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### Recommended Citation

Errera, S. J. and Mindlin, H., "Tests of a composite aluminum and concrete highway bridge, (Oral presentation at ASCE Annual Convention, October 1959)." (1959). *Fritz Laboratory Reports*. Paper 1752.  
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TESTS OF A COMPOSITE ALUMINUM AND CONCRETE HIGHWAY BRIDGE

(For Oral Presentation at ASCE Annual Convention  
in Washington, D.C., October 22, 1959)

by

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Fritz Engineering Laboratory Report No. 275.2

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## A. INTRODUCTION

1. Background

Based on experience with aircraft structures, the Kinetics Division of Fairchild Engine and Airplane Corporation designed a composite aluminum and concrete highway bridge using the principles of semi-monocoque construction. The design permitted shop fabrication of large triangular cellular units of the aluminum portion of the bridge. Compared with conventional bridge structures, this combination of light weight material and shop fabrication offered the following advantages (Slide 1):

1. lower dead weight and dead weight stresses.
2. abutments, footings, and end supports could be of lighter construction.
3. lower transportation costs from the point of fabrication to the erection site.
4. field erection costs would be reduced.
5. maintenance costs after erection would be lower.

Designed with the assistance of the Bureau of Public Roads, in accordance with the American Association of State Highway Officials Specifications and the American Society of Civil Engineers Specifications for the alloy used, a two-lane test structure of 50-foot span was fabricated at the Fairchild plant, Hagerstown, Maryland, and erected and tested on the Lehigh University Campus in Bethlehem, Pennsylvania.

## 2. Purpose and Scope

The primary purposes of the test program were as follows:

1. determine the response of the structure to an applied static load, enabling comparison between predicted and actual behavior.
2. determine suitability of the structure for highway service as indicated by an anticipated lifetime of load repetitions.
3. determine the ultimate static strength of the structure.

### B. DESCRIPTION OF TESTS

#### 1. The Test Bridge

One of the final steps in the plant fabrication was the mating of the components on special jigs to insure a proper fit at the erection site (Slide 3). The aluminum portion of the bridge consisted essentially of three 50-foot long hollow triangular beams, each mounted on its inverted apex, bolted together at the upper corners. Two horizontal plates tied the three lower apices together forming a complete bridge. Each beam was composed of three longitudinal extrusions, three stiffened plates, and stiffening frames at 5-foot spacing. Attached to the top plating was 2 1/2" deep corrugated decking upon which the reinforced concrete roadway was later poured. Z-Section shear connectors were attached to the main longitudinal extrusions. Nine-inch cantilevers at the edge of the outside triangular beams completed the full 24-foot width of roadway. To react the stresses caused by the difference in thermal coefficients of aluminum and concrete, a shear transfer device called a "thermal beam" was used near each end of the structure.

The five basic sub-assemblies, consisting of the three triangular

beams and two bottom plates, were then transported by truck to the test site (Slide 4). The erection sequence for the aluminum structure was as follows:

1. Mating of two triangles and one bottom plate to form the first unit placed on the end supports (Slide 5).
2. Placing of the second bottom plate (Slide 6).
3. Placing of the third triangular beam (Slide 7).
4. Completion of the field bolting.

Steps one to three were completed the first day and step four was completed the following day. Cold driven rivets were used in the shop fabrication, and standard nuts and bolts and commercial type lockbolts were used in the field. The completed structure assembled 11,360 lbs. of 6061-T6 aluminum alloy extrusion and plating into a five cell semi-monocoque bridge.

The concrete used for the deck embodied a slag-aggregate whose light weight helped to minimize the dead-weight stresses in the aluminum. The deck extended  $5 \frac{7}{8}$  inches above the top of the  $2 \frac{1}{2}$ -inch deep corrugations in the top surface of the aluminum structure (Slide 8). All steel reinforcing bars were placed above the corrugations and separated from the aluminum by insulation. The need for any external support of the formwork during the pouring of the concrete deck was eliminated by bolting the aluminum side forms directly to the top outer edges of the aluminum structure.

