will be one of the world's longest prestressed concrete cantilever type bridges when completed. Test results are given that show suitability of design and conformance with specifications.

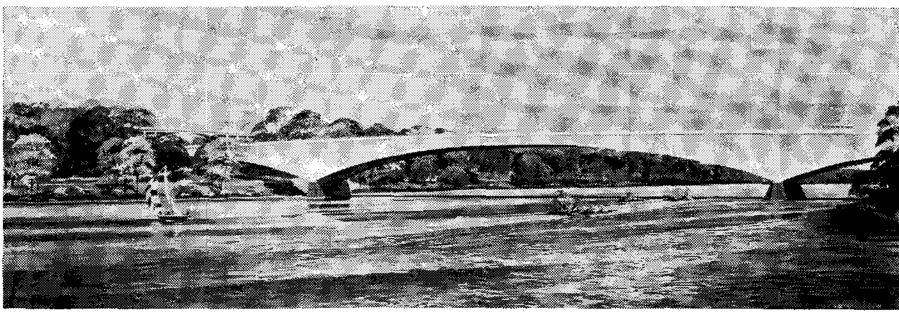

Fig. 1. Proposed prestressed concrete cantilever bridge for I-266 Potomac River crossing near Three Sisters Islands

scribed in detail elsewhere[1].

The webs and fascias of the prototype box girder bridge will be 18 in. (0.46 m) thick and 60 ft. (18.3 m) deep at the pier, more slender than any previously used in this type of bridge. In addition, the fascias are heavily curved in cross-section. Both side spans are curved in plan and therefore subjected to considerable torsion. Since the construction of this bridge will set several precedents, it was decided that structural model tests should be used to assist in the design. The tests were carried out to study performance of the model bridge under application of dead load and design live load. In addition, behavior of the model under extreme overload was to be determined.

This report describes construction of the 1/10-scale prestressed concrete model, the testing procedure, and the results of both service load and design ultimate load tests. It is shown that the model supported the design service load without structural cracking and safely withstood the severe overload of 1.5D + 2.5 (L + I)* required by Section 1.6.6–Load Factors, AASHO *Standard Specifications for Highway Bridges*[2].

*D = effect of dead load
L = effect of design live load
I = impact load

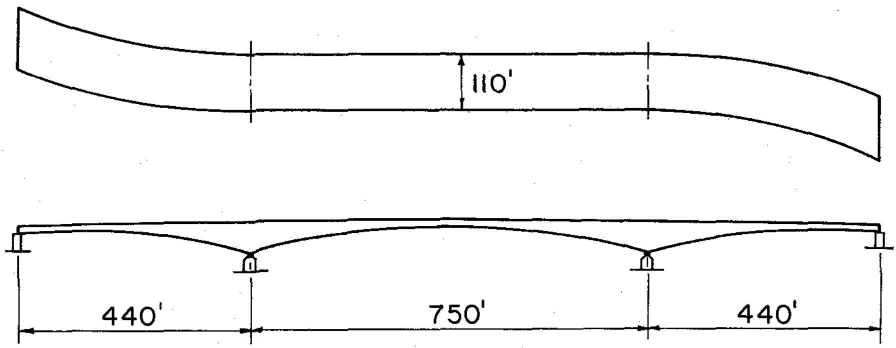

Fig. 2. Plan and elevation of prototype bridge

MODEL CONSTRUCTION

Assembly of superstructure. Although the prototype is designed to be cast in place in segments, the model was constructed of precast 3-ft. (0.91 m) segments that were sequentially grouted in position and post-tensioned together to form the complete bridge. This use of precast segments was strictly for convenience in the laboratory. To simulate the field construction, continuity of reinforcement was maintained across all joints. Dimensions of a model segment near the pier are shown in Fig. 4. Complete details of construction and testing will be given elsewhere(3).

The superstructure segment directly over the pier was cast in a special form and set on a steel "rocker" supported on a reinforced concrete block representing the pier. Next, the adjacent first main-span and side-span segments were positioned and the joints connecting them to the pier segment were cast. The second main-span and side-span segments were then positioned against the completed 3-segment portion of the bridge, the connecting joints were cast, and the appropriate tendons were tensioned. This procedure was repeated until the model bridge was constructed.

Post-tensioning of the prestressing tendons proceeded with erection of each new segment. In general, the negative moment longitudinal tendons required to connect a given segment to the completed portion of the bridge were tensioned one day after the joint was cast. Appropriate diagonal web tendons crossing the joint and a similar number of fascia tendons and deck transverse tendons were also tensioned soon after each new segment was in place. The remaining tendons were tensioned later, as stresses calculated for the erection plan permitted. Other tendons located in the pier diaphragm, the abutment diaphragm, and the soffit positive moment region near the abutment were also tensioned when calculated stresses indicated that it was appropriate(3).

