ual has been successfully followed in repairing damaged bridges of the Hanshin Expressway.

ACKNOWLEDGMENT

The authors would like to thank many colleagues with whom they have discussed these tests, and they will be pleased if this paper is helpful to people concerned with bridge repair.

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# Scale-Model Tests for Full-Depth Precast Concrete Panel-Decked Composite Bridge Span

**MRINMAY BISWAS, ROBERTO A. OSEGUEDA, and JAMES S. NOEL**

ABSTRACT

The Texas Transportation Institute, under the joint sponsorship of the Texas State Department of Highways and Public Transportation and the FHWA, has undertaken a multiyear research program to investigate the strength and structural performance of a bridge deck system using full-depth precast concrete slab panel modules supported on steel stringers. The primary objective of the study is to evaluate the full range of behavior of a composite bridge deck system that uses epoxy mortar and shear studs as joint material for a modular construction using full-depth, full-width precast concrete panels with block-outs. Understanding this behavior would facilitate the use of such a method for rapid bridge deck construction and rehabilitation. The first phase of this program, a series of static load tests on a one-third scale laboratory model, has recently been completed. The design and detailed construction of the scale model, the details and results of the load tests, and the evaluation of the results are presented in this paper. The results indicate that, within the elastic stress range, the construction system described here would develop composite action in a satisfactory manner.

Use of full-depth precast concrete panels is a viable method of bridge deck construction and replacement. Such a method has been used since the early 1960s. More than a dozen transportation agencies have built at least twenty-five bridges using such a method. Many of the projects were for rehabilitation, and some were new constructions. Since 1973 many of these bridges have been constructed to provide composite action. Composite action is afforded, typically, by the use of mechanical shear connectors; or structural mortar based on epoxy, cement, or polymer; or both connectors and mortar. Except for some scattered and minor nonstructural failures, all the bridges are reported to be performing well.

In the case of noncomposite construction, the structural behavior of a span is not greatly dependent on the precast panel decks, although some incidental development of composite action has been reported. In the case of composite construction, on the other hand, structural behavior would be dependent primarily on the performance of the separate components as well as on that of the system as a whole.

Attempts to investigate structural behavior and strength of such construction have been rare. Because the rehabilitation and construction of structures on existing highways are done under extremely restrictive tactical constraints, a reluctance to engage in such research and investigation involving a bridge pressed for service is understandable. Because of the significance of such an investigation, the Texas State Department of Highways and Public Transportation, jointly with FHWA, has undertaken a research program conducted by the Texas Transportation Institute, Texas A&M University, involving laboratory tests, prototype construction, and subsequent tests of the prototype under field conditions. The first phase of this program, the design and construction of a one-third scale laboratory model and static load tests, has recently been completed. The progress and results of this research are described.

## DESIGN PROTOTYPE

To establish a basis for design of the model, a typical design of a two-lane, 60-ft nominal span bridge was selected from the Standard Drawings of Steel I-Beam Bridges of the Texas Highway Department (1). A typical cross section is shown in Figure 1(a). The stringers are old standard 36WF150 rolled sections, spaced 8 ft center to center. One signif-

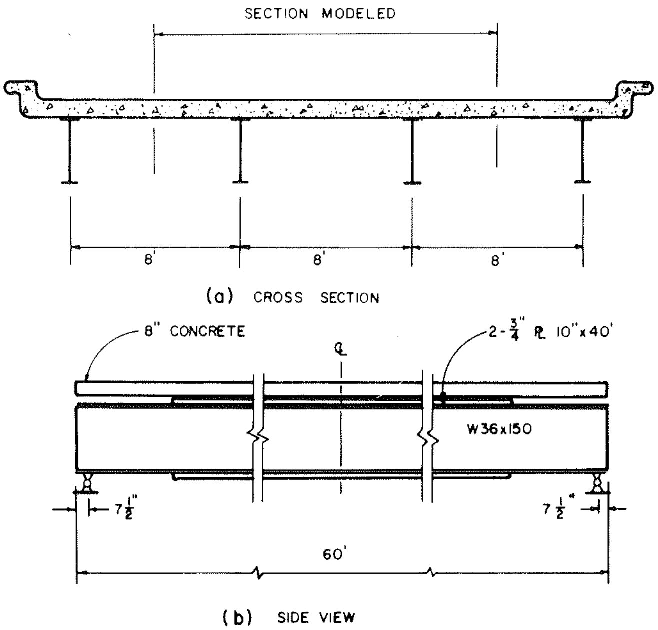


FIGURE 1 Schematic views of the prototype bridge (1).

icant feature is that welded cover plates, 3/4 in. thick, 10 in. wide, and 40 ft long, were used top and bottom.

It was assumed that the existing deck of such a bridge would be replaced by a series of full-depth and full-width, 8-in.-thick, precast panels, typically 6 ft long. Standard welded stud shear connectors and epoxy mortars would be used (2). The average nominal gap between the top of the stringer and the bottom of the precast panel would be 3/4 in. A side view of the prototype stringer and replacement panel is shown in Figure 1(b). It was further assumed that an isotropic reinforcement system consisting of same size bars, spaced equally both ways top and bottom, would be used for the precast slabs. Such reinforcement for cast-in-place decks is specified by the Ontario Bridge Design Code (3) and has been used experimentally by the New York State Department of Transportation (4).

## MODEL DESIGN AND COMPONENT CONSTRUCTION

### General Considerations

After consideration of the facilities available and review of the experience with models at other installations, a one-third scale was selected. Dead load and mass density effects were not included because the composite action is usually and primarily engaged to resist live loads only. This greatly simplified the design and loading scheme of the model. Necessary dimensional analyses were done to simulate the structural mechanics parameters in relation to live load only (5).

A 16-ft typical width of the prototype, including two interior stringers, was modeled. It was considered important to physically include cover plates in the model to simulate a realistic construction situation. A schematic layout of the model is shown in Figure 2. The values of sectional property parameters of the prototype, an ideal one-third scale model, and the calculated values of the actual model design are given in Table 1.

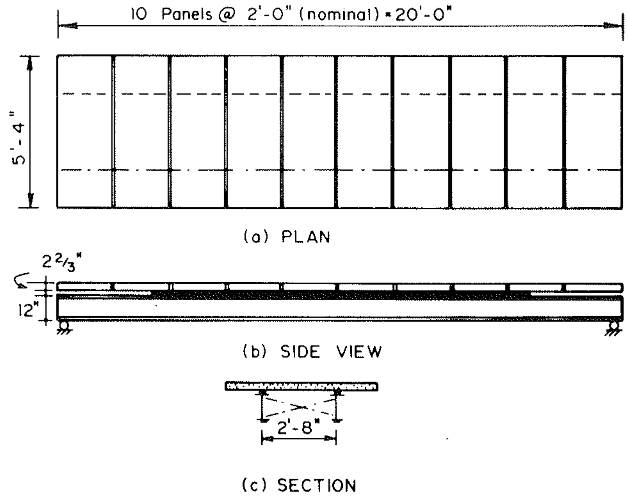


FIGURE 2 Layout of one-third scale model.

