Tests of a 1/10-scale concrete model to aid design of a large prestressed bridge

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Tests of a 1/10-Scale Concrete Model to Aid Design of a Large Prestressed Bridge

Essais sur modèle en béton à l'échelle 1 : 10; complément pour l'étude du projet d'un grand pont précontraint

Versuche an einem Betonmodell im Massstab 1 : 10 als Hilfsmittel beim Entwurf einer grossen vorgespannten Brücke

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1. HIGHLIGHTS

As noted by the Reporter, the use of structural models provides a powerful aid to analytical design. Models permit direct determination of elastic behavior of complex structures that must be simplified to permit structural analysis. Even more important, concrete models provide the only reliable means of determining the ultimate strength of complex concrete structures.

Planned for construction across the Potomac River near Washington, D. C., the proposed Three Sisters Bridge, shown in the artist's rendering in Fig. 1, is unique in several respects. In addition to its 750 ft. (228. 6 m) main span, it has curved 440-ft. (134. 1 m) side spans and the 110-ft. (33. 5 m) roadway is widest of any post-tensioned, cantilevered bridge yet constructed. Consequently, it was decided that this design should be supplemented by the construction and testing to destruction of a 1/10-scale model of the prototype.

Fig. 1. Proposed I-266 Potomac River Bridge

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Under contract with the designers Howard, Needles, Tammen and Bergendoff, Consulting Engineers of New York City, a 1/10-scale concrete model of the Three Sisters Bridge was constructed and tested in the Structural Development Laboratory of the Portland Cement Association. Dimensions of the prototype bridge are shown in Fig. 2. Since the prototype is symmetrical about the center of the main span, only one-half of the bridge was modeled. At 1/10-scale, the model shown in Fig. 3 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. (38 cm) deep at midspan and at the abutment. Constructed using materials having properties similar to those of the prototype, the model represented the "Direct" method of structural modeling as described in detail elsewhere. (1)

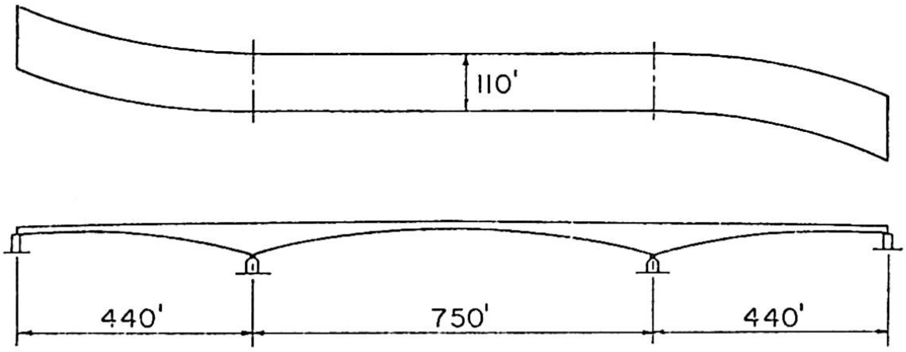

Fig. 2. Plan and elevation of prototype bridge

This report describes construction of the 1/10-scale prestressed concrete model, the testing procedure, instrumentation and data processing. Important results of service load tests, design ultimate load tests, and tests to destruction are reported. It is shown that the model supported the design service load without structural cracking and safely withstood the severe overload of 1.5 D + 2.5 (L + I)* required by "Section 1.6.6 - Load Factors," the AASHO <u>Standard Specifications for Highway Bridges.</u>(2) Furthermore, it withstood an overload of 1.5 + 6.0 (L + I) before the flexural capacity of the bridge was reached in the main span at the pier.

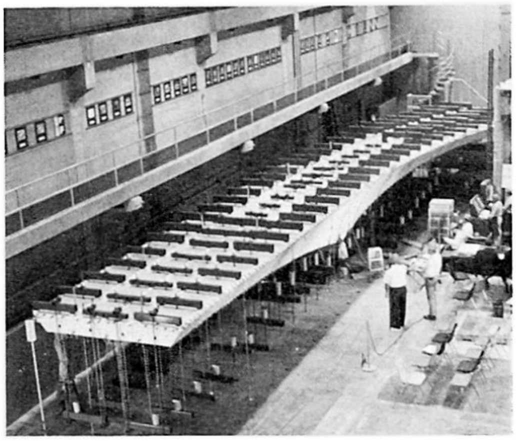

Fig. 3. Bridge Model

2. 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. (91.4 cm) 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 field construction, continuity of non-prestressed reinforcement was maintained across all joints. Dimensions of a model segment near the pier are shown in Fig. 4. Details of construction and testing are given elsewhere. (3)

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

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 a temporary support initially located 5.5 ft. (1.7 m) from the pier and later moved to a position 29.3 ft. (8.9 m) from the pier. This support was removed when the bridge was seated on the abutment. At all times after the first longitudinal tendons were stressed, a 3,000 lb. (1.36 t) weight representing the 300,000-lb. (136.4 t) weight of construction equipment on the prototype, was kept near each end of the model. It is intended that the same cantilever construction procedure will be used for the prototype.

