Lateral Load Distribution for Two Continuous Steel Girder Highway Bridges

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Three Tennessee state highway bridges and one Franklin County highway bridge were located in an area that has been inundated as a part of the Tennessee Valley Authority (TVA) Tims Ford Reservoir. These bridges were made available by the TVA, the Tennessee Department of Highways, and the Franklin County Highway Department for testing purposes. Descriptions of this testing program have been published by Burdette and Goodpasture (1, 2, 3, 4).

This paper presents the results obtained as a part of the full-scale bridge testing program (FSBT) of the University of Tennessee and tank transporter tests by the Bureau of Public Roads to determine the lateral distribution of load on two multispan continuous steel beam and slab bridges.

The results of an ICES STRUDL II finite element analysis relating to lateral load distribution for each bridge (6, 7) are compared with the experimental results from FSBT. Both the analytical and experimental results are then compared to the empirical design equation in the 1973 AASHO specifications (8) and the proposed revisions to the AASHTO specifications made by Sanders and Elleby (9).

Further background information and derivation of all formulas may be found in the author's original thesis (10).

DESCRIPTION OF BRIDGES

Each of the four bridges tested in the full-scale bridge testing program was a two-lane deck girder bridge with four longitudinal girders. The Bureau of Public Roads (BPR) tests were performed on three of these four bridges (5). The findings reported herein are limited to two of the FSBT bridges, one of which was the BPR bridge. These bridges are numbered 1 and 4 in the FSBT final report (1) and will be so referenced here. A description of the bridges is given in Table 1. Both of the bridges appeared to be structurally sound before

testing. Approximate average daily traffic volumes in 1968 for bridges 1 and 4 were 600 and 200 respectively.

ROLLING LOAD TESTS

To determine the lateral distribution of applied loads, four different vehicles were driven across the bridges in each of the traffic lanes at speeds ranging from a crawl to 56 km/h (35 mph).

The vehicles used to load the bridges were as follows:

Vehicle	Load (Mg)
Federal Highway Administration	32.7
University of Tennessee	59.9
M-60 track-laying combat tank	47.6
Army HET-70 tank transporter	88.4

When the FHWA test vehicle is loaded, it closely simulates HS-20 design vehicle loading and is referred to as HS-20 here. When the University of Tennessee test vehicle is loaded with solid concrete blocks, its axle loads are almost double those of the HS-20 design vehicle load and it is therefore referred to as HS-40. The Army HET-70 tank transporter has eight pneumatic-tired axles.

STATIC LOAD TESTS TO FAILURE

To determine the lateral distribution of loads at high load levels and how the distribution changes with increasing load, static tests to failure were performed on each bridge. Application of the loads simulated an HS truck in each lane, and they were located in the approximate position to cause maximum positive moment near the center of the span. The points on the span at which load was applied were at the positions of the eight rear wheels of the two simulated trucks. A complete description of the tests and test results is given in the literature (1, 2, 3, 10).

EXPERIMENTAL DATA AND RESULTS

The lateral distribution of loads as determined by the bending moment supported by each girder is computed by using the measured strain in the top and bottom fibers

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Table 1. Description of bridges.

Bridge Number	General Description	Span (m)	Girder Spacing (m)	Skew (deg)	Location	Date	Loading
1	Four-span continuous, 91-cm steel rolled beams, composite in positive moment regions	21, 27, 27, 21	2.54	90	Tenn-130 over Elk River	1963	HS-20
4	Three-span continuous, 68.6-cm steel rolled beams, noncomposite	14, 18, 14	2.24	90	Mansford Road over Elk River	1956	HS-15

Note: 1 cm = 0.39 in; 1 m = 3.28 ft.