TABLE 1 Composite Sectional Properties of Prototype, a One-Third Ideal Model, and the One-Third Design Model: Middle and End Sections

| Section | Composite Sectional Parameter | Prototype | 1/3 Ideal Model | 1/3 Design Model |
|---|---|---|---|---|
| Midsection | I (in.$^4$) | 34,370 | 424.32 | 424.06 |
| | Q (in.$^3$) | 871.32 | 32.27 | 32.55 |
| | $S_{top}$ (in$^3$) | 2,878.08 | 106.04 | 105.92 |
| | $S_{bot}$ (in.$^3$) | 1,029.1 | 38.11 | 37.87 |
| | I/Q (in.) | 39.45 | 13.15 | 13.03 |
| End section | I (in.$^4$) | 25,778.7 | 318.26 | 324.78 |
| | Q (in.$^3$) | 713.77 | 26.43 | 27.08 |
| | $S_{top}$ (in.$^3$) | 2,453.7 | 90.88 | 91.37 |
| | $S_{bot}$ (in.$^3$) | 756.32 | 28.01 | 28.34 |
| | I/Q (in.) | 36.12 | 12.038 | 11.99 |

Note: I = moment of inertia, Q = horizontal shear, S = section modulus.

### Model Stringers

The computed sectional properties of the stringers could ideally be obtained by welding plates cut to precise dimensions. However, a number of fabricators indicated that this would be impractical because proper alignment could not be maintained. Compromise design was reached by using W 12 x 19 beam sections modified in the following manner: Cover plates, 3/16 in. thick, 2-3/4 in. wide, and 13 ft 4 in. long, were welded top and bottom. Both sides of top and bottom flanges at each end (total of 8 locations per stringer) were coped by grinding away 3/8 in. of the edges of the last 34-1/2 in. The design of the model stringers is shown in Figure 3.

To simulate realistic construction, available standard headed studs, 1/4 in. in diameter and 2-1/2 in. long, were used. The shear studs were placed in pairs, with a lateral spacing of 1-3/4 in. and a longitudinal spacing of 6 in. center to center through the whole length of the stringer. The calculated horizontal shear strength of the system was designed to match the ultimate flexural strength of the composite model stringer when subjected to a third-point loading.

### Model Precast Concrete Panel Modules

The overall dimensions of a typical, full-depth,

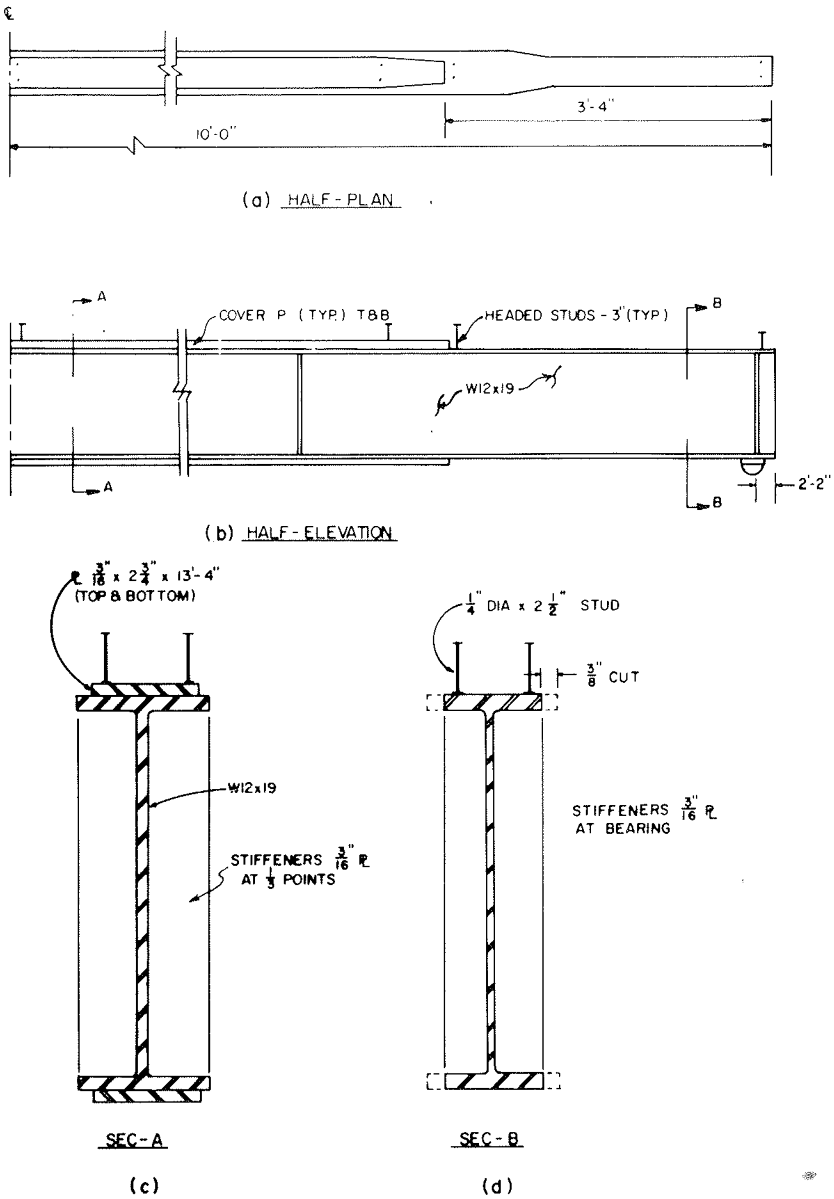


FIGURE 3 Details of model stringers.

one-third scale precast concrete model panel is shown in Figure 4(a). Ten such identical panels were used in series to form the deck of the model span. The blocked-out holes were designed and positioned to fit over the headed studs that had been previously welded on the model stringers. Figure 4(b) shows the details of the transverse joint between adjacent slabs.

