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EFFECTIVE FLANGE WIDTH AND LIVE LOAD DISTRIBUTION FACTOR FOR CONCRETE-FILLED STEEL GRID DECK

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ABSTRACT

As part of a five-year test plan for studying the behavior of the concrete-filled steel grid deck, a full-scale test was performed on a three-span deck truss bridge with superstructure consisting of stringers and floorbeams with a concrete-filled steel grid deck. Steel and concrete strain gauges were installed on the stringers, on the main bars of the grid, and in the concrete deck. A loaded two-axle truck was positioned at different locations on the deck and measured strains were recorded. Actual stresses in the stringers were computed using the recorded strains. They were then compared to theoretical stresses computed assuming effective slab widths equal to twelve times the slab thickness and center-to-center distance of the stringers. Also, compression in the deck was measured at different locations to show the participation of the deck in composite action with the stringers. The results of this test show that the effective slab for composite action of this deck and stringers is equal to the center-to-center distance between the stringers, 10" greater than 12T.

The distribution of wheel loads to the stringers was computed using the test results and compared with the AASHTO live load distribution factors for grid decks, and to NCHRP Report No. 12-26/1 for conventional reinforced concrete decks. Results show good correlation with the NCHRP report and that the AASHTO equation is very conservative.

INTRODUCTION

In the design of the superstructure components of bridges such as girders, stringers, and floorbeams, it is common to take advantage of the deck system by constructing the deck to act compositely with superstructure elements. This is accomplished by use of a mechanical connection between the deck and superstructure, such as shear studs. The American Association of State Highway and Transportation Officials (AASHTO) "Standard Specifications for Highway Bridges, 15th Edition" covers effective flange width in Section 10.38.3.1, stating that "In composite girder construction, the assumed effective width of the slab as a T-beam flange shall not exceed the following:

(1) One-fourth of the span length of the girder.

(2) The distance center to center of girders.

(3) Twelve times the least thickness of the slab."

Although AASHTO is clear in its statement and does not discriminate between different deck systems such as reinforced concrete and concrete-filled steel grid decks, some bridge owners and engineers do not take advantage of a grid deck's contribution when designing or analyzing the superstructure elements. Using common engineering principles, it can be shown that a concrete-filled steel grid deck should contribute to the composite action at least as much as a reinforced concrete deck of equivalent thickness.

One of the main reasons some bridge owners or engineers do not utilize the concrete-filled steel grid deck's contribution to composite action is the lack of research, testing, and published results in this area. Except for research conducted by U.S. Steel Corporation[1] in the 1950s, little testing has been performed.

Another effect of decks on superstructure behavior is the live load distribution to the stringers. AASHTO covers live load distribution to longitudinal beams in Section 3.23.2. Table 3.23.1 contains several distribution factors for use with steel grid decks, based on various factors such as beam spacing, grid depth (thickness), and number of traffic lanes. However, this article does not differentiate between open grid and concrete-filled grid deck systems. Recent tests[2], on concrete-filled steel grid decks, at the University of Pittsburgh show considerable plate behavior and, therefore, considerable live load distribution.

OBJECTIVE

The objective of this project is to utilize field testing results to develop the following items for concrete-filled steel grid decks:

1. The effective deck width (for composite design of supporting members such as girders, stringers and floorbeams)

2. Distribution of live load to longitudinal beams.

BRIDGE DESCRIPTION

The field testing was performed on the North Main Street Bridge, which spans the Cuyahoga River between Akron and Cuyahoga Falls, Ohio. The bridge is owned and maintained by the County of Summit.

The bridge was built in 1948 and consists of a three-span riveted deck truss carrying four traffic lanes and two sidewalks. (See Photo No. 1) The original roadway floor system was comprised of a 5" steel open grid deck, supported by rolled I-beam stringers and welded floorbeams.

In 1992, a rehabilitation project commenced, consisting of deck replacement and superstructure strengthening. The existing open grid deck was replaced with a 4-1/4" concrete-filled grid (see Photo No. 2) with a 1-1/4" monolithic concrete overfill and a 1/4" epoxy-urethane wearing surface. Lightweight concrete was utilized and the steel grid was epoxy-coated. The original simple-span stringers were also replaced with two-span continuous stringers installed at a wider spacing (6′ 4-1/2"). The new stringers have been installed with shear studs, but were designed as non-composite.

At the time of testing, the epoxy wearing surface had not been installed.

TESTING PROGRAM

Strain Gauge Installation

The test was located at a point 12′ south of Panel Point 21 (see Figures 1 and 5) on the eastern half of the bridge. Since the floorbeams are spaced at 30′, this location corresponds to the point of maximum moment in the stringers (0.4L).

A total of 12 steel strain gauges were installed on the stringers and nine concrete gauges were installed in the concrete fill. Additional gauges were placed on the steel grid, but were not utilized in this study. Those results will be presented in another report.

Gauges W-1 through W-12 were attached to Stringers 2 through 5 as shown on Figure 2. Installation was performed prior to erection of the stringers due to access limitations, except for the gauges on Stringer 5. The lead wires were pulled from the gauges to a point near the bridge centerline.

Gauges K-1 through K-9 were placed in the concrete fill, parallel with the bridge stringers, between Stingers 2 through 4 as shown on Figure 2. The lead wires were installed below the form pans and extended to a point near the bridge centerline.