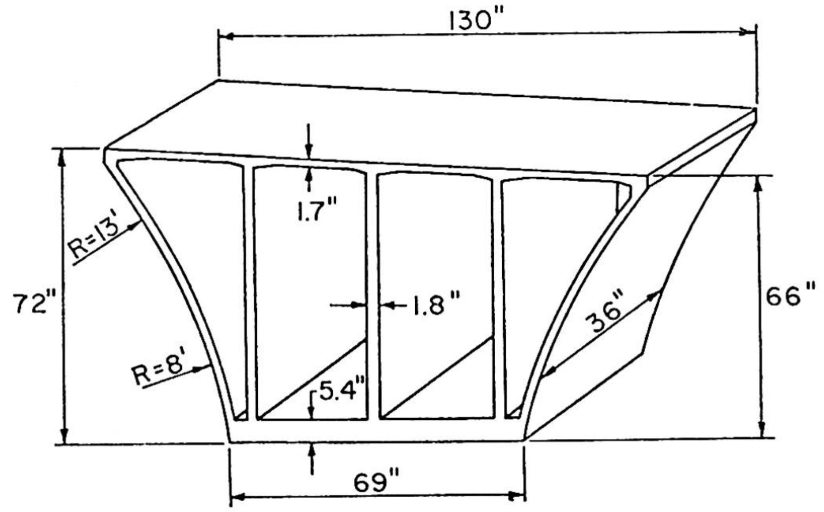

Fig. 4. Dimensions of model segment near pier

Construction of Segments - Each precast model bridge segment was constructed in three operations.(3) Roadway and soffit sections were precast separately and then joined together by casting webs and fascias. Figure 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 served initially as a base for casting soffit sections and subsequently as a working platform during erection of the bridge.

Roadway sections were cast in pairs, deck surface down on special adjustable platforms. Continuity of the roadway section was ensured by aligning longitudinal ducts with metal templates and by casting each new section against a previously cast section.

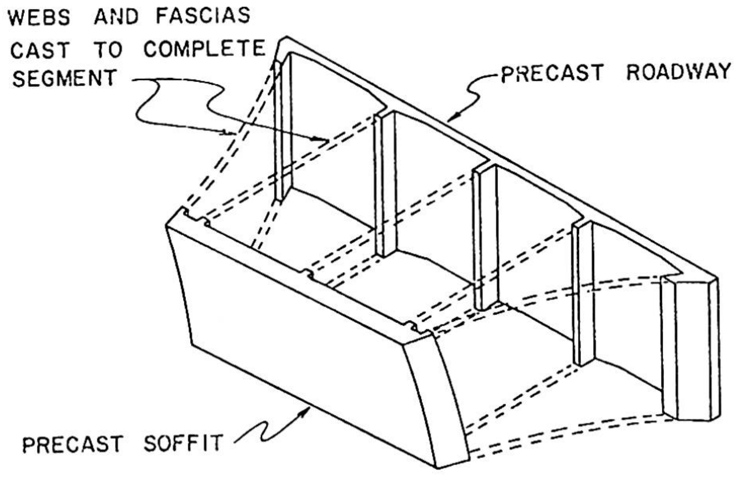

Fig. 5. Model bridge segment construction

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. 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, the bridge segment was completed by casting the center portions of webs and fascias.

Erection of Segments - The erection of each piece of the bridge model began with lifting 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, joint concrete was placed. To complete the erection cycle, tendons were stressed and the temporary supports were removed.

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. (3) An adjustment device was placed between the friction anchor and the anchor plate to provide a means for precise tensioning of each tendon.

All tendons were stressed initially to 20 percent above the final desired value. This overstress was chosen to compensate for calculated losses from steel relaxation, concrete creep and shrinkage and tendon friction.

