Unintended Composite Action in Highway Bridges

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All available data on unintended composite action in beam-and-slab bridges are reviewed and the factors that may influence the existence of unintended composite action in noncompositely built beam-and-slab bridges are investigated. Test reports summarized in this paper have shown that the existence of a natural or a chemical bond is the single most important factor in determining whether a noncompositely built beam-and-slab system can be counted on to act compositely. There is considerable evidence that indicates the presence of composite action in bridges in which no provisions were made for such action. This composite action may reduce the stress under a given load by a significant amount. However, the uncertainty surrounding the presence of composite behavior and the difficulty associated with verifying the existence of composite behavior make the assumption of composite behavior in a bridge designed noncompositely a questionable one.

An opinion that appears to be widely held by structural engineers involved in the design or the rating of bridges is that composite action will exist in a steel beam–concrete slab bridge whether such action was provided for in the design of the bridge or not. Bridge 4 in the Tennessee bridge tests (*1*) was a steel girder bridge that was designed to act noncompositely but that, in fact, acted compositely up to the load at which yield in the steel girders would have occurred in the noncomposite bridge. This rather widely publicized result has contributed to the widely held opinion just described. Unfortunately, consideration of all available data does not permit the drawing of a general conclusion in this regard.

PURPOSE AND SCOPE

The purpose of this paper is to review all available data on unintended composite action in beam-and-slab highway bridges, to investigate the factors that may influence the existence of unintended composite action, and to make recommendations concerning the consideration of this phenomenon that the writers hope will prove useful to an engineer charged with the task of determining the load capacity of bridges.

Although some discussion of tests performed on other types of bridges is included, this paper is primarily concerned with concrete slab–steel beam bridges designed and built with no provision for composite action.

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REVIEW OF TESTS ON BEAM-AND-SLAB BRIDGES

This section contains a review of tests done on beam-and-slab bridge systems in which unintended composite action was considered. For each test, the bridge is described briefly, and the significant test results as related to unintended composite action are presented. The names of the authors are given for each case and the date when the test was performed is given in brackets.

Burdette and Goodpasture [1970]

One of the four full-scale bridges that was tested to failure by Burdette and Goodpasture (*1*) was a noncomposite, three-span continuous, concrete slab-and-steel-beam bridge (designated Bridge 4). A 7-in. (17.8-cm) slab was supported by four 27-in. (68.6-cm) steel rolled beams. The ultimate load in the actual test was compared with the ultimate load computed in a manner consistent with the AASHTO specifications and with a theoretical ultimate load considering the entire cross section as a wide beam.

Figure 1 shows the load-deflection curve for Bridge 4. The computed load-deflection curve was first developed assuming no composite action of the girders and bridge deck. Observation of the actual test and resulting strain data indicated that a considerable degree of composite action did exist at load levels approaching the load that would cause yielding of the steel in the noncomposite bridge. The bond between steel and concrete and the friction forces developed at the steel-concrete interface were sufficient to develop forces that resulted in composite action of the girders and deck. The average shearing stress at the steel-concrete interface was approximately 230 psi (1586 kPa) at a load of 500 kips (2225 kN). The load-deflection curve calculated on the basis of composite action in the elastic range matched the measured data almost perfectly up to a load near the capacity of the noncomposite bridge.

Kissane [1985]

Full-scale laboratory and field testing was performed to determine the restraint to elastic buckling of a steel beam supporting a noncomposite concrete deck (*2*). In the laboratory testing, a reinforced concrete slab 6 in. (15.2 cm) thick was placed on top of an American Standard Beam. Before the

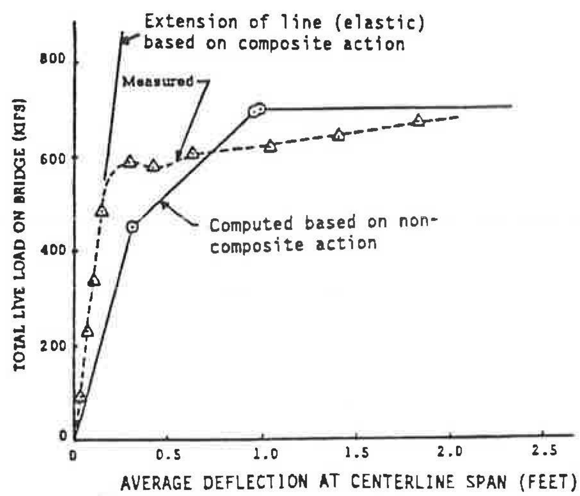

FIGURE 1 Comparison of measured and computed load-deflection curves for Bridge 4 (*1*).