Using the cantilever method, the model bridge was constructed so that the superstructure was always heavier on the abutment side of the pier. Overturning of the partially completed bridge was prevented by

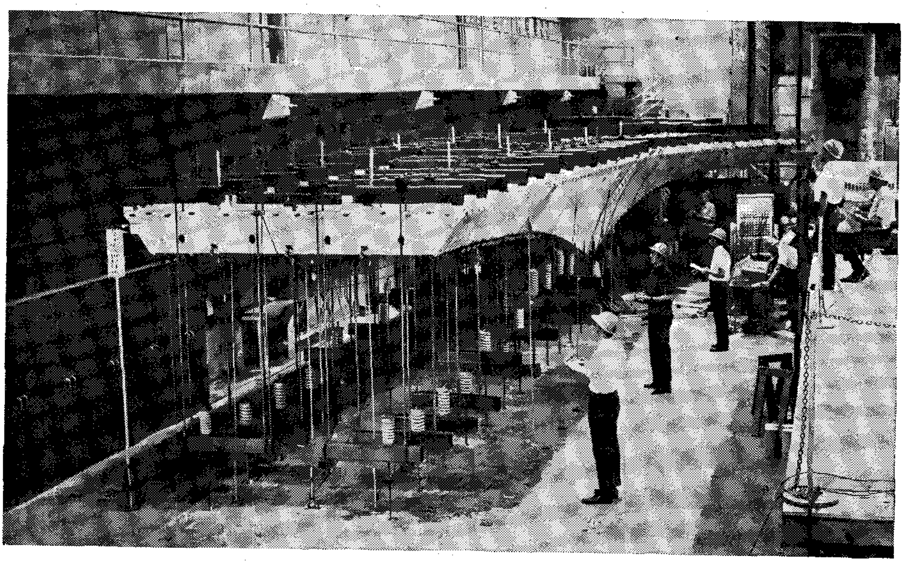

Fig. 3. Test setup for 81½-ft. long model

a temporary support initially located near the pier in the side span and later moved to a position about two-thirds of the side span away from the pier. At all times after the first longitudinal tendons were stressed, a 3,000-lb. (1360 kg) weight representing the 300,000-lb. (136 t) weight of construction equipment on the prototype, was kept near each end of the model[3]. It is intended that the same cantilever construction proce-