## 2. Test Program (Slide 9)

The test program was designed to check the structure statically before and after each series of dynamic load applications. Thus any damage or change

in strain distribution within the structure could be detected. The test bridge was subjected to 13 static tests, and a dynamic test program summarized as follows:

1. 250,000 cycles at design live load plus impact moment,  $M_{LL}$ .
2. 250,000 cycles at 125%  $M_{LL}$ .
3. 753,000 cycles at 150%  $M_{LL}$ .
4. 200,000 cycles at 125%  $M_{LL}$  with load applied eccentrically producing a torsional moment of 6,220,000 in-lbs.

This is estimated to be well beyond the cyclic loading endured by a bridge on a Class I highway in more than 100 years of service.

In addition to the static and fatigue tests, three impact loadings were applied to the span to determine the natural frequency of the structure.

The final static test was to destruction.

### 3. Test Procedure

A test frame, erected over the bridge at mid-span, supported two Amsler hydraulic jacks, each in bearing against a transverse loading beam which applied the load concentrically in the 12-foot traffic lane (Slide 10). Each beam acted against the deck through two 13" x 26" steel bearing pads 6 feet center to center, designed to simulate the rear tire spacing of an H15-44 truck. For the eccentric static and dynamic load tests, jacks were placed three feet on each side of the centerline of one traffic lane to produce one lane loading.

The reaction for the applied loads was provided by the dead weight of the frame, the frame footings, and steel slabs stacked on the frame and footings. For the destruction tests, the jack loads were augmented by steel slabs placed directly on the bridge.

For the dynamic tests, each jack was driven by an Amsler pulsator which produces sinusoidal variation of load. The two pulsators, connected in parallel to insure synchronization, applied the load at a rate of 250 cycles per minute.

To eliminate the effects of temperature during the static tests, three readings were made to determine the effects of each load increment:

1. Readings of all gages with no load on the span.
2. Readings of all gages with the span loaded to the appropriate increment.
3. Final readings of the span again completely unloaded.

Averaging the loading and unloading increments minimized temperature effects on the results. To check the accuracy of the method, one test was run during the night, a period of small temperature change, and then repeated over a normal day-time variation of ten to fifteen degrees Fahrenheit. Very good correlation was obtained.

#### 4. Instrumentation

Because of symmetry of the test structure and applied load, instrumentation was applied to the east half of the bridge only and measured the following:

- (1) deflections at the centerline and quarter point.
- (2) strains at the centerline due to bending.
- (3) strains at the quarter point due to bending and shear.
- (4) strains in the center frame and end frame.

All aluminum strain measurements were made with resistance type SR-4 uniaxial or rosette electrical strain gages bonded to the metal surface.

A strainometer, also an electrical resistance type gage, was used to measure internal concrete strains. On the concrete deck surface, strains were measured with a mechanical Whittemore gage over a ten-inch gage length. Deflections of the span were measured with dial gages under the three main longitudinal members at the centerline and quarter point. Scales were placed on the deck at the centerline, quarter point and over the end supports to check the dial deflections, determine any relative deflection between the deck and tension members, and measure any possible support settlement. These scales were read against a fixed reference with an engineer's level. Dial gages were also used to measure the horizontal movement of the free end of the bridge relative to the center fixed pedestal of the end support, and the relative movement between the concrete deck and a top longitudinal member.

During all dynamic load tests maximum centerline deflections under the north and south extrusions were measured with slip-gages, mechanical devices employing a dial gage to record maximum downward movement (Slide 11). During one dynamic test in which load was applied eccentrically to the span, a record was made of centerline deflections and strains in the three bottom longitudinal extrusions using a six-channel Brush Recorder. The natural frequency of the structure was determined by recording the instantaneous centerline deflection of the span due to a suddenly applied load, using the Brush equipment and a cantilever bar mounted with SR-4 gages.

The ambient air temperature and temperature distribution within the span were recorded throughout the testing period.



### C. THEORETICAL ANALYSIS

The theoretical analysis of the test structure made by Fairchild Engine and Airplane Corporation is described in their Report No. 50-S1 and is briefly summarized in Fritz Laboratory Report No. 275.1. The results of this analysis will be used as a basis of comparison for the test results.

A comparison of the live load plus impact moment diagram for H15-44 AASHO loading and for the equivalent test loading is shown in Slide 12. A test load of 69,000 lbs. applied at the centerline produced a bending moment equal to the live load plus impact moment of 10,313,000 in-lbs. required by AASHO specifications.

