TEST TO FAILURE OF A PRESTRESSED CONCRETE BRIDGE

Edwin G. Burdette
Associate Professor of Civil Engineering
The University of Tennessee
Knoxville, Tennessee

David W. Goodpasture
Associate Professor of Civil Engineering
The University of Tennessee
Knoxville, Tennessee

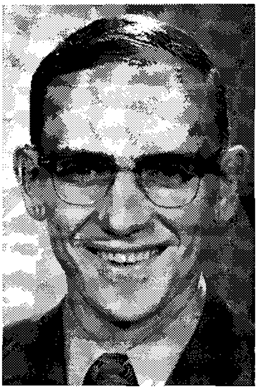
The test to failure of two spans of a simple span, prestressed concrete highway bridge is described, and the measured and computed load-deflection curves for the bridge are presented and compared. The calculated load-deflection curve, based on strain compatibility relationships, compares reasonably well with the curve obtained experimentally.

Severe dishing of the deck slab led to a separation between the bridge deck and the girders; the result was a loss of composite action and subsequent shear failure of the girders.

Questions are raised regarding the state of existing knowledge of the behavior of composite, prestressed concrete, multi-beam bridges.

Four highway bridges, located in Franklin County, Tennessee, were tested to failure during the summer of 1970. These bridges were located in an area which has since been inundated by the Tennessee Valley Authority's Tims Ford Reservoir. Since the test bridges had been replaced by new bridges at higher elevations, they were made available by TVA and the Tennessee Department of Transportation, Bureau of Highways, for unlimited testing purposes.

The research effort was carried out



Edwin G. Burdette

David W. Goodpasture

by personnel from The University of Tennessee's Civil Engineering Department as a part of a research project sponsored by the Tennessee Department of Transportation, Bureau of Highways, in cooperation with the Federal Highway Administration.

A complete description of the testing program, along with test results, is given in the *Final Report* for the project.[1] Briefer descriptions of the testing program, with specific emphasis on the static tests to failure, are found in References 2 and 3. This paper deals exclusively with the static test to failure of the prestressed concrete bridge.

Fig. 1. Test bridge.

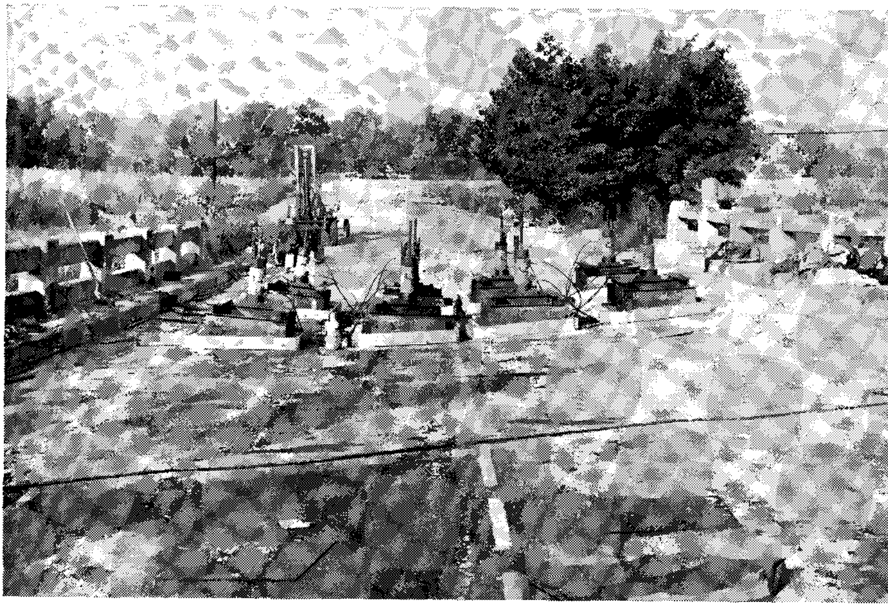


Fig. 2. Test bridge showing loading devices in place.

The specific objective of the paper is two-fold: (1) to describe, in some detail, the behavior of the bridge as it was loaded from zero load to failure and (2) to compare the measured load-deflection behavior of the bridge with that computed on the basis of strain compatibility relationships.

The opinions, findings, and conclusions expressed herein are those of the writers and not necessarily those of the State of Tennessee or the Federal Highway Administration.