Formwork

Clear acrylic sheets, 1/4 in. thick, were selected based on the experience of other researchers. The material can be cut precisely using standard woodworking equipment. It allows visual inspection of the underside and sides of the form. No bond release agent is needed. Smooth clean surfaces are obtained, which are amenable to bonding to epoxy mortar subsequently used at shear keys. The form does not absorb water, and it eliminates bleeding. The completed forms are light and can be easily handled. To facilitate vibration, the plastic formwork was placed on a plywood frame, which in turn was placed on a table-type vibrator. In addition, two pneumatic ball vibrators were used at each end of the plywood frame to assure consolidation of fresh concrete evenly throughout the form.

Reinforcing Steel

Deformed reinforcement under No. 3 bar size is not available. Welded-wire fabric (3 x 3--D3 x D3) was used as top and bottom reinforcement for the precast slab panels. This essentially eliminated all problems related to modeling the reinforcement. The mesh was equivalent to deformed 0.195-in.-diameter bars, placed at 3.0 in. center to center both ways. The steel area provided is 0.38 percent of the gross cross-sectional area of the slab. This is less than the conventional steel requirement for transverse flexure, and it slightly exceeds the usual longitudinal distribution steel requirements. The isotropic

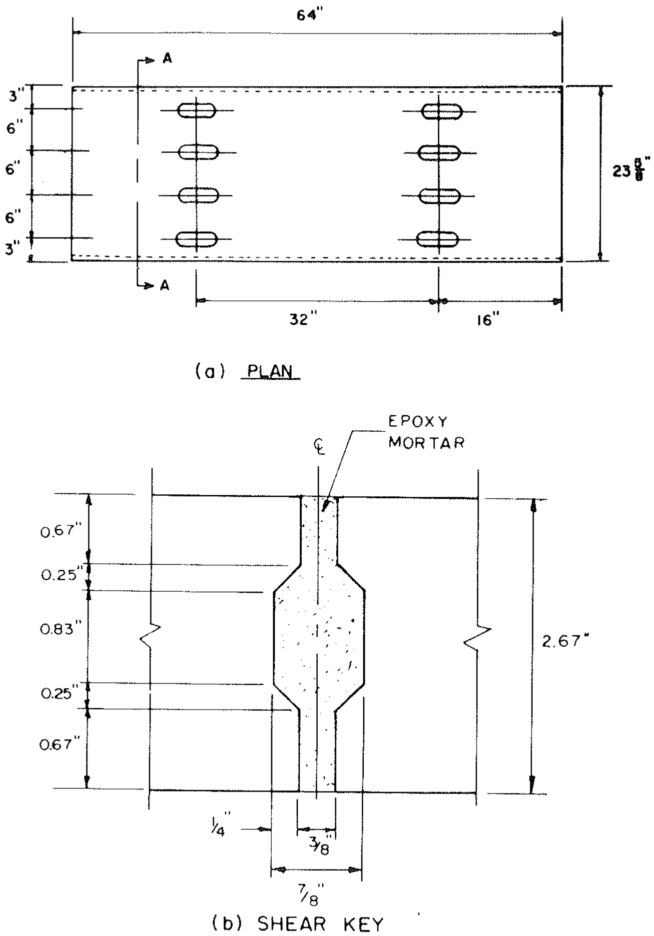


FIGURE 4 Details of model precast concrete panel.

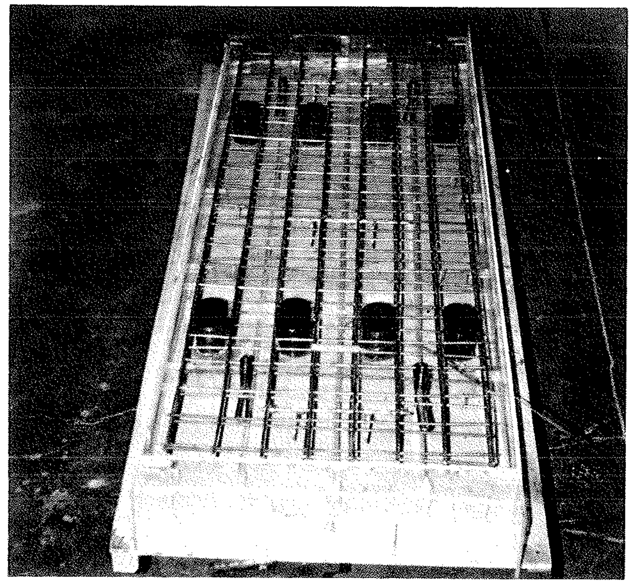

FIGURE 5 Acrylic plastic form including welded wire isotropic reinforcement and block-out molds.

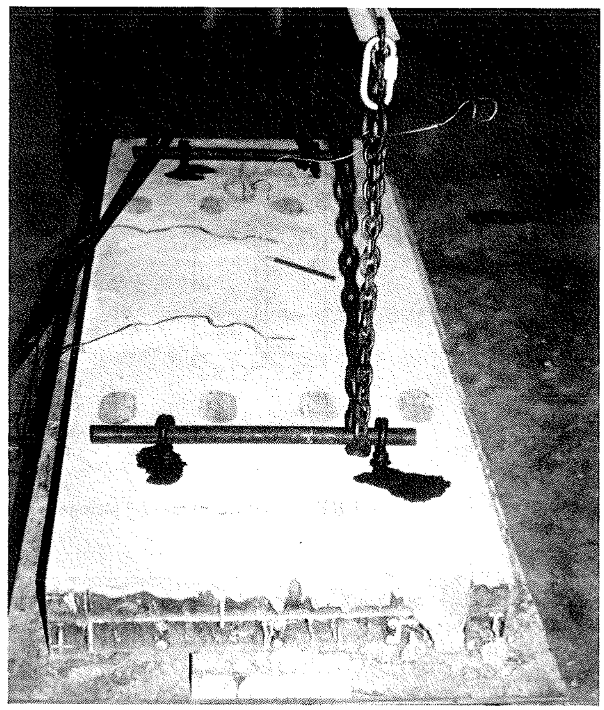

FIGURE 6 Concrete panel after casting.

reinforcement provided, however, exceeds the requirements of the specifications of the Ontario Highway Department and the practice of the New York State Department of Transportation.

Model Concrete and Slab Casting

To model the concrete to the extent possible, nominal 3/8-in.-max pea gravel, passed once through a 1/4-in. sieve, was chosen as the coarse aggregate and fine masonry sand was selected as the fine aggregate. The gradation of the coarse and fine aggregate was mostly within the geometrically scaled down limits of AASHTO requirements. Because of the presence of a larger amount of fines than is found in usual concrete, a larger amount of water was needed to obtain proper workability, and consequently a relatively large cement factor of 9 bags per cubic yard was needed to obtain the necessary strength.