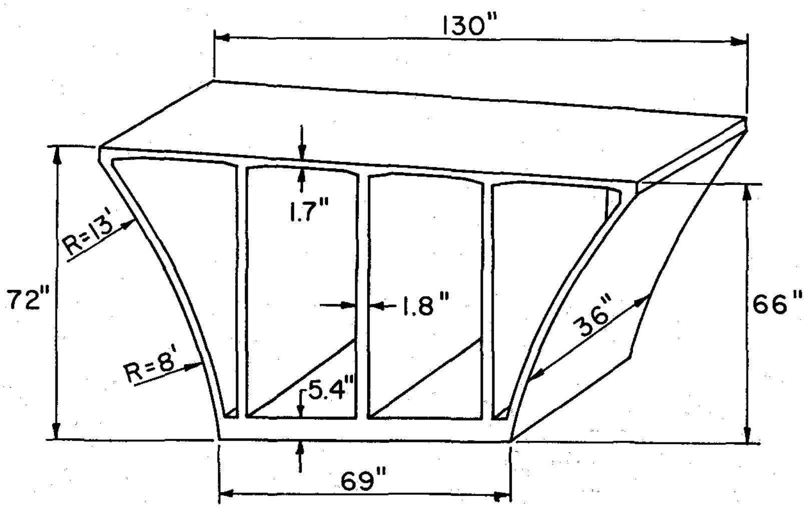

Fig. 4. Dimensions of precast model segment near pier

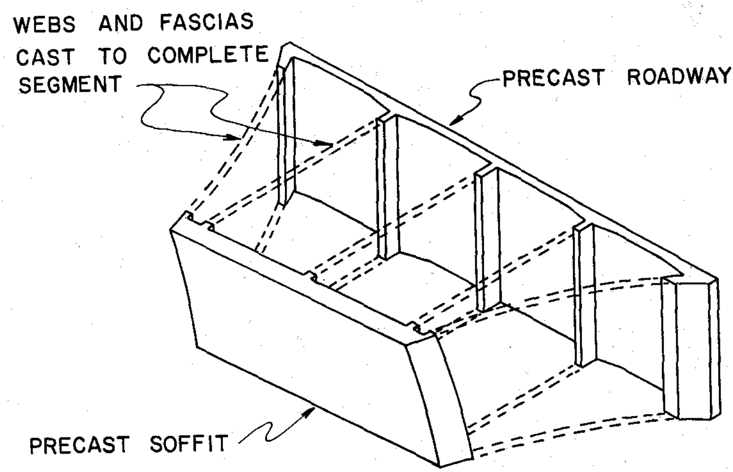

Fig. 5. Model bridge segment construction

dure will be used for the prototype.

Construction of segments. Each precast model bridge segment was constructed in three operations. Roadway and soffit sections were precast separately and then joined together by casting webs and fascias. Fig. 5 shows this procedure schematically.

Soffit sections were cast on a continuous platform built each side of the pier at the location where the model bridge was later to be assembled and tested. The platform, shown in Fig. 6, was constructed to the shape of the bottom surface of the soffit. It served initially as a base for casting soffit sections and subsequently as a working platform during erection of the bridge. Curved side forms for fascia stubs, interior forms for web stubs, and greased rods to form prestressing ducts in the stubs were attached to the platform. Continuity of geometry was ensured by casting the soffit sections in sequence starting each side of the pier.

Roadway sections were cast deck surface down on special adjustable platforms as shown in Fig. 7. Side forms for the fascia stubs, interior forms for the web stubs, and greased rods to form prestressing ducts in the stubs were attached to the platforms. Longitudinal and transverse prestressing ducts were formed by greased steel rods encased in polyvinyl chloride (PVC) tubing and placed in the deck. Continuity of the roadway sections was ensured by aligning longitudinal ducts with metal templates and, as can be seen in Fig. 7, by casting each new section against a previously cast section.

The precast roadway and soffit sections were placed in an assembly frame that was adjusted to ensure longitudinal prestressing duct continuity and proper relative geometry. In the assembly frame, web and fascia prestressing ducts were formed by steel rods encased in PVC tubing.

Web duct continuity between segments was established by use of adjustable templates. After forms were attached to the web and fascia stubs on the deck and soffit sections, as shown in Fig. 8, the bridge segment was completed by casting the remaining portions of webs and fascias.

Erection of segments. The erection of each 3-ft. (0.91 m) section of the bridge model began with the hoisting of the segment onto an adjustable temporary scaffolding and clamping it to the completed portion of the bridge. The temporary scaffolding and clamping were then adjusted until observation by surveying instruments[4] indicated that the segment was in its proper position relative to the completed portion of the bridge.

Once a segment was properly aligned, duct formers for prestressing tendons were placed across the joint, and other required joint forming was done. Prior to placing the joint concrete, a slow curing epoxy