concrete slab was poured, the top flange of the beam was sanded to remove mill scale and irregularities that could offer mechanical restraint at the concrete-steel interface. The bare metal surface of the flange was covered with a light oil to prevent chemical bonding between the concrete and steel.

The ratio of compression to tension flange strain was used as an indicator of composite behavior between the concrete and steel. Experimental data show that there existed some amount of composite behavior, which increased as higher loads were applied. The amount of composite behavior translates to an approximately 10 percent increase in the bottom flange section modulus above that for a noncomposite value.

Because mechanical and chemical bonds were inhibited between the steel and concrete, the partial composite behavior was attributed to friction between these components. As higher loads were applied, there was an increase in friction and pressure developed at the steel-concrete interface, thereby permitting increased composite behavior.

A field test was also performed on a 146-ft (44.5-m) simple span truss bridge built in 1924. Two identically loaded haul trucks were used to load the bridge. Results showed that there was a small change in neutral axis position from that of a noncomposite section. However, the upward shift of 0.82 in. (2.08 cm) in the neutral axis position was considered insignificant because of the low magnitude of the measured stresses and the approximately 300 psi (2068 kPa) experimental error in the measurement. The author concluded that there was no significant evidence of composite behavior between the steel and concrete (*2*).

Bakht and Csagoly [1975]

An extensive diagnostic load test was undertaken (*3*) to detect the sources of distress in the Perley Bridge, where in March 1973 a failure of the connecting angles of the girder-column connection of a trestle span caused the span to drop a few inches. One of the objectives of the diagnostic testing of the trestle span was to determine the degree of composite action that existed between the floor beams and the concrete deck slab in the absence of shear connectors.

Approximately 87 percent of the deck slab area was supported by girders and trusses through the floor beams. The test established that the composite action between the slab and the floor beams varied from beam to beam and could not be relied upon with certainty.

AASHO Road Test [1962]

The AASHO Road Test (*4*) included a study of 18 beam-and-slab bridges. Each bridge was a simple span structure consisting of three beams and a reinforced concrete slab. The beams spanned 50 ft (15.2 m). Ten of these bridges had wide-flange, rolled-steel beams with or without cover plates. Two of the 10 steel bridges were built compositely with shear connectors, whereas the other 8 were noncomposite. The top surfaces of the steel beams in the noncomposite bridges were coated with a 1:4.43 mixture of graphite and linseed oil to inhibit formation of bond. In composite bridges, the interaction between the slab and the steel beams was obtained with 4-in. (10.2-cm), 7.25-lb (3.29-kg) channels 5.5 in. (14.0 cm) long welded to the top flanges.

In the design of the beams, two of the steel bridges were assumed to have complete interaction between the slab and the beams, whereas six of the bridges were assumed to have 10 percent composite action to account for the effects of friction between the slab and the beams. No composite action was assumed for the remaining two bridges in the design.

After the bridges had been tested with repeated stresses, the actual locations of the neutral axes were determined from strains measured on the bottom and top of beams at midspan, assuming straight line strain distribution. In calculating the theoretical location of the neutral axis, the compositely built steel bridges were considered fully composite; for the other steel bridges a complete absence of interaction between the slab and the beams was assumed. Table 1 gives the distance of the experimental location for all 30-mph runs within the 10-month observation period.