### D. PRESENTATION OF RESULTS

#### 1. Deflections

A comparison of the predicted and measured deflections under a load of 69 kips (100%  $M_{HL}$  at the centerline) is shown in Slide 13 for measurements made at the centerline and quarter point.

#### 2. Stresses

In Slide 14, the predicted stresses in the bottom longitudinal members at the centerline due to the design live load plus impact moment are compared with the stresses derived from the measured strains, using a modulus of elasticity  $E$  of 10,000,000 psi. For the same load (69,000 lbs.) eccentrically applied, the stresses are as shown in Slide 15. The stresses in the top longitudinal aluminum members were negligible.

The

Test results indicated the location of the neutral axis was in the

plane of the top sheet, approximately 47.4 inches above the gages on the bottom members (Slide 16). The calculated height of the neutral axis from the same reference was 46.6 inches.

Stresses in the concrete under concentric loading varied from 280 psi to 360 psi; under eccentric loading the concrete stresses varied from 390 psi in the loaded lane to 180 psi in the unloaded lane. All concrete stresses were based on a modulus of elasticity of 3,000,000 psi. Results indicated that the entire deck was active in bending.

Calculated shear stresses and the stresses derived from the measured strains using a shear modulus  $G$  of 3,840,000 psi are compared in Slide 17 for the concentric loading, and in Slide 18 for the eccentric loading.

The maximum measured stress in any member of the centerline frame was approximately 3200 psi under an eccentrically applied load of 69,000 lbs. The maximum measured compressive stress in the end frame also occurred under the eccentric loading condition and was equal to 3200 psi. The measured stresses were considerably less than the 6150 psi calculated live load design stress.

### 3. Temperature Response and Natural Frequency (Slide 19)

The centerline deflection of the span averaged 0.0062 inches downward for a one degree rise in ambient air temperature, compared with a predicted value of 0.00628 inches per degree temperature change. The natural frequency was predicted to be approximately 400 cycles per minute and was measured at 333 cycles per minute.

### 4. Effect of Repeated Loads

Static tests before and after each dynamic test indicated the bridge

did not suffer any visible damage or loss of structural integrity due to the application of over 1,450,000 cycles of load producing from 100% to 150% design live load plus impact moment.

#### 5. Destruction Test

A view of the final static test is shown in Slide 20. Up to a shear load of 241,800 lbs. (3 times design live load plus impact shear) and a bending moment of 63,500,000 in-lbs. (over 6 times design live load plus impact moment) there was linear relationship between static load and all measured stresses or deflections (Slide 21).

The highest load sustained by the bridge produced a bending moment of 100,000,000 in-lbs. (970% of the live load plus impact design moment) at the centerline, with a corresponding shear force of 378,800 lbs. ((470% of the AASHO design requirement of 81,000 lbs.)). This load was held for ten minutes, then partially released. Failure occurred at 885%  $M_{LL}$  during an attempt to reload the span (Slide 22).

#### Summary

Summarizing, with reference to the three objectives stated earlier:

1. There was close correlation between theoretical and experimental behavior of the Fairchild Aluminum Bridge under static load.
2. The structure withstood over 1,250,000 cycles of load producing from 100% to 150% of design live plus impact bending moment, and 200,000 cycles of 125% of design live load plus impact moment applied eccentrically, without evidence of distress.
3. Final failure of the structure occurred at a load producing a moment more than 8 times the design live plus impact bending moment, and a corresponding shear more than 4 times the design live plus impact shear.

## LIST OF SLIDES

1. Advantages of the Design
2. Purposes of the Test Program
3. Shop Assembly of Aluminum Structure
4. Transportation of Sub-Assemblies
5. First Unit Placed on End Supports
6. Placing Second Bottom Plate
7. Placing Third Triangular Beam
8. Pouring the Concrete Deck
9. The Test Program
10. The Test Set-up
11. Deflection Gages
12. Comparison of AASHO and Test Loading
13. Comparison of Predicted and Measured Deflections
14. Comparison of Predicted and Measured Stresses in Bottom Longitudinal Members (Concentric Loading)
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18. Comparison of Predicted and Measured Shear Stresses (Eccentric Loading)
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