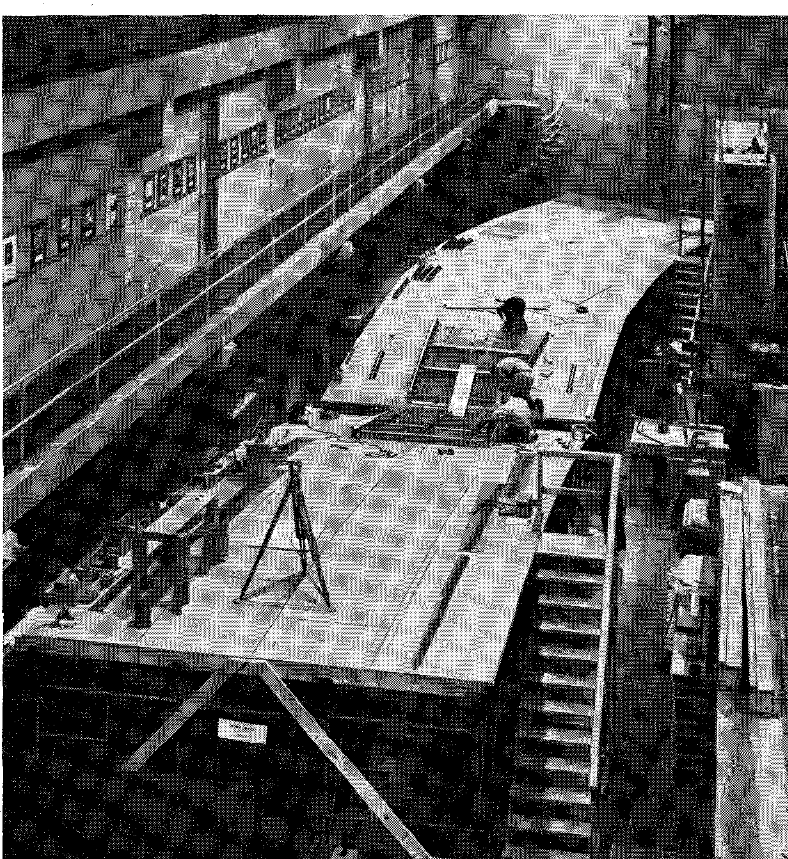

Fig. 6. Soffit platform used for casting soffit slabs

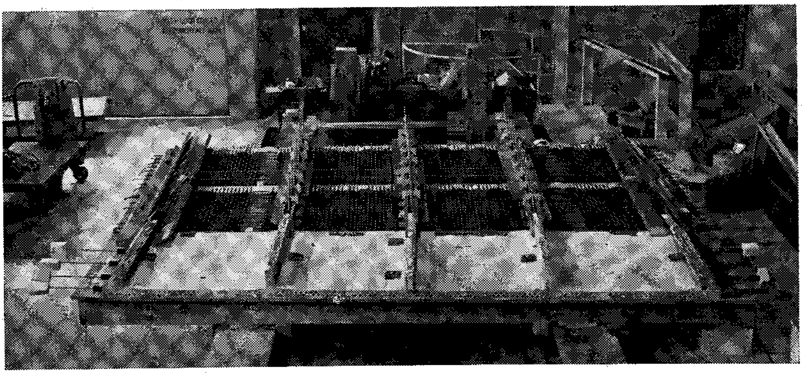

Fig. 7. Precast deck slabs were cast on adjustable platforms

adhesive was placed on those portions of the joint surfaces that approximately paralleled the planes of principal stress and would be subjected to high shearing stresses during the test.

After curing, the joint was stripped, duct formers were removed, tendons were threaded through the ducts, and anchor assemblies were attached. To complete the erection cycle, tendons were stressed and the temporary supports were removed. Generally, each cycle was completed on the third day after placing the joint concrete.

Method of prestressing and compensation for losses. Prestressing tendons in the deck, soffit, webs, fascias and diaphragms were anchored at the dead end with a button head and at the stressing end with a friction anchor. An adjustment device was placed between the friction anchor and the anchor plate to provide a means for precise tensioning of each tendon. Fig. 9 shows the anchorage assembly used for deck tendons.

Two operations were required to complete tensioning of a tendon. First, a stressing jack gripped and pulled the wire while pushing against the end of the friction anchor. When this jacking force was released, the friction anchor gripped the tendon. Loss in tendon force caused by slip within the anchor was recovered in the second operation when the jack pushed against the bearing plate instead of against the anchor. This lifted the grip from the adjustment device so that it could be extended to hold the anchor in its correct position.

All tendons were stressed initially to 20 percent above the final desired value. This overstress was chosen to compensate for losses calculated for steel relaxation, concrete creep and shrinkage and tendon friction. A representative deck tendon was estimated to retain 81 percent of original prestress after losses of 8.4 percent from relaxation, 6.0 percent from creep, 0.4 percent from shrinkage and 4.2 percent from friction. Relaxation loss was calculated using an equation developed at the University of Illinois[5]. Creep, shrinkage and friction losses were determined from material tests and fric-

