COMPARISON OF MEASURED AND COMPUTED LOAD-DEFLECTION BEHAVIOR OF TWO HIGHWAY BRIDGES

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Four deck girder highway bridges in Tennessee, located in an area to be inundated as part of a Tennessee Valley Authority reservoir, were tested under static load to failure. Research was directed toward comparing actual bridge behavior with that which could be calculated by using accepted structural analysis methods. This paper is concerned specifically with a comparison of measured and computed load-deflection relationships of two steel girder continuous bridges. Load-deflection curves calculated on the basis of strain compatibility relationships, assuming that the entire bridge with curbs acts as a wide beam, gave results that compared reasonably well with the actual load-deflection curves. Actual and calculated curves were particularly close for bridge A. The behavior of bridge B in the elastic region indicated that, although no provision was made to ensure composite action, such action did exist. The computed load-deflection curve compared very well with the measured curve in the elastic range when composite action was assumed to exist. It was concluded that the method presented for the prediction of load-deflection relationships was satisfactory. The total moment capacities calculated by this method were close to the measured capacities for both 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 that has since been flooded as a part of the Tennessee Valley Authority's Tims Ford Reservoir. At the time of testing, these bridges had already been replaced by newer bridges at higher elevations.

Each of the two bridges considered was a two-lane, continuous deck girder bridge, whose girders consisted of steel rolled beams with cover plates at interior supports. Bridge A was a four-span continuous bridge with span lengths of 70, 90, and 70 ft. It was designed in 1963 for an HS 20 loading. Studs were provided in the positive moment regions to ensure composite action.

Bridge B had three spans with span lengths of 45, 60, and 45 ft. It was designed in 1956 for an H-15 loading. No provision was made for composite action.

Both bridges were excellent test specimens. Bridge A was on a slight sag vertical curve; each was straight and had a 90-deg skew. Figures 1 and 2 show photographs of the two bridges.

The testing procedure is described in detail elsewhere (1, 2, 3). The position of the applied loads is shown in Figure 3. The loads on bridge A are intended to simulate two HS trucks, one in each traffic lane. Because of difficulties in rock drilling, only six loading points were used on bridge B.

Among other data obtained from the tests were load-deflection curves for various points on the bridge decks. An average of the deflection readings over the four girders at the centerline of the loaded span was obtained so that a representative load-deflection curve for each bridge could be plotted. These curves will be compared to the load-

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Figure 1. Bridge A.

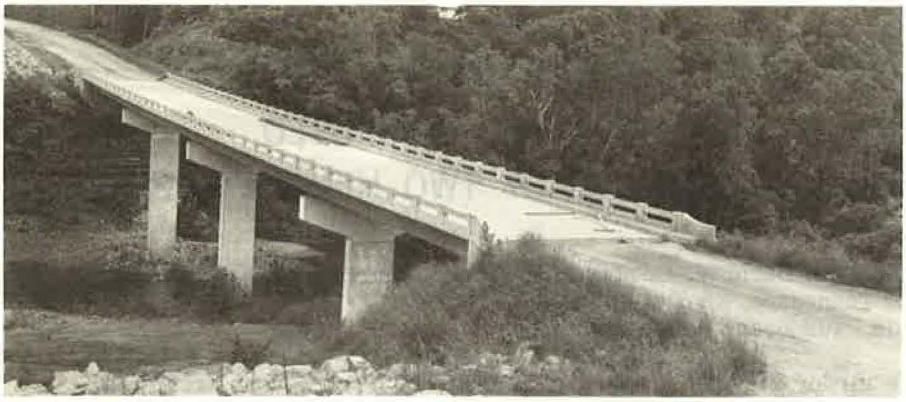


Figure 2. Bridge B.