DESCRIPTION OF BRIDGE

The test bridge consisted of four, Type III, AASHO-PCI precast girders and a 7 in. (18 cm) thick cast-in-place slab. Photographs of the bridge are

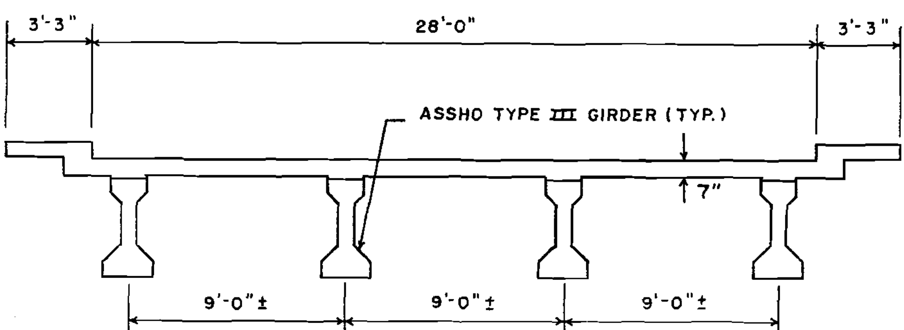


Fig. 3. Bridge cross section.

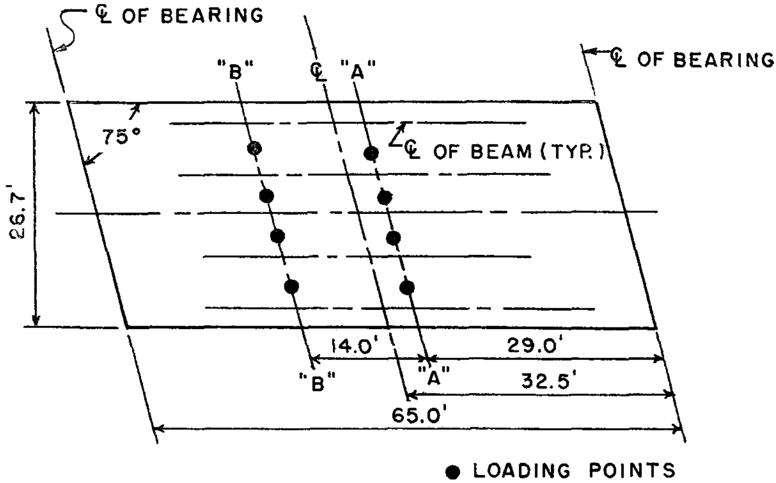


Fig. 4. Plan of bridge showing position of loads.

shown in Figs. 1 and 2 and an idealized cross section is given in Fig. 3.

There are four, 66 ft (20 m) simple spans. The bridge had a 70-deg skew, was located on a grade of approximately 4.5 percent, and had a superelevated roadway due to a 4½-deg horizontal curve. Due to the latter features, the bridge was something less than the ideal test specimen.

The bridge was designed in 1963 for an AASHO HS 20 loading. The test loading system chosen was designed to simulate the eight rear wheels of two HS 20 trucks, one truck in each traffic lane.[1,2,3] The loads were placed on the two spans tested in the position resulting in maximum positive moment near the center of the span. A sketch showing the load placement is given in Fig. 4.

Stirrups (No. 3 bars) spaced at 15 in. (38 cm) on center extended from the precast girders into the cast-in-place deck slab. These stirrups crossing the interface between girder and deck, along with the bond between the two, were counted on to assure composite action between girder and slab.

TEST PROCEDURE

The load was applied to the bridge at each of the eight load points referred to earlier, each load point representing one of the eight rear wheels of the two simulated HS trucks. The loads were applied incrementally through the use of eight hydraulic rams jacking against eight rock anchors. All eight jacks were connected to a common reservoir, so that the pressure was the same at each ram. The loads were distributed to the deck at each load point through a "bearing grill," consisting of a rigid grillage of wide flange beams and channels. The loading system is described more fully in References 1-3.