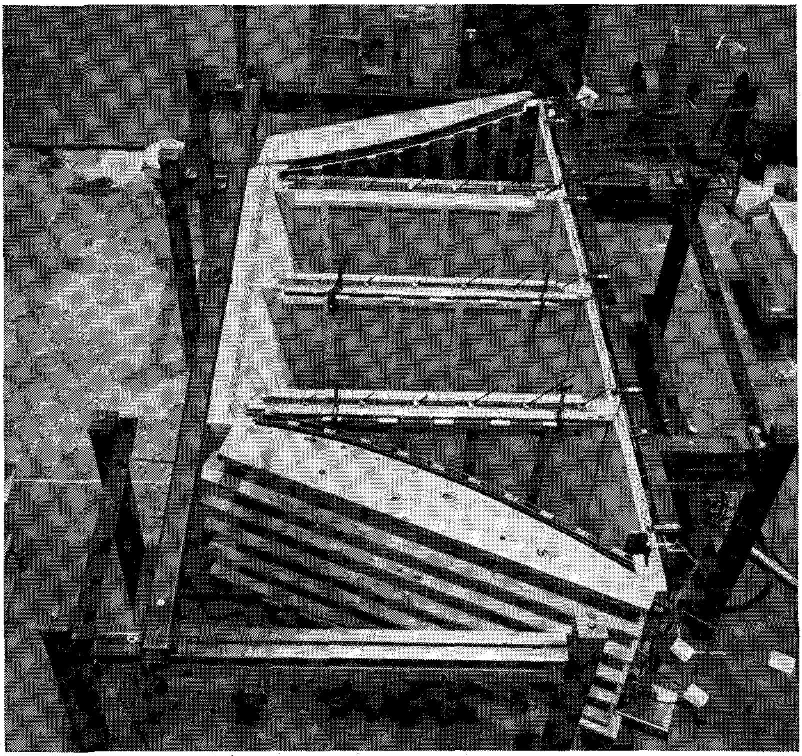

Fig. 8. Precast segment with web and fascia forms in place

tion tests carried out in conjunction with construction of the model.

Similitude of reinforcement. Grade 60 deformed bar reinforcement[6] in the prototype was represented by galvanized welded wire fabric having a yield stress of about 60 ksi (4220 kgf/cm^2) and meeting requirements of ASTM Designations: A185-68 and A82-66[7,8]. Mesh sizes used were 2 x 2 in.–12/12 and 2 x 2 in.–14/14 (5.1 x 5.1 cm–2.05/2.05 mm and 5.1 x 5.1 cm–1.63/1.63 mm). The size and amount of mesh was varied to provide about the same percentage of reinforcement at each section in the model as in the prototype.

The 32 mm (1¼ in.) prestressing bars considered in the design of the prototype were represented by 5 mm or ¼ in. (6.35 mm) prestressing wire in the model. Final desired prestress-

Fig. 9. Anchorage assembly

Table 1. Properties of model concrete at 28 days

Location of concrete	Test of 6 x 12-in. cylinders*	Average values psi	Standard deviation psi	Coefficient of variation, %
Segments	f'_c	5,870	520	8.9
Segments	E_c	3,350,000	192,000	5.7
Joints	f'_c	7,770	595	7.7

* f'_c = compressive strength
E_c = modulus of elasticity
1000 psi = 70.3 kgf/cm²

ing force at a nominal stress of about 0.6 f'_s, where f'_s is the strength of the wire, was 7.07 kips (3210 kg) for the ¼-in. wire and 4.35 kips (1970 kg) for the 5-mm wire. The total prestressing force rather than the individual tendon force was modeled. Consequently, the ¼-in. and 5-mm prestressing wires in the model represented approximately six and four tendons, respectively, in the prototype.

Longitudinal ducts for the ¼-in. (6.35 mm) tendons in the roadway were placed in two layers with a 1-in. (2.5 cm) transverse spacing. A total of 170 tendons passed through the five segments in the vicinity of the pier. Approximately 10 percent of these were anchored at each end of the first five segments and a like amount were anchored at each joint beyond.

As seen in Fig. 7, ducts for transverse 5-mm roadway tendons were placed in a single layer between the deck longitudinal tendons. Where required near the abutment in the side span, ¼-in. (6.35 mm) longitudinal soffit tendons were placed at mid-depth of the soffit slab and spaced 3½ in. (8.9 cm) on centers. Web prestress was applied with ¼-in. or 5-mm tendons spaced at 6¾ in. (17.2 cm) or 9 in. (22.9 cm) as required to reproduce prototype stresses.

Properties of concrete and prestressing reinforcement. A micro-concrete mix with proportions of 1 part Type III cement to about 5.25 parts Elgin fine aggregate was used to cast the segments. A water-cement ratio of 0.7 gave a slump of about 1 in. (2.5 cm).

The 28-day compressive strength and elastic modulus measured by compression tests of 6 x 12-in. (15 x 30 cm) cylinders are summarized in Table 1.

Joints between the segments were made from a low slump mortar of 1 part Type III cement and 3 parts masonry sand. The 28-day compressive strengths, measured by compression tests of 6 x 12-in. (15 x 30 cm) cylinders, are listed in Table 1.

Prestressing wire met the requirements of ASTM Designation: A421-65[9]. Strengths obtained from tests of representative samples of wire used in the model are shown in Table 2.

TESTING PROCEDURE

Application of loads during construction. As construction of the model proceeded, forces were applied to simulate the dead load of the prototype[1]. At 1/10-scale, the model required application of nine times the self-weight of each segment to reproduce prototype stresses. A schematic drawing showing loading equipment provided at 3-ft. (0.91 m) intervals along the bridge is shown in Fig. 10.