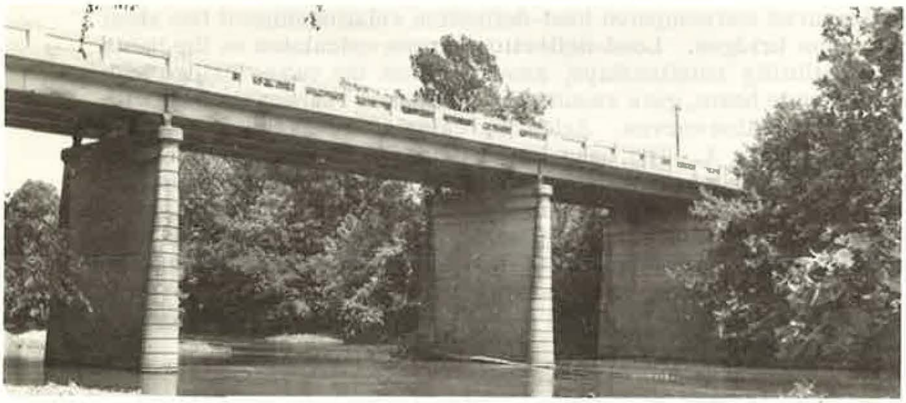
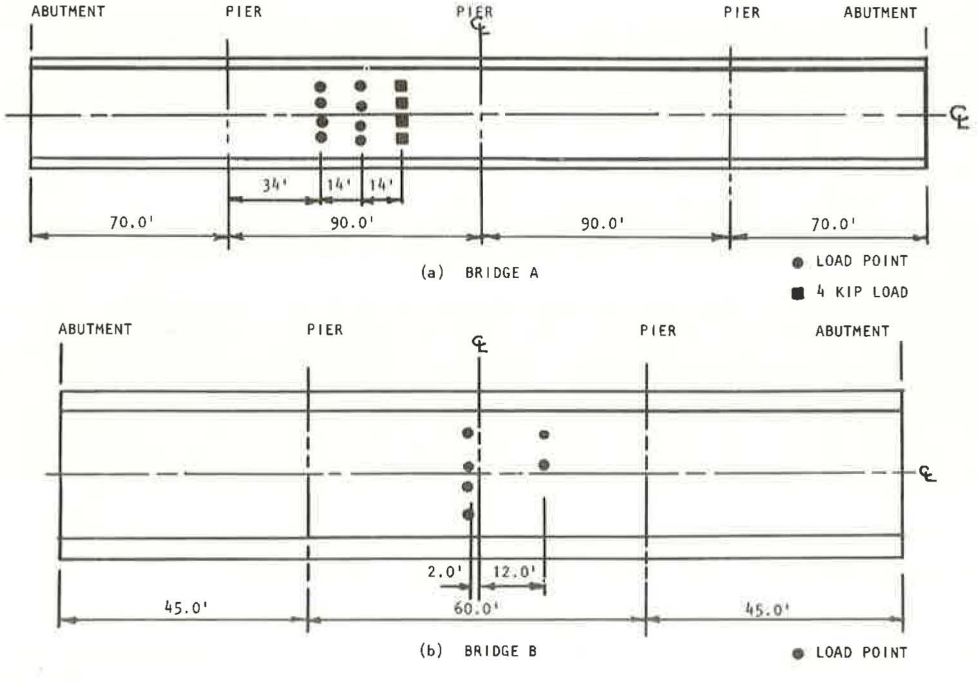


Figure 3. Position of loads used for bridges A and B.



deflection curves obtained analytically. Figures 4 and 5 show photographs of the bridges at failure.

A comparison of measured and computed ultimate strengths of the four bridges was discussed by Burdette and Goodpasture (2). This paper compares the measured load-deflection relationships for the two steel girder continuous bridges with results of an analytical solution based on strain compatibility relationships.

METHOD FOR OBTAINING THEORETICAL LOAD-DEFLECTION CURVES

The method used for calculating deflections for particular loads on the two bridges was based on the determination of moment-curvature relationships (4, 5). Load-deflection curves were determined by taking a typical cross section of the bridge and developing a resisting moment versus curvature relationship or M-ϕ curve. The curvature, ϕ, is equivalent to M/EI for an elastic member. The deflections for these bridges were determined with the ICES STRUDL-II computer program and the principle of superposition. The basic idea in this procedure is that, as load on a bridge increases beyond first yielding, the bridge properties change. To obtain the total deflection of the bridge requires that the deflections be added or be superimposed every time the properties of the bridge change. So that these procedures could be used several assumptions were made.

Assumptions

The locations of sections at which plastic hinges would form were predicted. For bridge A, these were at the line of applied loads nearest midspan and at the section over the pier nearest the first section. For bridge B, plastic hinges were assumed to form at the line of applied loads nearest midspan and over the two piers. The actual test confirmed these assumptions except that plastic hinges formed at the ends of the cover plates a short distance from the piers—on the other side of the piers from where the loads were applied rather than directly over the piers.