Figure 1. Comparison of actual and theoretical distribution factors for bridge 1.
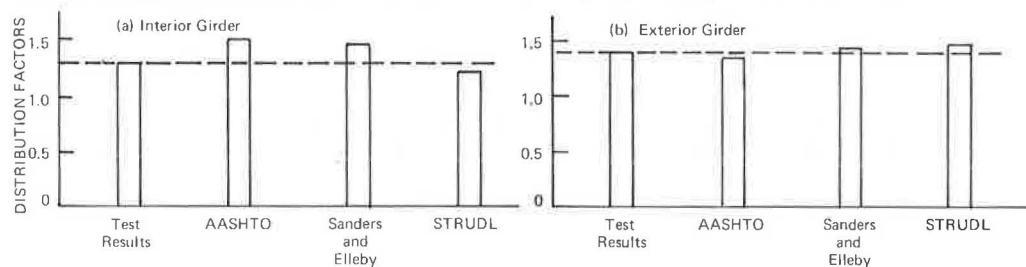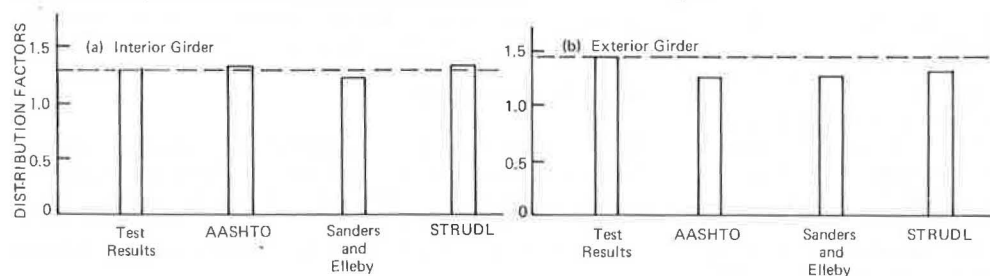
Figure 2. Comparison of actual and theoretical distribution factors for bridge 4.

in combination with the properties of the girder cross section.

Instrumentation

Lateral distribution of loads was determined by mounting a foil gauge on the underside of the top and bottom flanges of each of the four girders at the point of maximum moment. These gauges were monitored with a minicomputer connected with a digital-to-analog converter interface.

Data Interpretation

From the measured strains in the top and bottom flanges, the moment in each individual girder was computed. The exact method used is discussed elsewhere (10).

Results

The term load distribution coefficient is used to denote the percentage of total live load bending moment supported by a particular longitudinal girder. McDougle (10) lists these coefficients for rolling load tests and static load tests for both subject bridges.

PREDICTION OF LATERAL DISTRIBUTION

The theoretical analysis of bridges to determine the lateral distribution of load is a difficult, time-consuming, and expensive process. For more than 30 years,

AASHTO has published and authorized the use of an empirical method of distribution to simplify the problem. Since that time, numerous researchers have advanced their own theories and proposed revisions to the AASHTO load distribution equation. A comparison of the results of the proposed revisions by Sanders and Elleby, the current AASHTO specifications, and a STRUDL analysis of each bridge is given below.

AASHTO Specifications

The current AASHTO specifications (8) state that, for a two-lane bridge with a concrete deck over I-beam stringers spaced not more than 4.3 m (14 ft) apart, the live load bending moment for each interior stringer shall be determined by applying the fraction $S/5.5$ (S = stringer spacing) of a wheel load to the stringer. For a similar bridge, the live load bending moment for an exterior stringer is determined by applying the fraction $S/(4.0 + 0.25S)$ of a wheel load.

Sanders and Elleby Proposal

In their final report, Sanders and Elleby (9) recommend significant changes to the AASHTO specifications for determining lateral distribution of load. Specifically, they recommend for beam and slab bridges that the live load bending moment for each beam (both interior and exterior) be determined by applying the fraction of a wheel load determined by their formula, which is based on bridge geometry and material properties.

STRUDL II Analyses

Both bridges were modeled by using the STRUDL II subset of the ICES computer program. The models included a combination of plate bending elements to simulate the concrete deck and grid members to simulate the longitudinal beams. Distribution coefficients were computed by loading these models of the bridges. Distribution factors for interior and exterior girders were then determined by superimposing distribution coefficients for two trucks in the desired position.