Figure 5 shows the acrylic sheet formwork, the welded wire re-bar cage, and the block-out molds. Figure 6 shows a typical cast panel. One panel was cast every 3 days. The panels were cured using wet burlap. Test cylinders 2 in. x 4 in. and 4 in. x 8 in. were also cast. They all tested at over 6,000 psi in compression at 28 days.

Epoxy Mortar

Epoxy Compound

Texas Highway Department (THD) standard epoxy binder B-102 was selected. It has customarily been used as a binder in a patching compound to repair existing concrete structures. It was not formally classified in accordance with the recent ASTM C-881 specifications. It, however, appeared to be similar to the ASTM type III, grade 1, class C epoxy.

Aggregate

A compromise was made between using graded silica sand and common construction sand. Commercially available sand blasting sands, namely Texblast grades No. 2, No. 3, and No. 4, were considered. On the basis of sieve analyses, a 50-50 percent blend of Texblast No. 2 and No. 4 was selected. This blend meets the requirements of THD grade No. 1 aggregate that is specified for mixing with B-102 epoxy compound (6).

### Mix Design

The design of epoxy mortar mix was based on needed workability. Trial batches were made with sand-to-epoxy weight ratios of 2.75 through 3.50 at increments of 0.25.

A sand-to-epoxy weight ratio of 3 to 1 was selected to produce a trowelable mix that would be used to cast the pockets around the pairs of shear studs. A weight ratio of 2.75 to 1 was selected to produce a flowable mix that would be poured into the transverse keyways between adjacent slab panels.

Cylindrical test specimens 2 in. x 4 in. were cast. The material exhibited consistently high strength of about 12,000 psi in compression and about 1,500 psi in (split cylinder) tension when tested at about 24 hours after casting. The average tangent modulus was about 1.4 ($10^6$) psi.

## MODEL ASSEMBLY AND LOAD TESTS

The prefabricated components of the model were moved by truck to a site about 10 miles away for assembly and subsequent load testing. This simulated a realistic construction situation. Composite action develops in steps that follow the sequence of placement of slab panels and the subsequent grouting of various joints. The test sequences were designed to monitor such stepwise development of the composite action.

### Loading System and Instrumentation

The layout of the load points and the key to locations of various sensors are shown in Figure 7.

### Loading

A third-point loading was used with four identical RC-250 Blackhawk hydraulic rams. The rams were connected to a single pump through a four-way manifold. Calibrated load cells were used under each ram to monitor equal load application. Properly designed and fabricated AASHTO grade elastomeric bearing pads were placed between the loading devices and the top of the model.

### Strain Gauges

Longitudinally oriented uniaxial strain gauges were placed at cross sections 1 through 8 (Figure 7) to measure flexural deformations. Surface bonded gauges were used at the flange and web of the stringers and on the underside of the concrete panels. In addition, small strain gauges were bonded to the reinforcement bars, and large capsulated concrete strain gauges, to be embedded in the precast concrete panels, were placed in the formwork. Strain gauge rosettes were placed at four sections, A through D, (Figure 7) to measure shear deformations.

### Dial Indicators

A total of six dial indicators were used, placed at the quarter points on the underside of each stringer. A reference beam was used to measure relative deflection as a function of load.

### Displacement Transducers

Four electrical displacement transducers were used to measure the relative horizontal slip displacement between the bottom of the precast panel and the top of the stringer. One transducer was placed 18 in. (longitudinally) away from each end of the two stringers. A typical installation is shown in Figure 8. In all there were 4 load locations and a total of 103 sensor locations.

### Test 1: Noncomposite Stringer

The purpose of this test was to validate the loading system, the instrumentation system, and the calculated structural properties of the steel stringers. Only two precast panels were placed on top of the stringers, over a strip of elastomer, to provide horizontal loading planes at a suitable height under the rams. Figure 9 shows the setup of test 1. A maximum load of 5 kips per ram was applied. Figure 10 shows a flexural normal strain distribution of a typical section within the middle-third length. Figure 11 shows shear strain distribution at a section near the support. Midspan deflection as a function of load per ram is shown in Figure 12. The average flexibility is 0.094 in. per 1 kip third-point load per ram, or the average stiffness is 10.638 kips/ram/1 in. of midspan deflection.

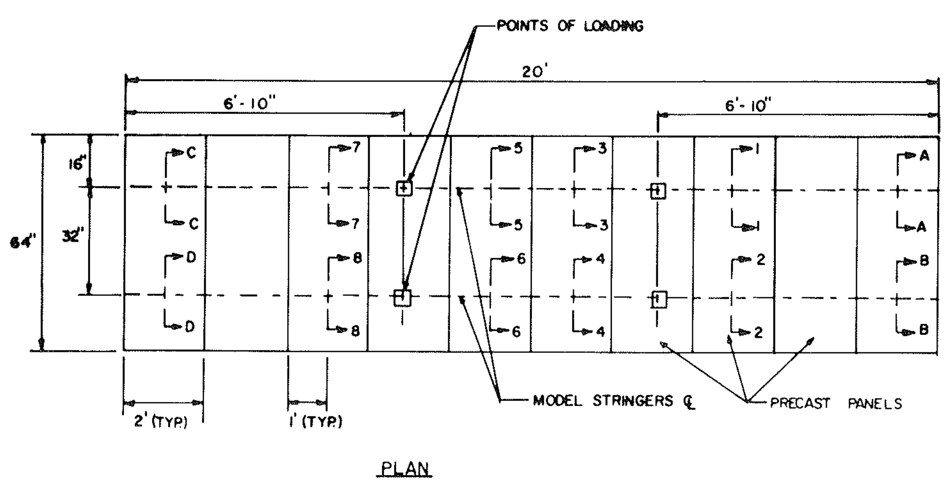


FIGURE 7 Key to location of instrumented sections.

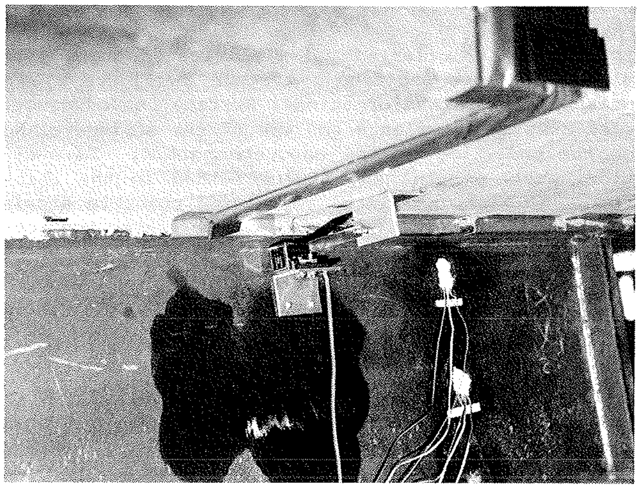

FIGURE 8 Typical displacement transducer installation to measure horizontal slip.