The difference between the measured and the theoretical locations of the neutral axis was small, indicating that the bridges with mechanical connectors were fully composite, whereas the others had practically no composite action. The moment-deflection diagrams also showed a much stiffer sec-

TABLE 1 DIFFERENCE BETWEEN EXPERIMENTAL AND THEORETICAL LOCATION OF NEUTRAL AXIS (*4*)

Bridge	Location of * Neutral Axis (inch)
1A, NC	0.0
1B, NC	+0.2
2A, NC	+0.1
2B, C	+0.2
3A, NC	+0.2
3B, C	-0.1
4A, NC	+0.2
4B, NC	-0.1
9A, NC	+0.1
9B, NC	0.0

NC - Non-Composite, C - Composite
*Plus sign indicates position above theoretical

tion for the composite beams; for example, Bridges 1A and 3B had beams of the same depth, but for a midspan moment of 500 kip-ft (678 kN-m), the deflection at midspan for Bridge 1A was 2.2 in. (5.59 cm), whereas Bridge 3B had only 0.6 in. (1.52 cm) of deflection. The report concluded that there was no composite action present for the bridges with no shear connectors.

Thomas [1949]

A 1:3 scale model of a bridge deck system was tested to determine the extent of lateral load distribution of the system and the extent to which the steel joists and the concrete slab acted together in resisting the induced bending moments (*5*). The model consisted of a reinforced-concrete deck 3 in. (7.62 cm) thick supported on six standard 8-in. (20.3-cm) by 4-in. (10.2-cm) longitudinal steel beams spaced at 3-ft (0.914-m) centers. The slab was cast independently of the steel joist system to minimize bond induced by construction methods.

Load was applied to the model by means of a hydraulic jack. The tests were made in three stages: load applied to the slab in its uncracked condition, load applied to the slab after it had been systematically cracked, and final loading to failure.

For the test on the uncracked model, a comparative study of the strains measured in the upper and lower flanges of the beams showed that some partial composite action existed between the beams and the slab. The effect was more pronounced for beams near the load, suggesting that composite action was primarily due to friction between the slab and the flanges of the beams. For the test on the cracked model, a study of strain measurements indicated that there was slightly less composite action between the beams and the slab as a result of cracking. The slab failed by punching shear at a load of 45.1 kips (201 kN). The author concluded that shear connectors should be used if full allowance of composite action is to be made.

Siess and Viest [1945]

Laboratory tests were made on three 1:4 scale models of continuous right I-beam bridges (*6*). Each structure was a two-span right bridge consisting of five steel beams supporting a reinforced-concrete slab. The bridges were labeled N30, C30, and X30. Bridge N30 was a noncomposite bridge, Bridge C30 had shear connectors welded to the top flanges of the I-beams at regular intervals throughout the full length of the bridge, and Bridge X30 had no shear connectors in the negative moment region in the vicinity of the center pier. The top flanges of the beams for Bridge N30 were covered with a coating of wax to prevent a bond between the slab and the beams. For Bridges C30 and X30, the top surfaces of the beams were left in the as-rolled condition. All tests were made with one, two, or four pairs of concentrated loads simulating the rear-axle loads of one or more trucks.

For the test on Bridge N30, it was observed that the bottom and top flange strains were practically equal for applied loads at midspan, an indication that there was no interaction between the slab and the beams. The test data are also in agreement with the calculated values. For the test on Bridges C30 and X30, observation of strain data indicated the presence of composite action for both bridges as expected. Bridges X30 and C30 failed ultimately by punching of the slabs at a load of 10.4 kips (46.3 kN), and Bridge N30 failed by buckling of the beams at a load of 8.49 kips (37.8 kN), even though Bridge N30 had larger beams. On the basis of the test results, it was concluded that the behavior of Bridge N30 was that of a truly noncomposite structure, and Bridges X30 and C30 acted like composite structures.

Tharmabala [1984]

A structural evaluation and load test studies were performed (*7*) on the Flack River Bridge, which consists of pony trusses spanning 70 ft (21.3 m) with a concrete deck cast over stringers and floor beams. Two heavily loaded trucks were used to induce member forces to reach ultimate limit states defined by the Ontario Highway Bridge Design Code (OHBDC). The stringer moments were calculated using measured strains on the bottom flange and both composite and noncomposite section properties. These moments were compared with the theoretical moments using grid analysis. Although the deck is of noncomposite construction, it was observed that the moments obtained with composite section properties were closer to the theoretical moment graphs. Therefore, it was concluded that the deck behaved in a partially composite mode under the applied loads.