Similitude of Reinforcement - Grade 60 deformed bar reinforcement in the prototype was represented by galvanized welded wire fabric having a yield stress of about 60 ksi and meeting requirements of ASTM Designation: A185-68 and A82-66. Mesh sizes used were 2x2 in. (5x5 cm) - 12/12 and 2x2 in. (5x5 cm) - 14/14. 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 (1-1/4 in.) prestressing bars considered in the design of the prototype were represented by 5 mm (0.2 in.) or 1/4 in. (6.35 mm) prestressing wire in the model. The total prestressing force rather than the individual tendon force was modeled. Consequently, the 1/4-in. and 5 mm prestressing wires in the model represented approximately six and four tendons, respectively, in the prototype. Details of placement of the reinforcement are provided elsewhere. (3)

Properties of Concrete and Prestressing Reinforcement - A concrete mix with proportions of 1 part Type III cement to 5.25 parts Elgin fine aggregate was used to cast the segments. The 28-day strength and elastic modulus measured by compression tests of 6x12-in. (15x30 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 strengths measured by compression tests of 6x12-in. cylinders are listed in Table 1.

TABLE 1 - Properties of Model Concrete at 28 Days

Location of Concrete	Test of 6x12-in. (15x30 cm) Cylinders*	Average Values psi (kg/cm^2)
Segments	f'_c	5,870 (412)
Segments	E_c	3,350,000 (235,000)
Joints	f'_c	7,770 (546)

* f'_c = compressive strength; E_c = modulus of elasticity

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

TABLE 2 - Properties of Model Prestressing Reinforcement

Diameter	Yield Stress at 1% elongation f_y ksi (kg/cm²)	Strength f'_s ksi (kg/cm²)
5 mm	264 (18, 500)	292 (20, 500)
0. 25 in. (6. 35 mm)	218 (15, 300)	254 (17, 800)

3. 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. Load was applied by means of temporary hydraulic rams located below the test floor. (3, 4) Anchor nuts on the loading rods were then tightened against the lower side of the floor to hold the required force. Coil springs in the system permitted small movements of the bridge during construction while maintaining the total load within a minimum of 93 percent of that intended.

Application of Dead Load - Hydraulic loading equipment below the test floor was arranged to apply dead load and live load to the model using techniques described elsewhere. (3, 4) Two independent hydraulic systems were used to apply these loads.

The first step in the test sequence was to transfer the load from the springs to the hydraulic system and hold the prototype dead load (1. 0 D), i. e., a load ten times the self weight of the model. A set of initial readings used as a "zero" reference for all tests was then taken.

Design Service Load Tests - After initial readings were taken under 1. 0 D, lane loads and concentrated loads representing 1. 0 (L + I) for HS20-44 loading (2) were added in 5 equal increments. All electronic instruments were read and the model was visually inspected at each increment. Under this design service load condition of 1. 0 D + 1. 0 (L + I), no cracking due to application of load was observed.

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 model was visually inspected at each increment. No cracking due to application of this load was observed.

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.

Test to Destruction - The dead load was first increased from 1. 0 D to 1. 3 D in one increment and to 1. 5 D in a second increment. Live load was then applied to bring the total to 1.5 D +4.0 (L + I) with five stops to take readings. The live load was then reduced to zero and the model was held at 1.5 D overnight.

The next day live load was again increased and another set of readings was taken at 1.5 D +4.0 (L + I). Two more increments of 1.0 (L + I) were then applied and all instruments were read. While maintaining a load of 1.5 D + 6.0 (L + I), the flexural capacity of the main span at the pier was reached and the load dropped off suddenly. When this occurred, crushing of the concrete over the entire cross section released all longitudinal prestress and deflections of the model released all applied load. This completed structural testing of the 1/10-scale model.

4. INSTRUMENTATION AND DATA PROCESSING

Loads, reactions, reinforcement strains, concrete strains, web and fascia horizontal deflections, superstructure deflections and rotations were measured during the test using procedures described elsewhere. (3, 4)

Eighteen 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 and Whittemore mechanical 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 fascia tendon.

Strain gages were placed on the concrete surface at 360 locations. This included gages on nine heavily instrumented cross sections, two diaphragms, and on the soffit near the pier. At each heavily instrumented cross section, longitudinal gages were provided over each web on both deck and soffit, over the fascias on the deck and at mid-depth of two webs and one fascia. At sections near the pier, additional longitudinal gages were placed on all webs and fascias near the bottom of the deck.

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 (4) displacement sensor at mid-height.

Deflections of the superstructure were measured 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 24-in. gage length including the first longitudinal deck tendon cutoff each side of the pier. Both rotation and web lateral deflection were continuously traced on oscillographic recorders. Selected load versus rotation, load versus vertical deflection, and load versus web lateral deflection outputs were displayed continuously on X-Y recorders with the Y-axis representing applied load.