Two spans of the bridge were tested. The first span was not tested to failure due to problems of punching shear around one of the bearing grills at a load point. Steps were taken, as described later, to eliminate this problem

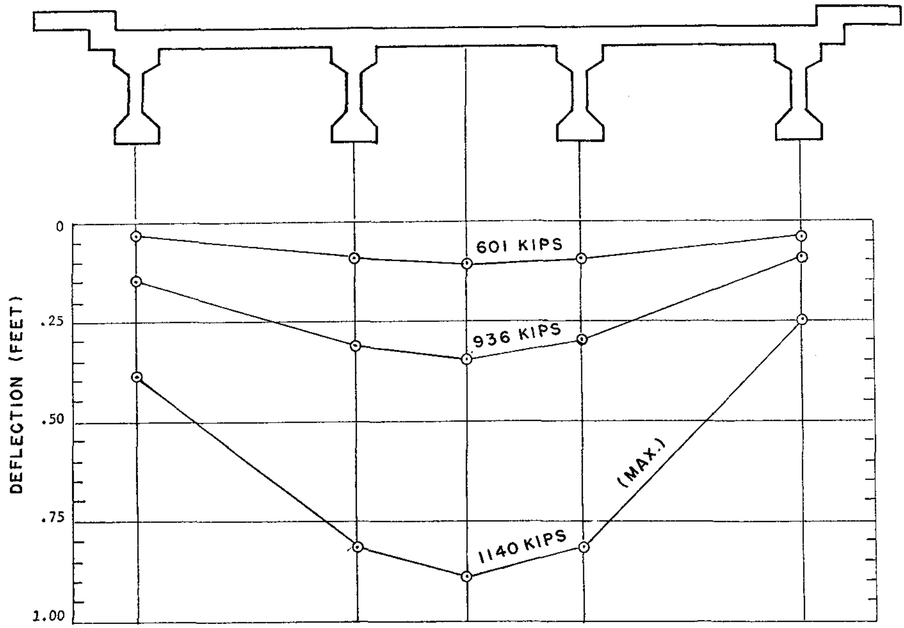


Fig. 5. Cross section plots showing deck distortion under load.

in the second test, and the test was completed to failure. It should be noted that "failure" is defined herein as the condition which exists when an increase in deflection of the bridge takes place under a decreasing load.

TEST RESULTS

Span 1

This span behaved in a predictable way up to a load of approximately 950 kips (4230 KN). At this load level, cracks between the precast girders and the cast-in-place deck were apparent, and diagonal cracks began to form in the girders near the abutment. The diagonal crack formation was audible as well as visible. It appeared that composite action had been lost and that shear failure of the precast girders was imminent. However, as the total load was increased to 978 kips (4350 KN), a local punching shear failure occurred around one of the bearing grills, and the test was terminated.

Span 2

Concrete bearing pads were cast on the deck of this span to provide a horizontal surface against which the bearing grills would apply load. These pads obviated the possibility of punching shear and considerably reduced bending in the rock anchor loading system. As a result, it was possible to test this span to failure.

This span behaved in a similar way to the first span tested. Visible cracking of the center diaphragm occurred at a bridge load of 433 kips (1930 KN). Cracking of interior girders at the section of maximum moment were visible at a load of 521 kips (2320 KN), one load increment (88 kips) above that at which cracking was detected by pulse