After a new segment had been stressed, force was applied to the lower rods of loading equipment at the completed joint. This was done by means of temporary hydraulic rams located below the test floor[4]. Anchor nuts on the rods were then tightened against the lower side of the floor to hold the required force. Coil springs in the system permitted

Table 2. Properties of model prestressing reinforcement

Diameter	Yield stress at 1% elongation f_y (ksi)	Ultimate strength f'_s (ksi)
5 mm.	264	292
0.25 in.	218	254

1000 psi = 1 ksi = 70.3 kgf/cm²

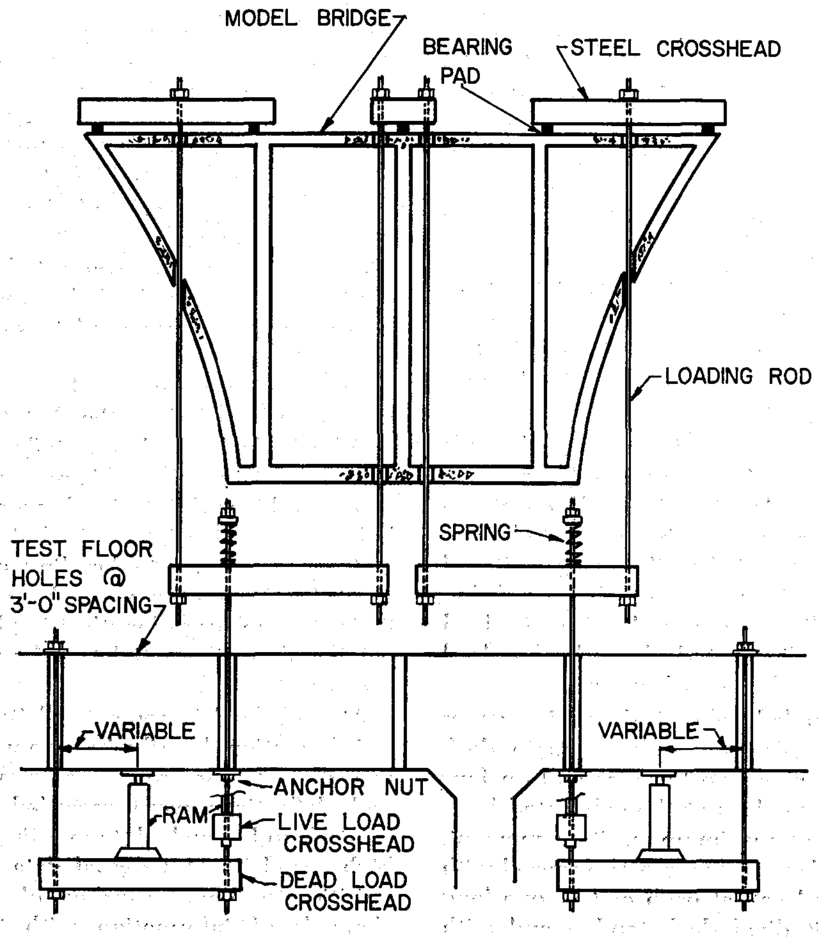

Fig. 10. Loading equipment details

small movements of the bridge during construction while maintaining the total load within a minimum of 93 percent of that intended. Total applied load during construction of the model was determined both by measuring the spring lengths and by measuring the pier and support reactions.

Temporary support was provided under the side span of the model during erection. The first support was located 5.5 ft. (1.7 m) from the pier until ten main span segments and eleven side span segments had been erected. Load was then transferred to a temporary support located 29.3 ft. (8.9 m) from the pier. This support was removed when the bridge was seated on the abutment. Forces at the temporary support and at the pier were measured at each stage of construction.

Instrumentation. Loads, reactions, reinforcement strains, concrete strains, web and fascia deflections, superstructure deflections, and rotations were measured during the test using procedures described elsewhere[3,10].

Eighteen calibrated load cells were used to measure forces. Ten cells under the pier and two at the side span abutment measured reactions. Applied loads were monitored both with pressure cells in each hydraulic system and with six load cells distributed through the dead load and live load systems.

Bonded electrical strain gages were placed on eleven main deck tendons above the pier and on four soffit tendons at the calculated location of maximum positive moment. Tendon forces were also sensed with a load cell at each end of one long longitudinal deck tendon and with one load cell each on a soffit tendon, a web diagonal tendon and a fascia tendon.

Strain gages were placed on the concrete surfaces at 360 locations. These included gages on nine heavily instrumented cross-sections, two diaphragms, and on the soffit near the pier.