At each load increment, all data were recorded. Loads, reactions, strains and deflections were recorded on printed and punched tape using a high speed VIDAR digital data acquisition system shown in Fig. 6. The punched tape can be used to feed the data directly into an IBM 1130 Computer. Recording of 400 channels of information with this equipment required about 40 seconds.

As soon as a complete set of data was recorded on the VIDAR, selected readings were fed into a Hewlett-Packard 9100B (H-P 9100B) desk computer located in the test control center as shown in Fig. 6. This computer was programmed to give measured values of reactions, key stresses and deflections. As the test progressed, the measured values were compared with previously calculated values. This immediate access to important reduced data permitted the test to proceed rapidly while still maintaining complete control.

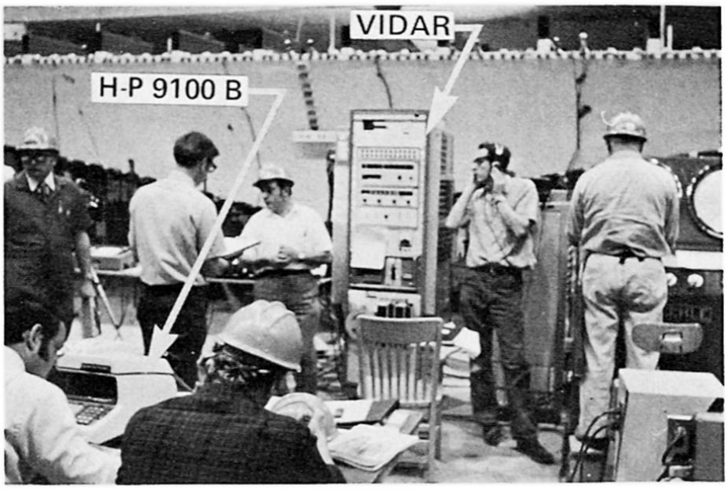

FIG. 6. Test control center

After each test was completed, the data were reduced and analyzed with the help of an IBM 1130 Computer. All readings were converted to strain, load or deflection and tabulated. The data were then analyzed to determine force distributions in the model, principal stresses at selected locations, load versus deformation relationships and other selected information.

In a separate operation, the H-P 9100B desk computer was used in combination with an on-line calculator plotter to plot load versus deformation data. This permitted a large amount of data to be plotted and studied in a very short time.

5. 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 concrete 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. 7. Calculated deflections were based on an uncracked section and measured material properties.

Behavior at service load was within 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 concrete model. Under this extreme overload 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.

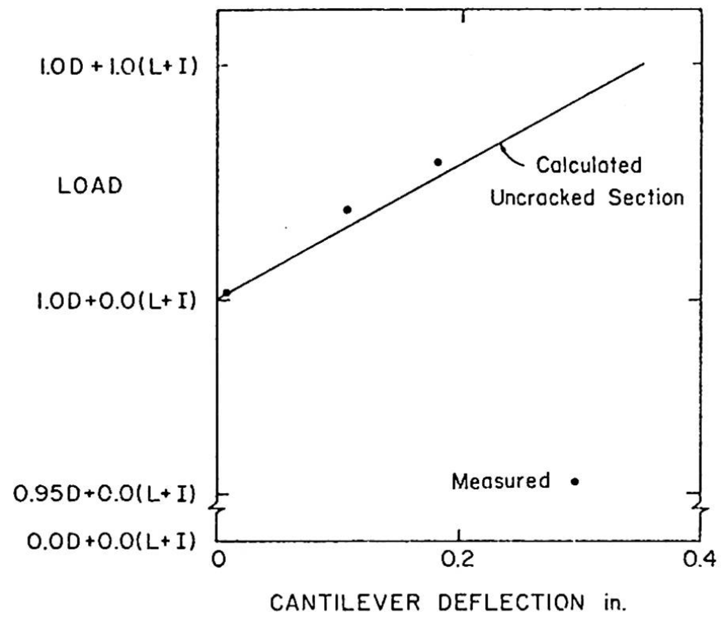

FIG. 7. Cantilever deflection - service load test

Figure 8 shows calculated and measured load versus 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.