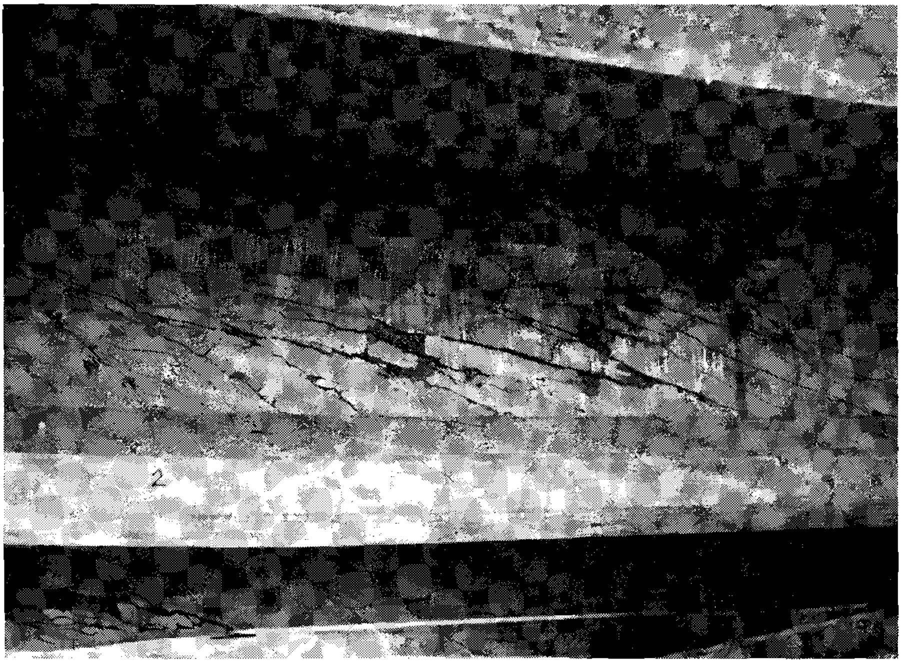


Fig. 6. Diagonal cracking in web at failure (Span 2).

velocity measurements.[1] The overall load-deflection behavior of the bridge was not measurably affected by the cracking of the two interior girders at the maximum moment section.

At a load of approximately 950 kips (4230 KN), considerable "dishing" of the bridge had occurred, with the interior girders being deflected considerably more than the exterior. This "dishing" is illustrated in Fig. 5. The result of the dishing was a tendency for the bridge deck to separate from the interior precast girders. Under the action of this 950-kip load, separation occurred and composite action of the interior girders was lost as the vertical stirrups crossing the interface between girder and deck were sheared.

After composite action was lost, the behavior of the bridge was radically changed. Almost immediately there was crushing of the extreme top fibers of the interior precast sections at the section of maximum moment. This crushing and accompanying rotation resulted in a redistribution of moments at the section, with the moment in the exterior girders being increased. As the load was increased above 950 kips, diagonal tension cracks began to form and propagate in the girder webs, as illustrated in Fig. 6. The formation of these cracks was accompanied by loud noise. Finally, as the load was increased to 1140 kips (4620 KN), the interior girders failed in shear, and the test was terminated. The failed condition of one of the girders is illustrated in Fig. 7.

Load-deflection curves for the two

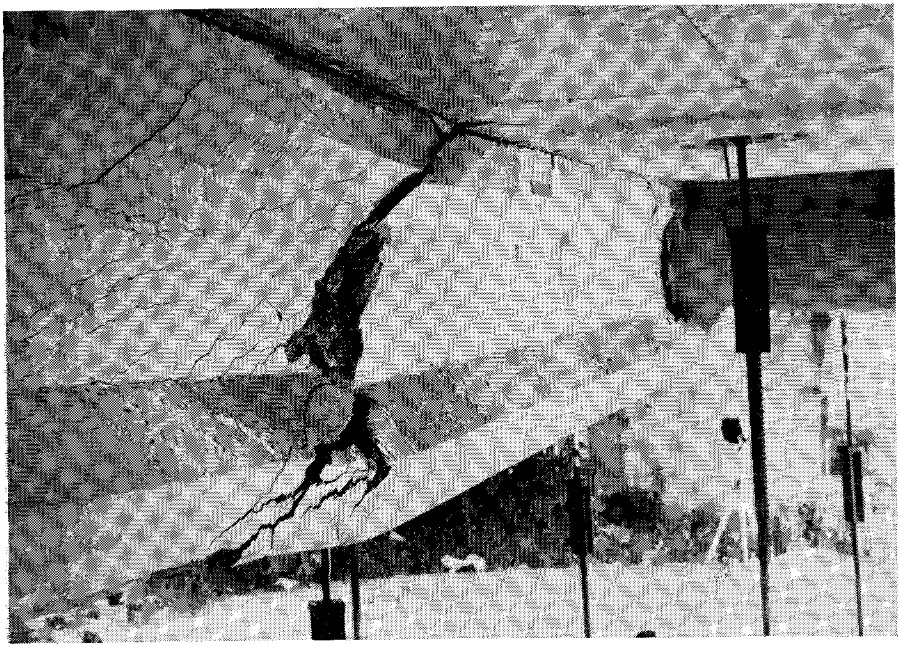


Fig. 7. Girder in Span 2 after shear failure has occurred.

spans are shown in Figs. 8 and 9. The points on each deck at which deflection was measured are indicated in the figure titles.

COMPARISON OF MEASURED AND COMPUTED LOAD-DEFLECTION RELATIONSHIPS

The computed load-deflection relationships are compared to those measured in the test of Span 2, as load-deflection data are available for this span from zero load to failure. The average of the deflection values measured on the deck at the four points directly above the four girders at the centerline of the span was plotted versus total bridge load; this curve was used as the measured load-deflection curve for comparison purposes.