Test Vehicle

A two-axle dump truck, furnished by Summit

County, was utilized for the live load testing. The truck was loaded with a rear axle weight of 21,160 lbs. See Figure 3 for axle and wheel spacing.

Test Procedure

The testing was performed in May, 1993. The bridge was closed to traffic due to the rehabilitation work; therefore, the test vehicle was the only live load in the vicinity of the strain gauges.

For this phase of the test, the truck was placed in three stationary positions (see Figures 4 and 5 for locations) and strain gauge readings were recorded for each position.

TEST DATA EVALUATION - EFFECTIVE SLAB WIDTH

Calculation of Stresses

The actual stresses were calculated from the field data in the following manner. The net strain gauge reading (loaded - initial) was multiplied by an adjustment coefficient. This adjustment takes into account the length of the wire as well as the gauge factor. The result of this calculation was the strain in the component, which was then multiplied by the modulus of elasticity to obtain the stress. The results for the stringer gauges are summarized in Tables 1-3.

The extreme fiber stresses in the stringers were then calculated by using linear stress distribution from the gauge locations (see Figure 2) and are shown in Table 4.

COMPARISON OF RESULTS - EFFECTIVE SLAB WIDTH

Theoretical Stress Calculation

For comparison with the test results, the theoretical live load stress in the bottom flange of the stringer was calculated for composite action using two different effective slab widths :

1. Twelve (12) times the deck thickness (5-1/2") = 5′ 6"

2. Center-to-center of stringers = 6′ 4-1/2"

The stresses were calculated at the test location using the actual wheel loads, placed with the heavy axle at 0.4L, multiplied by the AASHTO live load distribution factor (S/6.0). The results are shown in Table 6.

Comparison of Results

A comparison of the theoretical values versus the measured values indicates that the measured stresses are less than the theoretical stresses for both cases (see Table 6 for a summary).

TEST DATA EVALUATION - LIVE LOAD DISTRIBUTION

Calculation of Actual Live Load Stresses

The live load distribution factor was studied using the measured stresses of the bottom flange from Table 4 and utilizing composite section modulus for the bottom flange from Table 5 in the case of center-to-center of stringers. First, moments in each stringer at the locations of the gauges were computed by multiplying the section modulus by the measured stress for each live load position. It was assumed that these moments result from an unknown load "P" applied 12′ (location of gauges) from the first support of two-span continuous beams. Load "P" is that portion of a wheel load acting on the stringer. Computed loads "P" are shown in Table 7. As mentioned, the live load distribution according to AASHTO is S/6.0 for steel grid decks.

Comparison of Results

According to AASHTO, the portion of the live load wheel that acts on the stringer is 1.06 (S/6.0 = 6.375/6). The largest load in Table 7 is 6.937 kips, which is approximately S/9.5 of the average wheel load (10.580 kips) of the test truck.

CONCLUSIONS AND RECOMMENDATIONS

As shown at different live load positions in Table 4, Stringers S2 through S4 are in tension in their entirety when the test vehicle is positioned on top of these stringers, except for vehicle position 3, for which Stringer S2 shows small compression. This indicates that the neutral axis of the section that carries the vehicle load is above the top flange of the steel section, while Table 5 (theoretical section properties) shows that the neutral axis of the composite section, based on effective slab of center-to-center of stringers, is approximately 2" below the top of the top flange. The fact that there is full composite action between the stringers and deck slab, with effective flange equal to center-to-center distance, is also evident in Table 6, where theoretical and measured stresses are compared. Figure 6 also indicates concrete strain gauge readings showing relatively significant compression stress across the deck.

Results of this test corroborate the 1960 U.S. Steel test[1] and indicate that, when properly designed, there is full composite action between the concrete-filled steel grid deck and supporting superstructure elements of bridges. For this test, the effective flange width for composite action was the center-to-center distance between stringers, which is 10" greater than the 12T rule. Since there are many grid designs, it is recognized that this difference between the two rules of thumb (12T; center-to-center) may not hold true for every grid deck system. More tests are required to establish the upper limit. It is, therefore, recommended that at least 12T of deck be considered for composite action.

Evaluation of the Table 7 stresses shows that the concrete-filled steel grid deck has much better wheel load distribution than specified in the current AASHTO specifications. This is due to the fact that the ratio of steel is high in this deck system, which makes it relatively stiff. This was clearly shown by the tests conducted at the University of Pittsburgh. Note that for the tested bridge, there was a diaphragm (channel shape) between the stringers 3′ away from the location of the wheel load. This diaphragm makes a contribution to the wheel load distribution; however, based on the largest stresses measured and conservatively assuming the effective slab width of center-to-center of the stringers, the wheel load distribution factor is smaller than S/9.5. This distribution factor is approximately 20% smaller than the wheel load distribution factor recommended by NCHRP Report No. 12-26/1 for a conventional concrete deck of equal thickness. This difference is due to the relatively higher stiffness of this product, as previously mentioned. Therefore, it is recommended that, until further investigation, the same live load distribution recommended by the NCHRP report for a conventional concrete deck also be utilized for concrete-filled steel grid decks, as a minimum.

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REFERENCES

(1) "Test Data Showing Full Composite Action Between Concrete Filled Steel Grid Deck and Supporting Stringers", U.S. Steel Corp., August 29, 1960.

(2) "Plate Stiffness Summary and Strain Measurements on Grid Decks", C.P. Mangelsdorf, University of Pittsburgh Dept. of Civil Engineering, December 1991