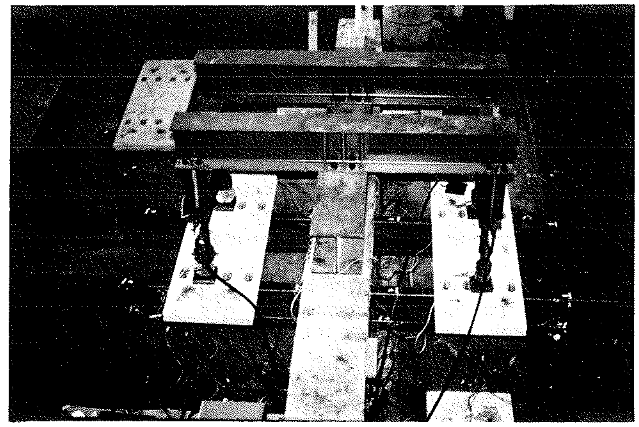

FIGURE 9 Setup for test 1 (noncomposite).

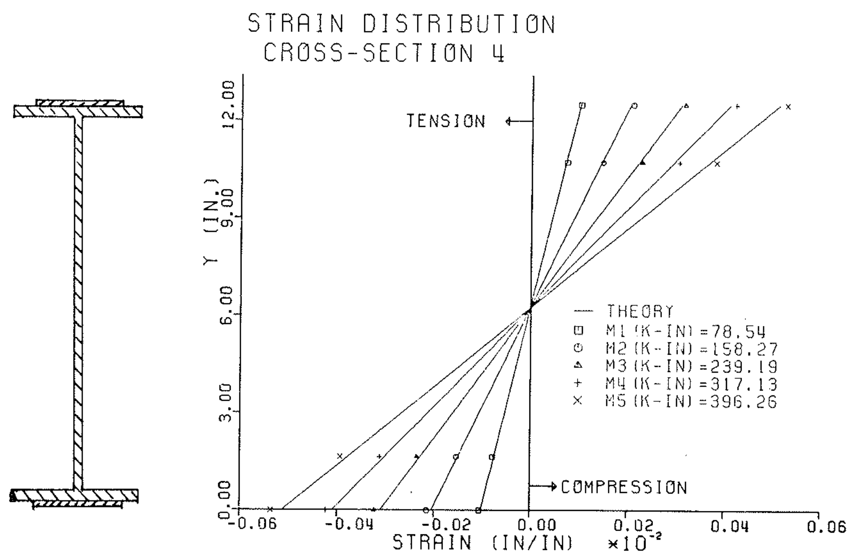


FIGURE 10 Flexural strain distribution at section 4-4: test 1.

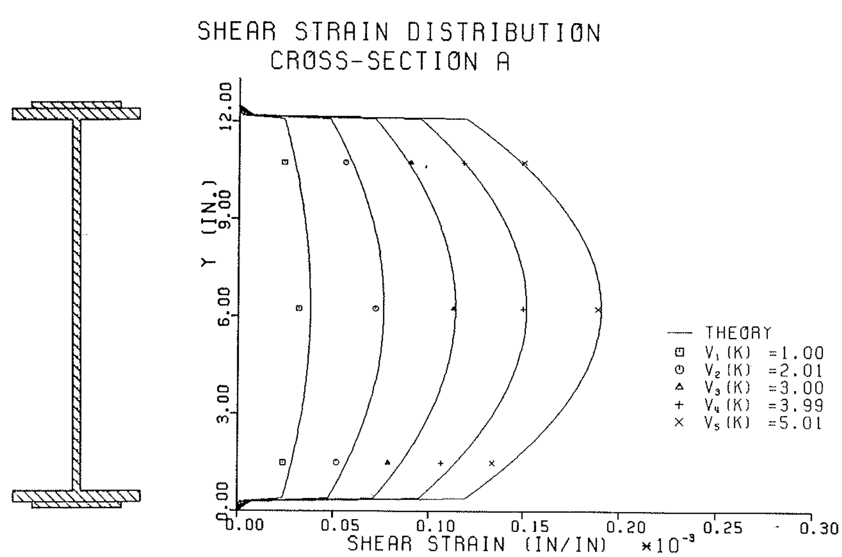


FIGURE 11 Shear strain distribution at section A-A: test 1.

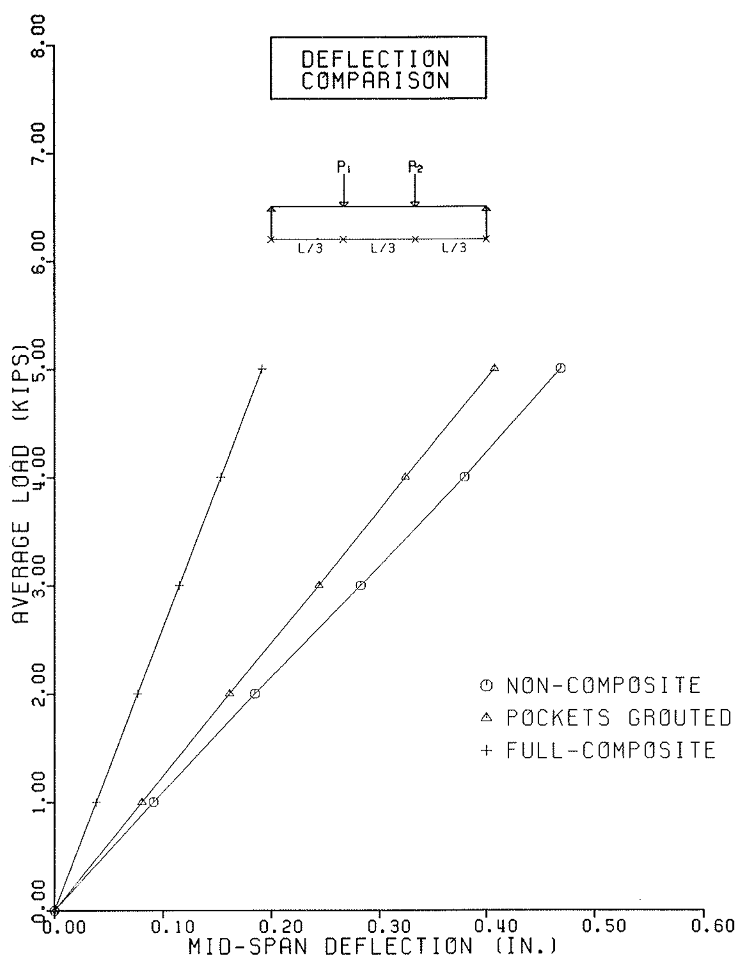


FIGURE 12 Comparison of midspan deflections for tests 1, 2, and 3.