The bridges were idealized to facilitate computations. Curvature caused by 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 caused by 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 bridges were considered to act as single beams. Bending about the longitudinal axis or the axis along the roadway centerline was not considered. Calculations considered the curbs or raised sidewalk portions as integral parts of the bridges. No effort was made in these computations to investigate individual parts of the bridge, i.e., comparing the amount of load carried by an interior girder versus that carried by an exterior girder.

Material Properties

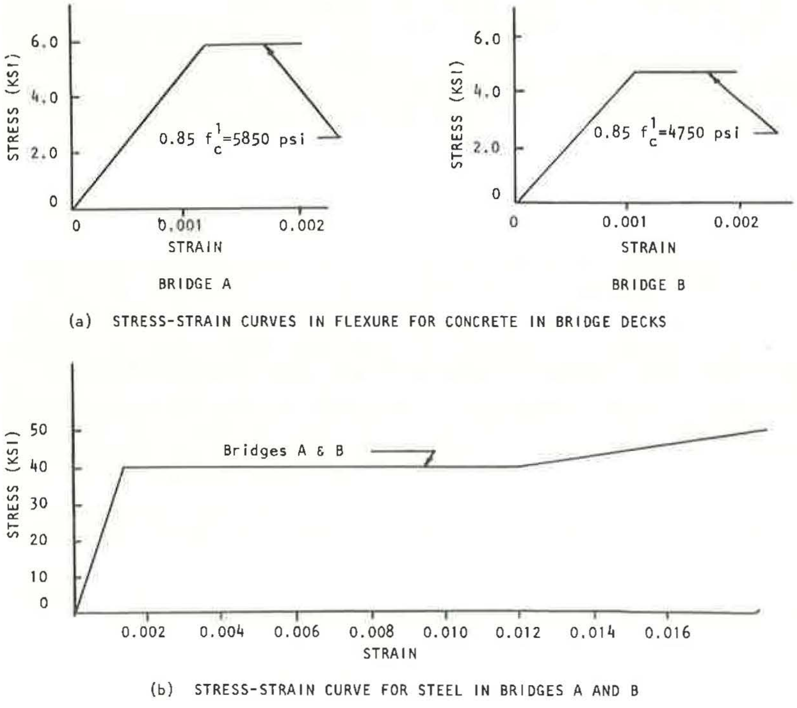
Stress-Strain Curves for Concrete—With the values of f_c' determined from the compression tests and formulas (6), idealized stress-strain curves for concrete were developed for each bridge. Taking $0.85 \times f_c'$ as the maximum flexural stress the concrete can attain and $w^{1.5}(33) f_c'$ as the modulus of elasticity of concrete, where w is the weight of concrete in pounds per cubic foot, enabled a stress-strain curve for the concrete in each bridge to be developed. Three assumptions were made so that the curves could be drawn: (a) all concrete weighed 145 lb/ft^3, (b) the stress was constant after reaching a maximum of 0.85 f_c' and up to a strain of 0.002, and (c) the modulus of elasticity was constant up to a stress value of 0.85 f_c'. No attempt was made to develop the curves past a strain of 0.002. The assumptions given (6) were used to calculate moment and curvature at concrete strains greater than 0.002. Figure 6 shows the stress-strain curves for the concrete in the two bridges.

Stress-Strain Curves for Steel—The stress-strain relationships for the steel in each bridge, which are shown in Figure 6, were used in the computations.

Figure 4. Bridge A at failure.

Figure 5. Bridge B at failure.

Figure 6. Stress-strain curves.



Determination of M-ϕ Curves

The determination of the M-ϕ relationships for the positive bending moment region of bridge A involved consideration of both the concrete and the steel, whereas the M-ϕ curves for bridge B and for the negative bending region over the piers of bridge A considered only the steel girders. In the regions where only steel was considered, the M-ϕ curves were calculated by assuming noncomposite behavior because there were no shear studs between the steel beams and the concrete deck. The methods used to calculate M-ϕ curves required application of the following necessary relationships:

1. Equilibrium of horizontal forces and moments,
2. Assumption of linear strain distribution,
3. Knowledge of the stress-strain relationships for concrete and steel, and
4. Perfect bond between beams and bridge deck where composite construction was assumed.