Material properties

Four inch (10.2 cm) diameter cores were taken from the bridge deck by personnel from the Tennessee Bureau of Highways in order to permit the installation of rock anchors. These cores were tested to obtain the compressive strength of the concrete. Forty-eight cores were tested to obtain an average strength of 6280 psi (43,300 KN/m^2). The average height of the cores was 7.0 in. (17.8 cm), and the strength value of 6280 psi includes a small correction to relate the strength measured to that based on a 2:1 ratio of height to diameter. Thus, 6280 psi was used as f_c'.

In order to calculate points on a

moment-curvature curve for the bridge, other than the point at ultimate, it was necessary to know or to assume a stress-strain curve for the concrete. An idealized stress-strain curve for the concrete in the bridge is shown in Fig. 10. It was obtained as follows:

1. The slope of the elastic portion of the curve was E_c, the modulus of elasticity of concrete as calculated from the ACI 318-71 Building Code.

2. The maximum compressive stress attainable in flexure was considered to be 0.85 f_c', consistent with ACI 318-71. This stress was considered to be independent of strain in the inelastic range, as illustrated in Fig. 10.

It should be noted that the idealized stress-strain diagram described in the previous paragraph was not used to calculate "ultimate" moment and curvature. The latter were calculated on the

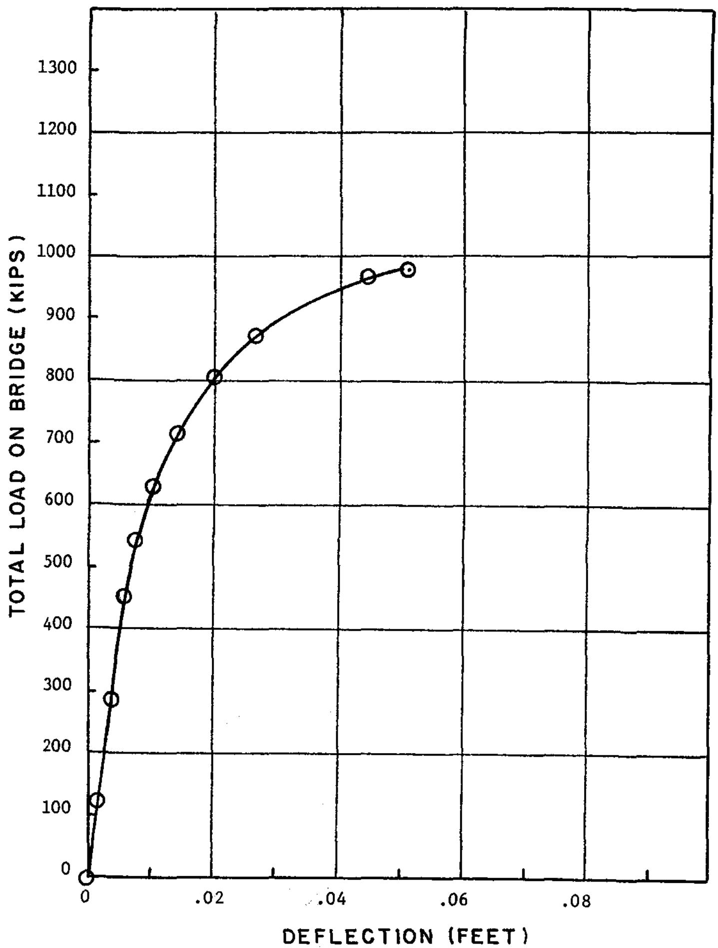


Fig. 8. Load-deflection curve; centerline of span at center of roadway; Span 1—Illustrating behavior of bridge from zero load to failure.