Lateral deflection at mid-depth of each web and fascia was measured at a location midway between the pier diaphragm and the first intermediate diaphragm in both the main span and the side span. To accomplish this, a vertical framework was attached near the top and bottom of each web and fascia to support a linear variable differential transformer displacement sensor at mid-height[10].

Deflections of the superstructure were sensed at 26 locations including the quarter-points and mid-lengths of each span and the free end of the main span cantilever. A linear potentiometer connected to the test floor directly below each measuring point was used to sense the vertical movements.

Rotation was measured over a gage length including the first longitudinal deck tendon cutoffs on each side of the pier. This measurement combines the strain above and below the section into a single indication of response to bending moment. Strains were measured over a 24-in. (61 cm) gage length using linear variable differential transformer sensors.

At each load increment, all items of information described above were recorded. Loads, reactions, strains and deflections were recorded on printed and punched tape using a high speed VIDAR digital data acquisition system. Recording of 400 channels of information with this equipment required about 40 sec-

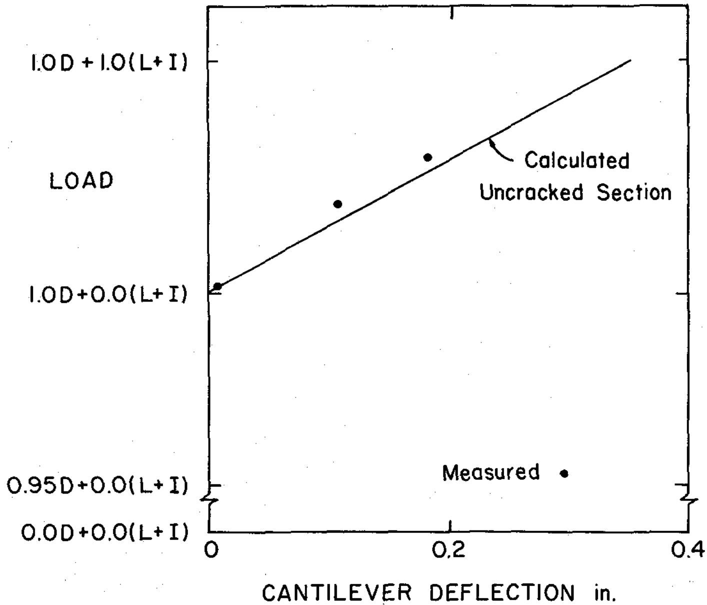

Fig. 11. Load vs. deflection of cantilever end for service load test

onds. Rotation and web lateral deflection were continuously traced on oscillographic recorders. Selected load vs. rotation, load vs. vertical deflection, and load vs. web lateral deflection outputs were displayed continuously on X-Y recorders with the Y-axis representing applied load.

Application of design loads. Hydraulic loading equipment below the test floor was arranged to apply dead load and live load to the model using techniques described elsewhere(4). The two independent hydraulic systems used to apply these loads are shown schematically in Fig. 10.

The first step in the test sequence was to transfer the load from the springs to the hydraulic system and hold 1.0 dead load of the prototype (1.0 D), that is, a load ten times the self-weight of the model. A set of initial readings was then taken.

Design service load tests—After initial readings were taken under 1.0 D, 1.0 (L + I) representing both lane loads and concentrated loads for HS20-44 loading(2) was added in five equal increments. All electronic instruments were read and the model was visually inspected at each increment. This represented the design service load condition of 1.0 D + 1.0 (L + I). No cracking due to application of this load was observed. The live load was then reduced to zero and all gages were again read.

Dead load tests—After completion of the design service load test, the dead load was increased from 1.0 D to 1.3 D in six equal increments. All electronic gages were read and the