Patel [1984]

A structural evaluation and live load test were performed (*8*) on the Irvine Creek Bridge, which is a two-lane steel truss bridge with a sidewalk and two identical steel trusses spanning 104 ft (31.7 m) over Irvine Creek. Two heavily loaded trucks were used for vehicular test loading. The stringer moments were calculated using grillage analysis of the deck under applied loading. Plots were made of corresponding test moments calculated using measured strains with composite and noncomposite section properties of the stringers versus theoretical stringer moments.

It was observed that the graphs for the theoretical moment values lay between the graphs of the moments calculated using the fully composite and noncomposite section properties, indicating that the stringer sections were acting partially composite with the concrete deck. From the strain data on the floor beams, it was observed that for each loading case the floor beam moments obtained from grid analysis compared very well with the measured moments using composite section properties. Even though the steel floor beams were built to be noncomposite with the concrete deck, they were found to act compositely because they carried higher applied loads than the steel beams could have carried by themselves.

REVIEW OF TESTS ON INDIVIDUAL BEAM-AND-SLAB SYSTEMS

This section contains a review of tests done on beam-and-slab systems in which unintended composite action was observed.

These tests are reviewed separately from those in the preceding section because the tests reported here deal with a single beam and slab as opposed to a multi-beam-and-slab bridge system. A brief description of the test and the test results are presented for each case.

Viest et al. [1948]

Four composite steel-and-concrete T-beams with channel shear connectors were tested on simple spans of 37.5 ft (11.4 m) by applying single concentrated loads at the center line and at other points (*9*). In three of the beams tested, provision was made during construction to prevent bonding between the slab and the steel beam to better observe the behavior of the shear connectors. However, one beam was allowed to develop natural bond in order to determine the effectiveness of bond in transmitting horizontal shear. The results of the first test made with this beam revealed the presence of bond. Only after 11 repetitions of a load of 40 kips (178 kN) was the bond broken. The test results showed that as long as bond was present, it proved to be an effective shear connection. Before bond was broken, there was practically no slip between the slab and the beam. The bond withstood a load equal to 1.7 times the design live load, corresponding to a shearing stress of 112 psi (772 kPa). The bond broke after the same load was applied 11 times. However, the researchers believed that if the static load had been increased instead of repeated, at loads approaching ultimate, large deformational bond stresses at the concrete-steel interface would have caused bond failure. Further, they noted that shrinkage and warping of the slab as well as dynamic loading might destroy the bond even at working loads. It was concluded by the researchers that even though bond is a very good shear connection, it may also be an unreliable one.

Viest [1960]

A review of research on composite steel-concrete beams was done by Viest (*10*) on all tests carried out in the period between 1920 and 1958. Tests of specimens with and without mechanical shear connectors were summarized and briefly described. Practically every test that was done led to the conclusion that so long as bond between the concrete and the steel was not definitely broken, complete interaction between the slab and the beam could be assumed. Bond strengths between 400 and 500 psi (2758 and 3448 kPa) were observed in some of the tests. However, it should be noted that most of these tests were pull-out tests on steel beams fully encased in concrete.

One of the first American tests was performed by Caughey. On the basis of his test results and tests published before 1929, he recommended an allowable bond stress of $0.03f'_c$. Viest recommended an allowable bond stress of 60 psi (414 kPa) when the steel beam was fully encased and 50 psi (345 kPa) when the steel beam was only partially encased.

Bryson and Mathey [1962]

An extensive test of bond between concrete and steel beams was performed by the National Bureau of Standards (*11*). Wide-flange structural steel beams with different surface conditions were embedded in concrete and subjected to push-out tests to determine the effect of surface condition on the bond between concrete and steel. Three types of surface conditions were studied: normal rust and mill scale, sandblasted and allowed to rust, and freshly sandblasted. Three push-out specimens for each surface condition were constructed. The specimens were either W 14 × 30 or W 14 × 34 steel sections embedded in 2 ft (61.0 cm) of concrete. The bonded area of the steel beam was limited to the surface of the flanges.

A summary of the test results is shown in Table 2. The results indicated a considerable difference in ultimate strength of the bonds. However, at low values of slip, bond stress was not greatly affected.