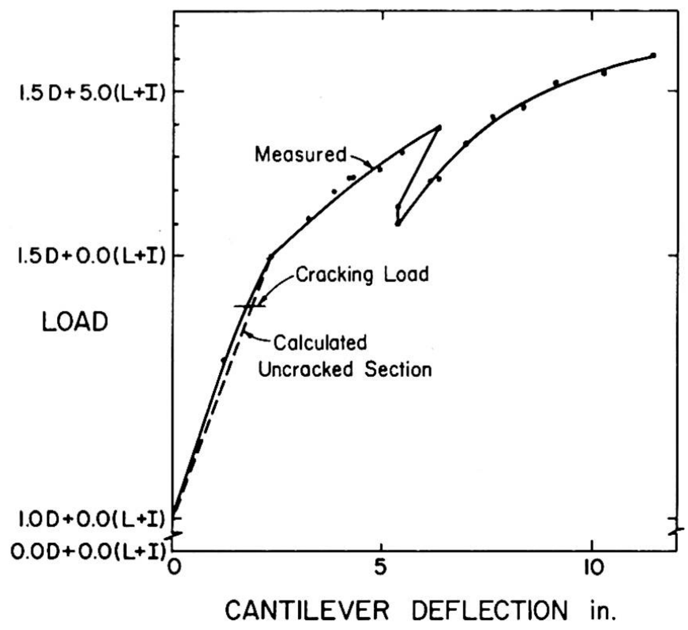

FIG. 8. Cantilever load vs. deflection - test to destruction

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.

Performance Under Test to Destruction - As overload was added during the test to destruction, additional flexural and torsional cracking developed. However, no large cracks were observed either in the webs or the fascias of the model.

After a total load of 1.5 D + 4.0 (L + I) had been applied, it was observed that an increase in load caused significant spalling of the concrete at the first joint in the main span. Measured strains indicated that general crushing of the concrete occurred as the load went from 1.5 D + 5.0 (L + I) to 1.5 D + 6.0 (L + I). At this load, the flexural capacity of the main span was reached and the test ended. Fig. 9 shows the pier region after the test.

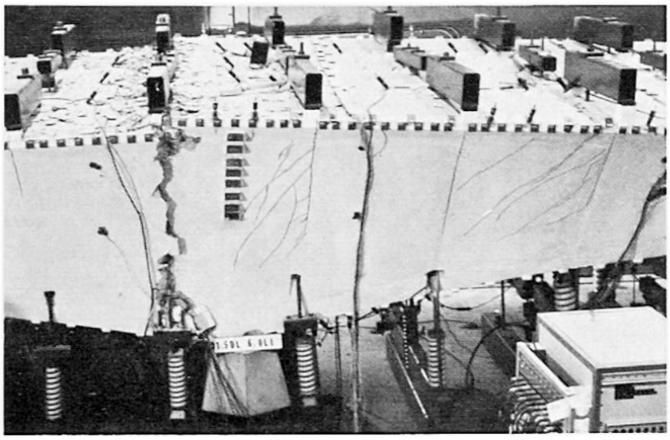

FIG. 9. Pier region after test

A comparison of measured moment capacity with that calculated on the basis of Article 1.6.10 of the AASHO Specifications (2) and using measured material properties gives a ratio of $M_{u\,(test)}/M_{u\,(calc)} = 1.15$. A similar comparison using moments calculated on the basis of compatibility of deformations and equilibrium of forces gives a ratio of $M_{u\,(test)}/M_{u\,(calc)} = 0.99$. These comparisons indicate that measured and calculated values of flexural strength are in satisfactory agreement.

6. ACKNOWLEDGEMENTS

This work was done under contract with the designers, Howard, Needles, Tammen and Bergendoff, Consulting Engineers of New York City. Mr. Gerard F. Fox is the partner-in-charge of design and Mr. Fred H. Sterbenz is the Project Engineer. Mr. A. Gordon Lorimer is the Consulting Architect on the project.

Mr. G. I. Sawyer, Assistant Director, Department of Traffic and Highways, District of Columbia, is the Owner's Representative. This work was carried out under Federal Aid Project No. DC-VA-I-266-2(102)1. The work was aided by staff of the Federal Highway Administration, U.S. Department of Transportation.

The model was constructed and tested in the Structural Development Laboratory of the Portland Cement Association. Professional staff members responsible for construction and testing of the model were Dr. J. E. Carpenter, Dr. H. G. Russell, N. W. Hanson, Dr. A. E. Cardenas, T. Helgason, Dr. J. M. Hanson and Felix Barda. Mr. A. P. Christensen and other personnel from the Transportation Development Section 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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8. SUMMARY

Previous model tests such as those of the Medway Bridge constructed in Southern England have shown the usefulness of a concrete model to aid in design. Tests of the Three Sisters Bridge model have carried the design process another step forward by providing a direct link between the model and the computer.

Results of this test indicate that all of the unusual features of the bridge are adequately accounted for in the design. Service load performance met or surpassed the design requirements and the required safety against overload was exceeded.