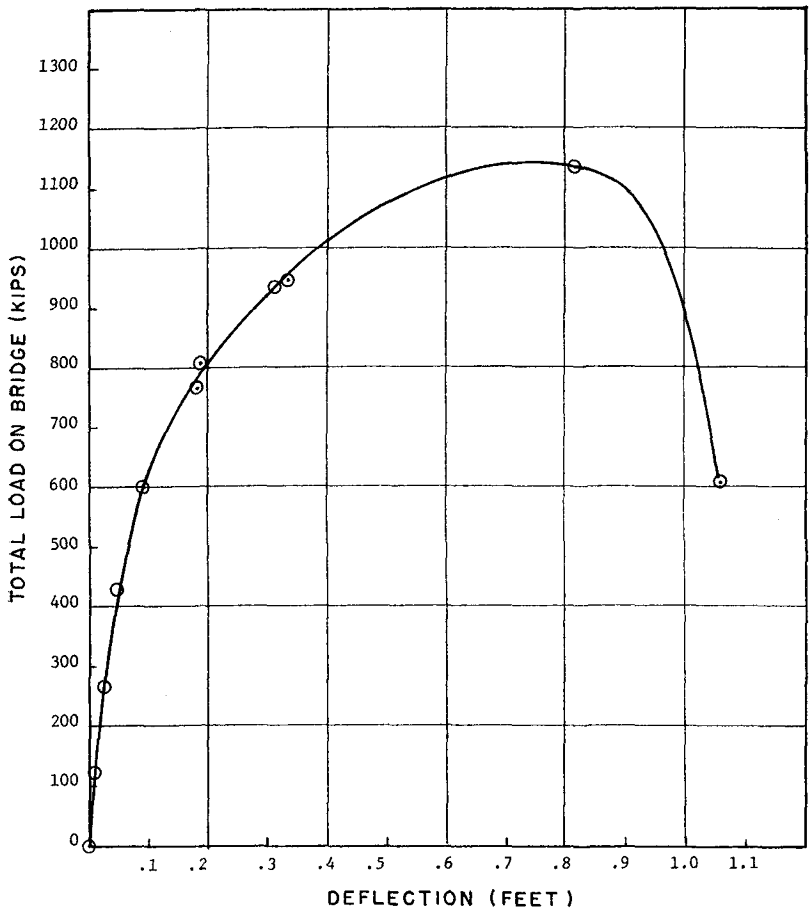


Fig. 9. Load-deflection curve; centerline of span over exterior beam; Span 2—Illustrating behavior of bridge from zero load to failure.

basis of the assumptions given in ACI 318-71, which are applicable for an extreme fiber strain in the concrete of 0.003.

The value of modulus of rupture was calculated from an equation given in Reference 5 to be 680 psi (4680 KN/m^2). This value was used in calculations of moment and curvature before cracking.

The stress-strain curve for the prestressing strand is also shown in Fig. 10. This curve was obtained from the manufacturer's data.

Calculation of load-deflection relationship

The method used for calculating deflections for particular loads on the bridge was based on the determination of moment-curvature relationships.[5,6] The load-deflection curve was determined by first taking a typical cross section of the bridge and developing a resisting moment versus curvature relationship, or $M - \phi$ curve. (The curvature ϕ is equivalent to M/EI for an elastic member.)

Then, moment diagrams were drawn for selected loads applied to the bridges from zero load up to failure. From the moment diagrams, corresponding ϕ diagrams were drawn by substituting the values of ϕ obtained from $M - \phi$ curves for corresponding values of M obtained from the moment diagrams.

Deflection for a corresponding load was then determined by loading the conjugate beam representing the bridge with the ϕ diagram determined for that load and calculating the bending moment in the conjugate beam at the point at which the deflection was desired. This moment computed due to the ϕ loading on the conjugate beam for the bridge was actually the deflection at the point where the moment was computed.

The bridge cross section was idealized to facilitate computations. Curvature due to the crown in the roadway and other shapes such as chamfered edges on the cross sections were idealized or ignored. Any effect of handrails was not considered. Supports were taken to be at the centerline of bearing and assumed to act as knife edges. Reinforcing steel in the bridge decks was not considered. The load due to the hydraulic rams was assumed, for calculation purposes, to have uniform lateral distribution; that is, the loads were treated as line loads extending across the bridge deck.

The bridge was considered to act as a single beam. Bending about the longitudinal axis, or the axis along the roadway centerline, was not considered. The curbs or raised sidewalk portions were considered as integral parts of the bridge. Fully composite action was assumed.