Test 2: Partial Composite Stringer

For this test, only the pockets around each pair of shear connectors were grouted. The keyways between adjacent slab panels were left ungrouted. This simulates a situation where a bridge might need to be briefly opened to traffic in between stages of construction. To isolate the effect of incidental development of composite action due to adhesion of epoxy mortar bedding at the stringer and slab interface, the areas of the top of the flange in between the pockets were dammed using peel-back foam insulation strips. These also served as bearings for the precast panels and provided the necessary gap between the stringer and the panels. This arrangement is shown in Figure 13. The load test (Figure 14) was conducted 24 hr after grouting the pockets and a maximum load of 5 kips per ram was attained. The midspan deflection as a function of load per ram is shown in Figure 12. The average flexibility is 0.08 in. per 1 kip of third-point load per ram, or the average stiffness is 12.5 kips/ram/1 in. of midspan deflection. This indicates a 17.5 percent increase in stiffness due to grouting only the pockets.

Test 3: Full Composite Stringer

For this test all the transverse joint keyways between the adjacent slab panels were filled with a flowable epoxy mortar. Peel-back foam insulation strips, backed by 1 in. x 2 in. furring lumber, were used to seal all the potential leak locations. Figure 15 shows the deck after grouting all the joints. The load test was conducted, 24 hours after grouting of the transverse keyways, to a maximum of 6 kips per ram or a maximum total load of 24 kips for the model span. Readings were taken at intervals of 1-kip load per ram. Figure 16 shows the flexural normal strain distribution at a typical middle-third region of the span. Figure 17 shows the shear strain

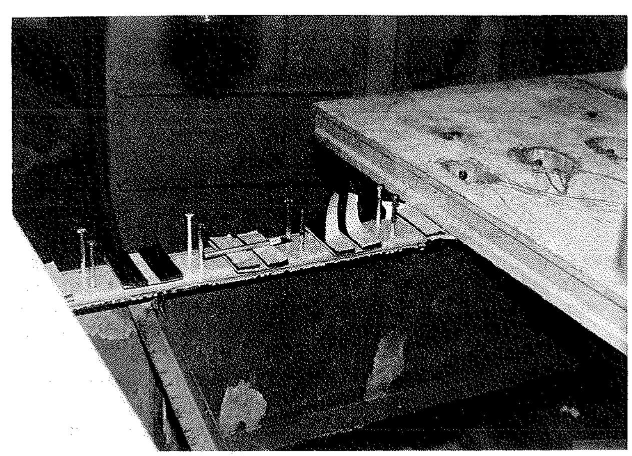

FIGURE 13 Peel-back foam insulation used to dam epoxy mortar and as bearings for precast panels.

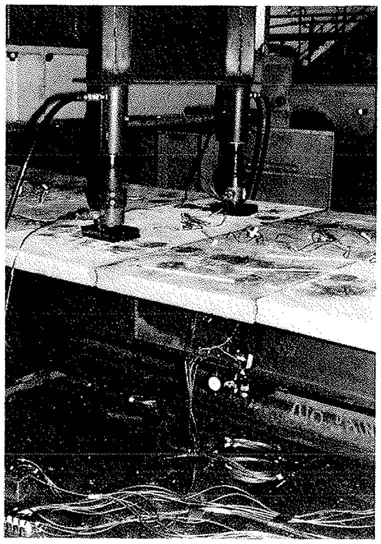

FIGURE 14 Setup for test 2 (partially composite).

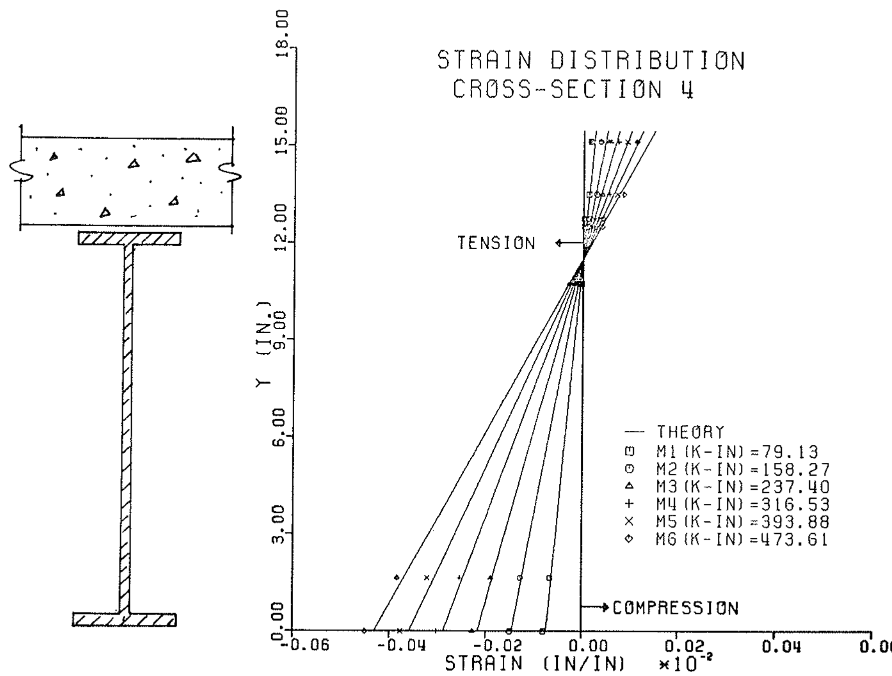


FIGURE 16 Flexural strain distribution at section 4-4: test 3.

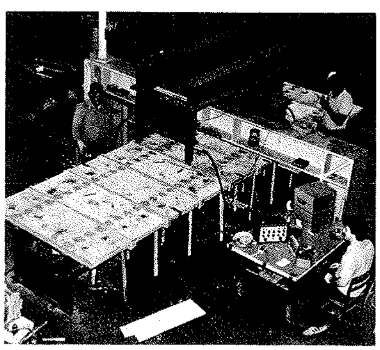

FIGURE 15 The one-third scale model after completion of all grouting before test 3 (fully composite).