Idealized M-ϕ diagrams for the negative moment region of bridge A and for both positive and negative moment regions of bridge B were obtained quite simply. First, the moment and curvature at first yield of the steel were calculated on the basis of elastic theory, and this point was plotted. Then, the plastic moment of the cross section was calculated, and a horizontal line representing this moment was plotted on the M-ϕ diagram. Finally, a straight line intersecting the horizontal line representing the plastic moment was drawn from the origin ($M = 0$, $\phi = 0$) through the point at yield. The resulting M-ϕ diagrams for positive and negative moment regions of bridge B are shown in Figure 7.

The M-ϕ curve for the positive moment region of bridge A could not be so easily idealized. The method used to calculate points on the curve is essentially that described by Warwaruk, Sozen, and Siess (4) and Khachaturian and Gurfinkel (5). The diagram was assumed to be perfectly linear up to first yield of the steel. From that point forward, points on the curve were calculated on the basis of a chosen strain in the extreme compressive fibers of the concrete. The M-ϕ diagrams for both positive and negative moment regions of bridge A are shown in Figure 8.

Load-Deflection Curves

The load-deflection curves for bridges A and B were developed by using the STRUDL-II subset of the ICES computer program, the law of superposition, and the M-ϕ curves developed previously.

Method of Computation—The plane frame option of the STRUDL-II subset of the ICES program uses a stiffness method of analysis to calculate moments, shears, reactions, and deflections of a beam or frame subjected to a prescribed loading. This program requires that the moments of inertia and modulus of elasticity be specified constants for each member. If a beam is not of uniform cross section throughout its length, a joint can be chosen at each section where the beam cross section changes, and appropriate E and I values can be specified for the several beams needed to represent the actual beam being analyzed. This procedure was used to analyze bridges A and B with cross-section properties that were not constant throughout their length.

Both bridges were treated as wide continuous beams as was done in the determination of M-ϕ curves. The beams were subjected to concentrated loads so that maximum moment at the load nearest midspan was produced. The loading was increased until enough plastic hinges were formed to create a mechanism, and the total load required to form the mechanism was considered to be the ultimate load for the bridge.

When the ultimate moment was reached at a section with an M-ϕ curve like those for bridge B and the negative moment region of bridge A, the structure was modified for further load application by considering that a hinge existed at the section. The load, acting on the "new" structure with a hinge, was increased until ultimate moment was reached at another section. If this section also had an M-ϕ relationship like those noted, a second hinge was introduced in the structure with the result that a third structure was formed. The load acting on this structure was increased until another plastic hinge was formed. This procedure was continued until a mechanism existed, and the

Figure 7. Moment-curvature curves for bridge B.