Comparison of results

The computed load-deflection curve for the bridge is shown in Fig. 11, along with the measured load-deflection curve. The curve based on test results is somewhat steeper throughout the elastic range and into the inelastic

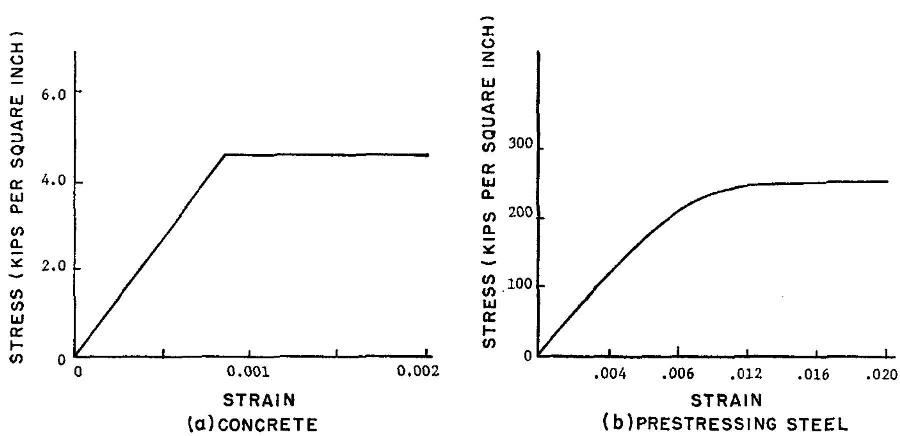


Fig. 10. Stress-strain curve for concrete and prestressing steel (used in calculation of load-deflection curve).

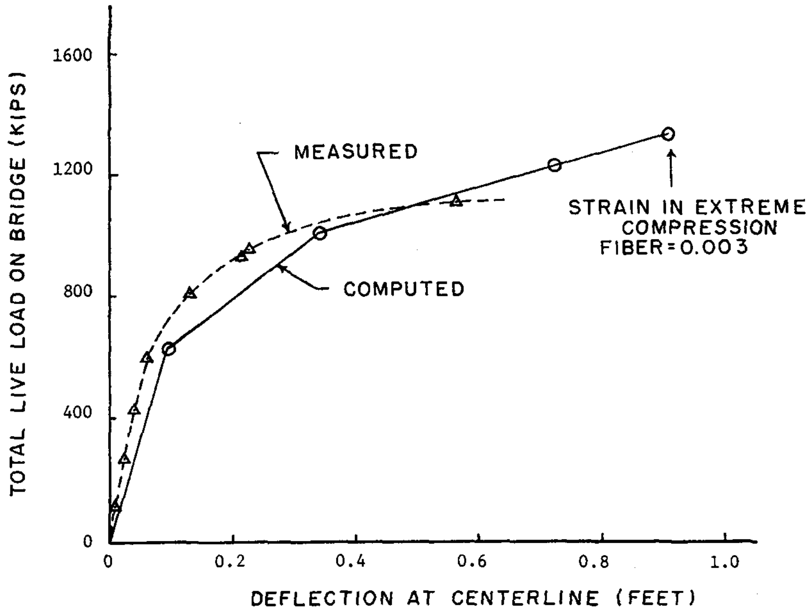


Fig. 11. Comparison of measured and computed load-deflection curves (Span 2).

range. Then, as composite action was lost, the bridge became less stiff than computations based on composite behavior indicated; and failure occurred at a lower load than the computed ultimate.

The computed value of strength was 1267 kips (5620 KN), and the measured value was 1140 kips (5050 KN). The loss of composite action and subsequent shear failure is, apparently, the reason for the difference.

CONCLUSIONS

Two pertinent conclusions follow from the test results and computations presented:

First, it is interesting to note that the idealization of the bridge considering it as a wide beam, including curbs, was adequate to permit the prediction of the bridge's load-deflection behavior with reasonable accuracy. This was true even though the bridge was on a relatively steep grade, was sharply skewed, and was superelevated for a 4½-deg horizontal curve.

The second conclusion of interest relates to the premature loss of composite action and is actually more a question than a conclusion. And the question, simply stated, is this: "How much do we really know about the behavior, from zero load to failure, of composite, prestressed concrete, multibeam bridges?" The results of the tests described suggest that the answer is, "not enough;" that is, not enough to be sure that longitudinal shear reinforcement provided on the basis of existing specifications[7] is adequate to assure that the bridge will attain its full flexural capacity. Further experimental work in this area would appear to be in order.

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Please forward your discussion to PCI Headquarters by October 1, 1974, to permit publication in the November-December 1974 PCI JOURNAL.