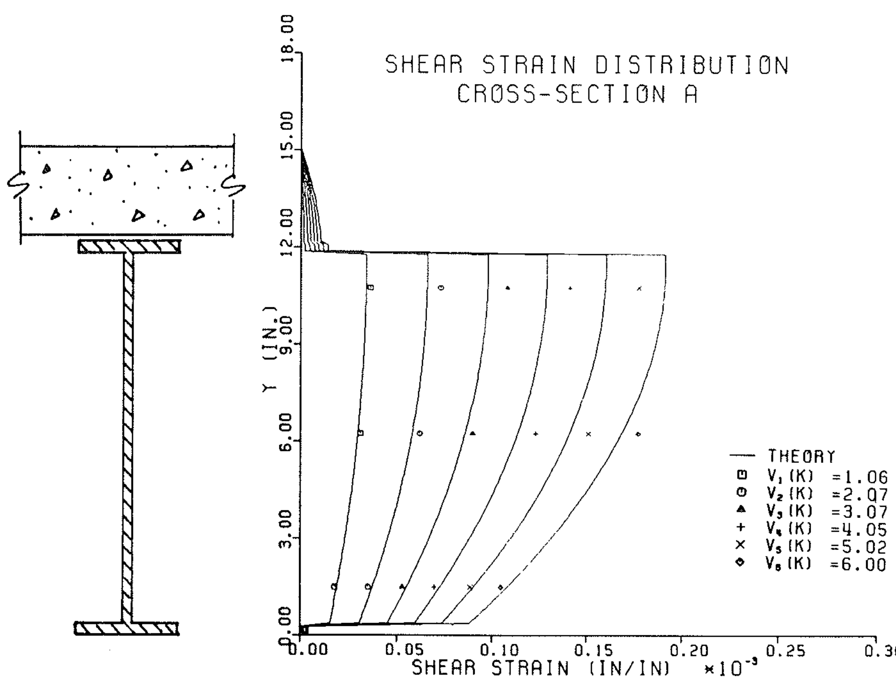


FIGURE 17 Shear strain distribution at section A-A: test 3.

distribution at a typical section near a support. The midspan deflection as a function of load per ram is shown in Figure 12. Average flexibility is 0.037 in. per 1 kip of third-point load per ram, or an average stiffness of 27 kips/ram/1 in. of midspan deflection. This is about a 155 percent increase in stiffness over the noncomposite section.

The maximum horizontal slip, measured at the displacement transducers, was about 0.001 in. A nearly linear relation between load and slip displacement was observed.

## Summary of Results

The various test results have been reduced and compared with calculated theoretical values whenever possible (5). Generally good correlations have been obtained.

Figure 18 illustrates the advantages achieved by grouting the shear connectors to get composite action. A 40 percent increase in the live load carrying capacity is realized because of composite action.

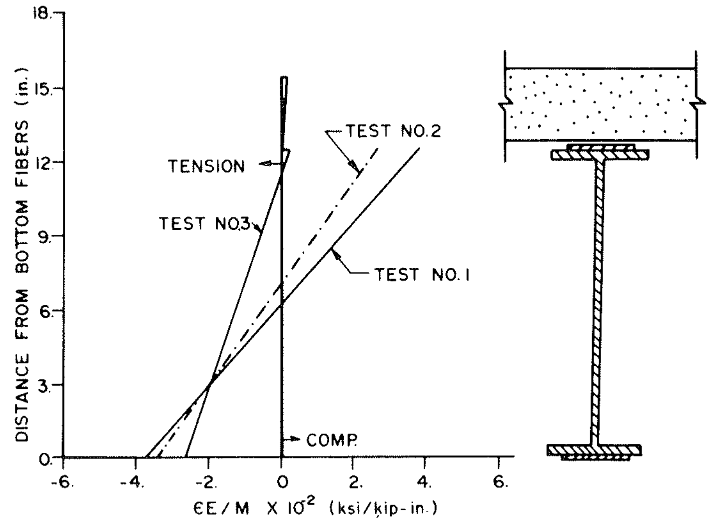


FIGURE 18 Comparison of flexural stress distribution for tests 1, 2, and 3.

Some of the results were scaled up to compare with the corresponding calculated values of the prototype. Figures 19 and 20 show the comparisons of normal strain at the bottom of a steel stringer and the (linearly projected) strain at the top of concrete, respectively. Figure 21 shows the comparison of midspan deflection.

Horizontal shear deformation or slip displacement was less than estimated. This may be due to adhesion of epoxy mortar at the shear pocket locations.

## CONCLUSIONS

In concluding the first phase of this experimental investigation the maximum applied load was limited to keep the steel stresses within elastic limit without failing the horizontal shear transfer system. A stepwise development of composite action corresponding to modular construction sequence was observed. After the completion of construction using the full-depth precast panels, satisfactory composite action developed. Load-deformation relations, in general, were linear. Excellent correspondence between model behavior and calculated prototype behavior was obtained. For the prototype considered in this investigation the results indicated nearly 50 percent over load capacity com-

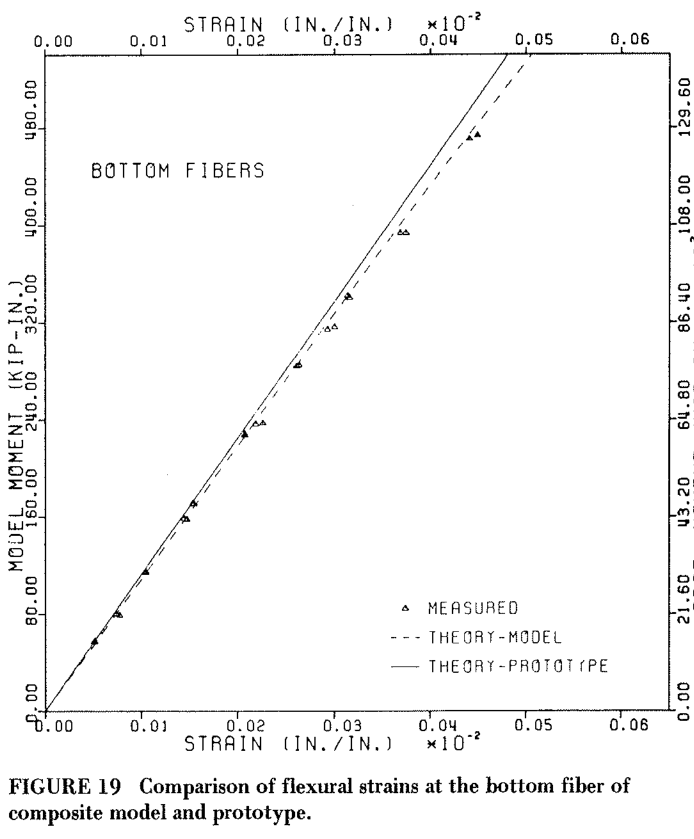


FIGURE 19 Comparison of flexural strains at the bottom fiber of composite model and prototype.