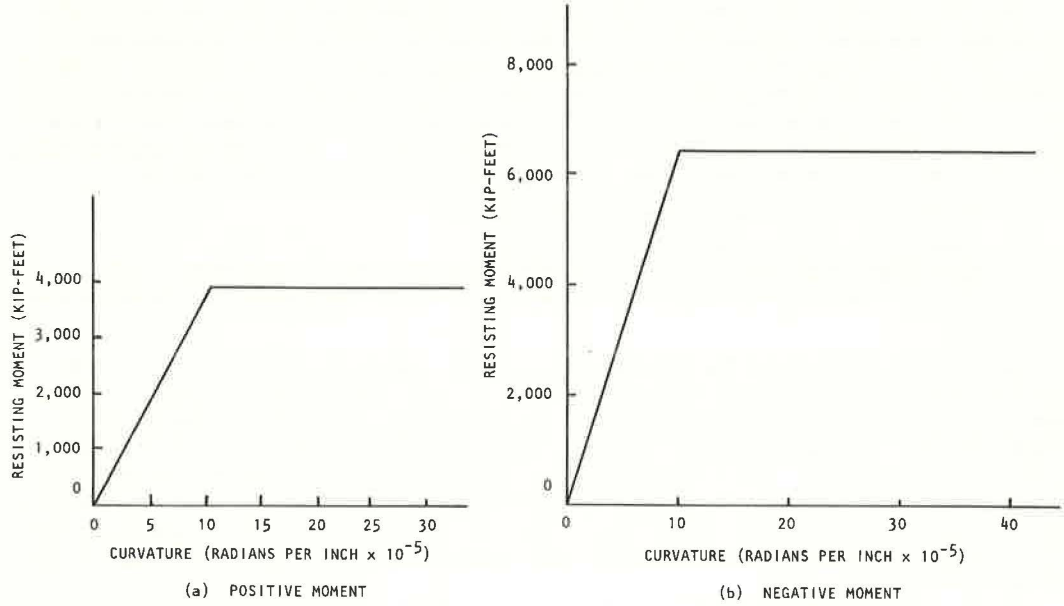
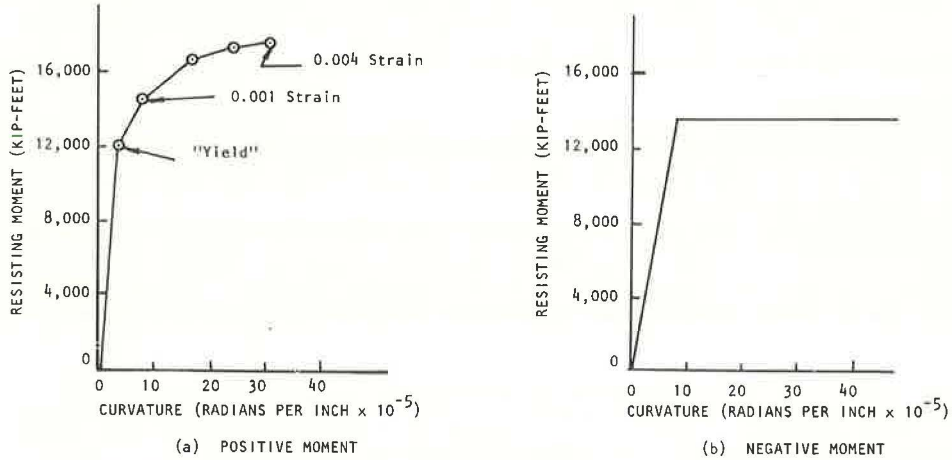


Figure 8. Moment-curvature curves for bridge A.



ultimate load was calculated as the sum of the loads that were acting on the structure at various stages to form plastic hinges.

If the M-ϕ curve for the section of maximum moment was like that for the positive moment region of bridge A, the fact that the section does not reach ultimate moment immediately after stopping to be elastic necessitates a somewhat modified approach. The procedure for handling this case was as follows:

1. When the load was sufficient to cause a moment at the section equal to that at which the section no longer behaved elastically, a new structure was considered for further application of load.
2. This new structure for additional load could not be considered as having a hinge at the maximum moment section, because the M-ϕ curve still had a positive slope. Instead, a small length of structure centered at this section was modified to have an EI consistent with the slope of the M-ϕ curve beyond the point of first loss of elastic action.
3. The length of this revised section was taken as the overall depth of the member which, for the composite section of bridge A, was approximately 4 ft.

One further consideration was required for bridge A. No provision was made for a downward reaction at the abutments; thus, during the test, the bridge lifted off the abutment nearest the loaded span. This occurrence was considered in the analysis by making appropriate modification to the structure at the load that caused a zero reaction at the abutment.

Results—The calculated load-deflection curves for the two bridges are shown in Figures 9 and 10 with the average curve obtained experimentally.

COMPARISON OF RESULTS

The calculated and measured load-deflection curves for bridge A compared remarkably well, as shown in Figure 9. The computed ultimate load was 1,270 kips, and the measured ultimate load was 1,250 kips.

The two curves shown in Figure 10 for bridge B do not compare so favorably. There are two probable reasons for this lack of agreement: (a) The loading on the bridge was not perfectly symmetrical (Fig. 3), and (b) the calculated behavior of the bridge was based on noncomposite behavior, whereas the actual behavior as indicated by strain measurements reflected a high degree of composite action up to near yielding of the steel. The first reason probably explains why the actual ultimate load (640 kips) was less than the computed ultimate (696 kips); that is, not all girders reached their plastic moment capacity under the same loading.

The second reason, i. e., the assumption of composite behavior, explains the discrepancy between the two curves for bridge B in the elastic region. The line obtained on the basis of full composite action coincided almost identically with the measured curve in the elastic range (Fig. 10).