Rolling Load Tests

Rolling load tests were performed in each lane on each bridge. Distribution factors are computed by superimposing the distribution coefficients of a given girder for two tests to obtain the maximum value for comparison with AASHTO values.

Comparisons of distribution factors for bridges 1 and 4 are shown in Figures 1 and 2.

CONCLUSIONS

From the data and comparisons made here, the following conclusions relating to the lateral distribution of load are offered.

1. The rolling load data suggest that, for a particular bridge, the lateral distribution of load depends almost entirely on the lateral position of the vehicle on the bridge. Although axle spacing, magnitude of vehicle load, and vehicle speed have a measurable effect on the total moment at a bridge cross section, these parameters appear to have only a minor effect on the lateral distribution of load to supporting girders.
2. The static load test data indicate that, as the load on a bridge is increased up to the order of magnitude that produces first yield of the steel, the load distribution coefficients are not significantly affected.
3. The AASHTO specifications require distribution factors that are slightly conservative for interior girders and slightly unconservative for exterior girders, indicating that they do not fully account for the additional stiffness contributed to the exterior girder by curbs and rails.
4. The Sanders and Elleby equation yields distribution factors that are conservative for the composite bridge and somewhat unconservative for the noncomposite bridge. If the noncomposite equation proposed in their report is adjusted to include partial slab participation, the factors are in closer agreement. Their exclusion of the effects of curbs and handrails is shown to be invalid.
5. The distribution factors predicted by the STRUDL II models were not entirely consistent with the experimental values. The relative stiffness of the interior girder-slab section and the exterior girder-slab-curb section appears to have a great deal of effect on the distribution of load. These parameters must be carefully evaluated and adjusted to ensure that the model acts like the real structure.
6. Although the AASHTO requirements did not predict the actual maximum distribution factors precisely as determined by test results, they were just as accurate, on the average, as any of the other methods reported here.
7. The M-60 and HET-70 loads were more evenly distributed to the longitudinal girders because of the vehicles' track and wheel spacings.

REFERENCES

1. E. G. Burdette and D. W. Goodpasture. Final Report on Full Scale Bridge Testing—An Evaluation of Bridge Design Criteria. Department of Civil Engineering, Univ. of Tennessee, Dec. 31, 1971.
2. E. G. Burdette and D. W. Goodpasture. Comparison of Measured and Computed Strengths of Four Highway Bridges. HRB, Highway Research Record 382, 1972, pp. 38-50.
3. E. G. Burdette and D. W. Goodpasture. Tests of Four Highway Bridges to Failure. Journal of Structural Division, Proc., ASCE, March 1973.
4. E. G. Burdette, D. W. Goodpasture, and S. K. Doyle. Comparison of Measured and Computed Load-Deflection Behavior of Two Highway Bridges. TRB, Transportation Research Record 507, 1974, pp. 17-25.
5. R. F. Varney. Preliminary Research Report of a Highway Bridge Loading Study With the HET-70 Main Battle Tank Transporter. Bureau of Public Roads, 1969.
6. A. C. Bogaty, Jr. A Study Concerning Data Input Into a Finite Element Analysis of a Highway Bridge. Univ. of Tennessee, Knoxville, MS project paper, 1973.
7. A. W. Rush, Jr. A Study of Wheel Load Distribution in a Highway Bridge. Univ. of Tennessee, Knoxville, MS project paper, 1973.
8. Specifications for Highway Bridges. AASHO, Washington, D.C., 1973.
9. W. W. Sanders, Jr., and H. A. Elleby. Distribution of Wheel Load on Highway Bridges. Engineering Research Institute, Iowa State Univ., Ames, 1968.
10. E. A. McDougle. Comparison of Measured and Computed Lateral Load Distribution for Two Continuous Steel Girder Highway Bridges. Univ. of Tennessee, Knoxville, MS thesis, 1975.