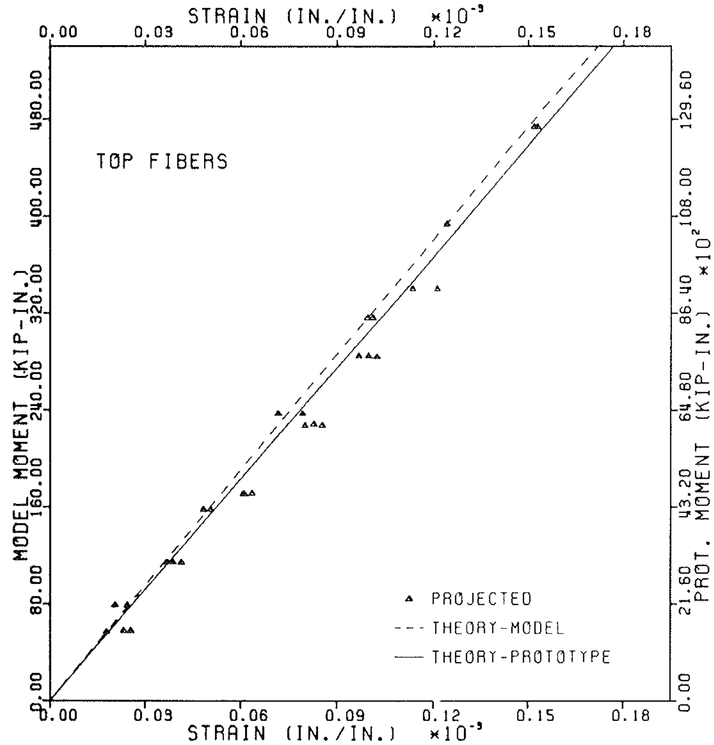


FIGURE 20 Comparison of flexural strains at top fiber of concrete deck panel of composite model and prototype.

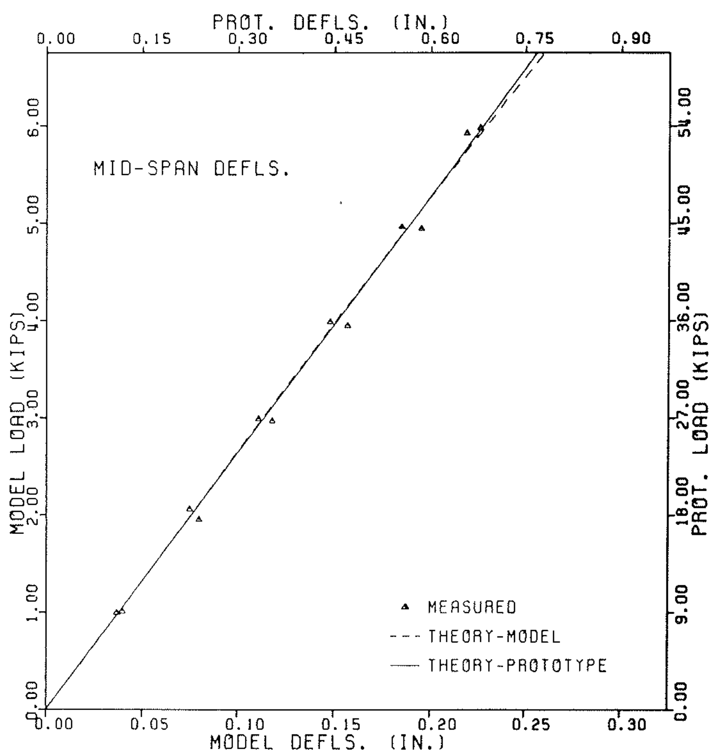


FIGURE 21 Comparison of midspan deflection of composite model and prototype.

pared to live load effects of an HS20-44 design truck.

The model is still intact and has been moved to a new testing facility about 10 miles away. Many other experiments to investigate other structural characteristics are anticipated.

ACKNOWLEDGMENT

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# Ohio Turnpike Cuyahoga River Bridge Rehabilitation

**WILLIAM S. FREEH**

ABSTRACT

Opened to traffic in 1955, the twin two-lane Ohio Turnpike bridges over the Cuyahoga River valley span 2,650 ft and reach as high as 175 ft above the valley floor. Each bridge is comprised of four 100-ft-long girder spans and nine 250-ft-long truss spans supported by 12 reinforced concrete piers. As the use of deicing salts increased during the 1960s, so did deterioration of the concrete portions of the bridges. The original design permitted salt water to flow directly onto the surfaces of the piers. By the mid-1970s deterioration of the piers became evident. Efforts to patch the piers and divert drainage were made, but the piers had already become so saturated with chlorides that deterioration continued. In 1980, under contract to the Ohio Turnpike Commission, Howard, Needles, Tammen & Bergendoff inspected the piers and found that about 40 percent of their surface area was spalled or near spalled. Subsequently the firm recommended methods of repair to prevent recurrence of the condition, prepared plans and specifications for shotcreting (selected alternative), and provided resident construction inspection.

The Ohio Turnpike, opened to traffic in 1955, was designed and constructed before serious consideration was given to mitigating the potentially destructive effects of deicing salts. The twin, two-lane bridges over the Cuyahoga River valley are the longest on the turnpike, spanning 2,650 ft and reaching as high as 175 ft above the valley floor. Each bridge is comprised of four 100-ft-long girder spans and nine 250-ft-long truss spans supported by 12 reinforced concrete piers. The concrete decks, when originally constructed, had open curbs for drainage.

As the use of deicing salts increased during the 1960s, so did deterioration of the concrete portions of the bridges. The deck, where the reinforcing steel is close to the surface, and the edges of the deck slab, where the salt-laden water flowed through the open curbs, were the first areas to show severe spalling. The deck was patched with relative ease, but patching the vertical edges of the slab outside the railing was difficult. To prevent salt water flowing over the fascias, the open curbs were closed in 1967 and all drainage from the decks was diverted through open toothplate-type expansion joints located above all but two of the piers. Joints were located 25 ft from piers 4S and 4N to prevent drainge falling on railroad tracks passing close to these piers.

The piers had been subjected to some salt water flowing through the joints since the first use of deicing salts, but the closing of curb openings increased the flow directly onto the surfaces of the piers. By the mid-1970s there was visible deterioration of the piers (except 4S and 4N). There were some efforts by Ohio Turnpike Commission (OTC) maintenance forces in later years to patch the piers and to divert drainage away from them, but the piers had