CONCLUSIONS

The method used in this paper to predict load-deflection relationships for the two bridges proved to be satisfactory. [A detailed discussion of the research results (1) and of certain aspects of the research (2, 3) is available.] The ultimate load, calculated on the basis of considering the bridge to act as a single wide beam, compared closely with that obtained in the field tests. This suggests the possibility of basing ultimate strength design methods on a comparison between the total bridge moment and the sum of the ultimate moment capacities of the individual girders.

That composite action existed in bridge B, even though it had been subjected to severe overloads and to vibratory loads before static tests began, is of interest. This indicates that composite action will generally occur in a deck girder bridge up to loads well beyond the design load for the bridge. Laboratory tests to confirm this observation and, thus, to lead to design criteria that appropriately reflect this phenomenon would appear to be in order.

Figure 9. Comparison of measured and computed load-deflection curves for bridge A.

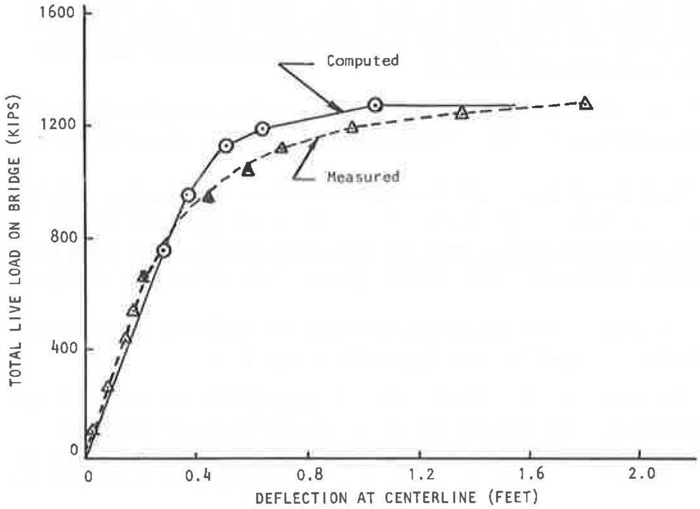
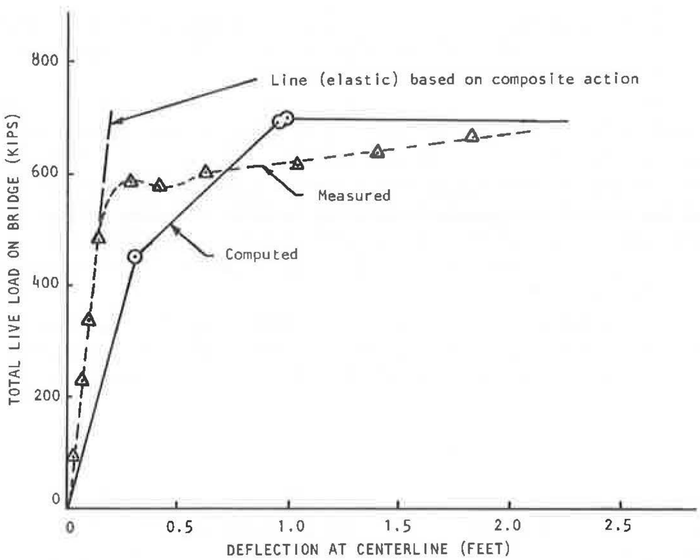


Figure 10. Comparison of measured and computed load-deflection curves for bridge B.



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REFERENCES

1. Burdette, E. G., and Goodpasture, D. W. Full-Scale Testing. Tennessee Department of Transportation and Federal Highway Administration, Final rept., Dec. 31, 1971.
2. Burdette, E. G., and Goodpasture, D. W. Comparison of Measured and Computed Strengths of Four Highway Bridges. Highway Research Record 382, 1972, pp. 38-50.
3. Burdette, E. G., and Goodpasture, D. W. Tests of Four Highway Bridges to Failure. Jour. Structural Division, Proc. ASCE, March 1973.
4. Warwaruk, J., Sozen, M. A., and Siess, C. P. Investigation of Prestressed Reinforced Concrete for Highway Bridges—Part III: Strength and Behavior in Flexure of Prestressed Concrete Beams. University of Illinois Engineering Experiment Station Bull. 464, Urbana, 1962.
5. Khachaturian, N., and Gurfinkel, G. Prestressed Concrete. McGraw-Hill, 1969.
6. Building Code Requirements for Reinforced Concrete. American Concrete Institute (ACI 318-71), 1971.