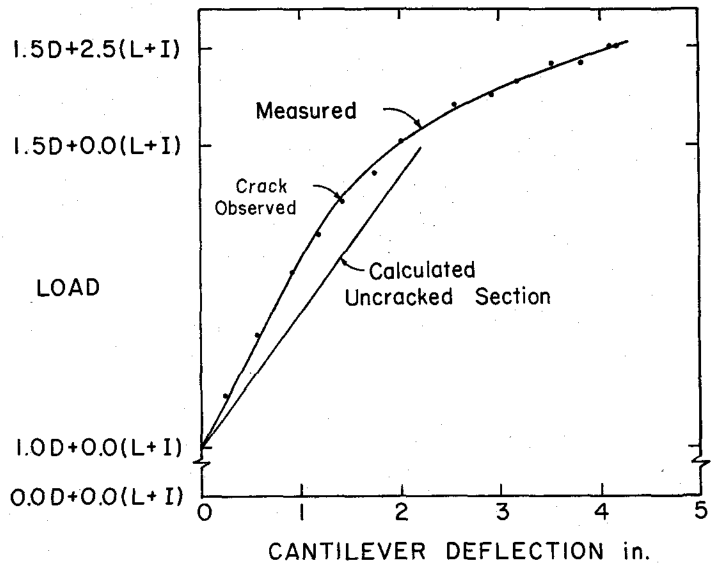

Fig. 12. Load vs. deflection of cantilever end for ultimate load test

model was visually inspected at each increment. No cracking due to application of this load was observed. The load was then reduced to 1.0 D and instruments were read to determine residuals.

Design ultimate load test—The dead load was first increased from 1.0 D to 1.3 D in three equal increments. Then four smaller, equal increments of load were added to bring the total up to 1.5 D. Finally, live load was applied in four increments until a total of 1.5 D and 2.5 (L + I) was carried by the model. All electronic gages were read and the model was visually inspected after each increment. Under application of the design ultimate load, cracks were observed both over the pier and near the abutment. Although some inelastic deformation was evident, the model safely supported this extreme overload. The load was then reduced to 1.0 D, and another set of readings was taken.

TEST RESULTS

Performance during construction. Observed reactions, strains and deflections during construction of the bridge model were all within anticipated limits. The bridge model was observed to respond elastically as each new segment was erected and dead load was applied. In addition, no cracks attributed to applied load were found.

Performance under design service load. Under the design service load of 1.0 D + 1.0 (L + I), the microconcrete model was observed to perform as anticipated. No cracks caused by applied load were found. Strains and deflections measured at critical locations were observed to be proportional to the applied load, indicating that the structure remained essentially elastic. A comparison of measured and calculated load deflection curves for mid-span (end of cantilever) deflections is shown in Fig. 11. Calculated deflections were based on an uncracked section and measured material properties.

Behavior at service load was within the limits generally assumed in design. These experimental results indicate that the serviceability requirements implied by the AASHO Specifications[2] are met by the design.

Performance under design ultimate load. After the service load tests, the design ultimate load of 1.5 D + 2.5 (L + I) was applied to the microconcrete model. Under this extreme overload, the model was observed to safely carry the applied loads. Some inelastic strains and deflections were observed, and cracks occurred both in the negative moment region over the pier and in the positive moment region near the abutment. All of the inelastic behavior observed under application of design ultimate load was within ranges anticipated.

Fig. 12 shows calculated and measured load vs. deflection relationships for mid-span (end of cantilever). Calculated deflections were based on an uncracked section and measured material properties. As can be seen, the observed deflections were in satisfactory agreement with those calculated.

After the overload was removed and the condition of 1.0 D had been restored, all cracks were observed to have closed until they were barely visible to the naked eye. This behavior indicated, and measured strains confirmed, that the longitudinal prestressing tendons remained elastic under application of the design ultimate load.

CONCLUDING REMARKS

Results of the structural tests carried out on this 1/10-scale model of the new Potomac River crossing, I-266, show that under the application of service load, representing dead load of the prototype and live load plus impact under HS20-44 loading, no structural cracking occurred and the bridge remained essentially elastic. Consequently, the design meets serviceability requirements implied by the AASHO Specifications. In addition, the results show that the model safely carried the extreme overload of 1.5 D + 2.5 (L + I) with only minor structural cracking. Consequently, it is concluded that the design also meets the strength requirements of Section 1.6.6—Load Factors of the AASHO Specifications.

Following completion of the design ultimate load test, several special tests will be made to compare computed and observed behavior of the model. Finally, the model will be tested to destruction.

ACKNOWLEDGMENTS

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Mr. G. I. Sawyer, Assistant Director, Department of Highways and Traffic, District of Columbia, is the owner's respresentative.

The model was constructed and tested by the Cement and Concrete Research Institute, a Division of the

Portland Cement Association. Work was carried out by personnel of the Structural Research Section with the assistance of personnel from the Paving and Transportation Research Section. Felix Barda, Associate Research Engineer, and A. P. Christensen, Senior Research Engineer, provided valuable assistance in the design and construction of the model.

Dr. Roy E. Rowe, Director of Research and Development, Cement and Concrete Association, Great Britain, served as Models Consultant.

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Discussion of this paper is invited. Please forward your comments to PCI Headquarters by March 1 to permit publication in the March-April 1972 issue of the PCI JOURNAL.