SUMMARY

Test reports summarized in this paper have shown that the existence of natural or chemical bond is the single most impor-

TABLE 2 SUMMARY OF TEST RESULTS (*11*)

Specimen	Steel Section	Surface Condition	Maximum bond stress, psi	Free end slip at maximum, inch
SB-1	14 WF 34	freshly sandblasted	420	0.015
SB-2	14 WF 30	"	474	0.008
SB-3	14 WF 34	"	470	0.006
Avg.			455	0.010
R-1	14 WF 30	sandblasted and allowed to rust	508	0.020
R-2	14 WF 34	"	403	0.018
R-3	14 WF 34	"	486	0.007
Avg.			466	0.015
N-1	14 WF 30	Normal rust and mill scale	310	0.003
N-2	14 WF 34	"	355	0.012
N-3	14 WF 30	"	287	0.002
Avg.			317	0.006

tant factor in determining whether a noncompositely built beam-and-slab system can be counted on to act compositely. Every reported test, be it full-scale or model, in which bond between the steel and concrete interface was prevented resulted in noncomposite behavior, with the exception of the laboratory test that was reported by Kissane. Caughey and Viest, Fountain, and Singleton suggested allowable design bond stresses of $0.03f'_c$ and 50 psi (345 kPa), respectively. Though not quantitatively stated, a large factor of safety was implied in all design stresses. In the full-scale tests by Viest et al. (*9*) at Illinois, the beam in which bond was not broken experienced full composite action at the maximum design load of 1.7 times the live load. The bond failed after 11 repetitions of the load. A horizontal shear stress of 112 psi (772 kPa) was resisted by bond at this loading.

Even though bond has been shown to be very effective in transmitting horizontal shear, it is also unreliable, because it is sensitive to fatigue loading, shrinkage, thermal stresses, and impact. There is also the possibility of physical separation of the concrete slab and the steel beam because of uplift forces generated along the beam by certain dispositions of the live load. These uplift forces may also contribute to the rapid deterioration of the natural bond at the steel-concrete interface. It was also stated by Viest et al. that shrinkage and warping of the slab as well as dynamic loading may destroy the bond even at working loads.

Tests have also shown that composite action could be induced by friction. However, the amount of friction that translates into a certain degree of composite action varies from one bridge to another depending on the weight of the deck slab, the magnitude of the load, and the surface roughness of the steel-concrete interface.

From the data obtained in the tests reviewed herein, it appears that even though certain beam-and-slab bridge systems have demonstrated the ability to act compositely without the use of mechanical shear connectors, the degree to which composite action can be counted on is very difficult to quantify because of the variables just discussed.

CONCLUSIONS

The work described in this paper leads to the following conclusions, which are presented in the form of statements that may be useful to a bridge engineer charged with calculating the capacity of a bridge.

First, composite action in a bridge designed to act noncompositely cannot be counted on to increase the load capacity of a bridge significantly if that capacity is defined as the load producing first yield of tensile steel. In Tennessee Bridge 4 the load capacity was simply the sum of the capacities of the four steel girders. The path from zero load to yield, on the other hand, was significantly affected by the presence of composite action. The bridge was effectively much stiffer than that calculated on the basis of noncomposite behavior; thus, at any load level below yield, the deflection was much less than that calculated. This behavior relates also to stresses: it follows that the stress in the steel girders at any load below yield in Tennessee Bridge 4 was less than that calculated on the basis of noncomposite behavior. Therefore, even though unintended composite action does not appreciably enhance load capacity, in the process of altering the behavior of a bridge in the elastic range, it does reduce the stress in the girders at any load level. This reduction in stress for a given load may very well prove to be of significant benefit to a bridge engineer concerned with bridge load capacity.

Second, the discussion just presented was based on the existence of unintended composite action like that found in Tennessee Bridge 4. In fact, the work presented in this paper leads to the conclusion that composite behavior in a bridge designed with no provision for composite action may or may not occur. In some cases the practical and conservative approach is to disregard any potential unintended composite behavior. For the cases in which such an assumption leads to calculated stress levels for certain required loads that are larger than those permitted, further investigation to verify the presence of composite action may be justified. Such an investigation would logically begin with an inspection of the bridge by an experienced bridge engineer—an inspection that would include visual observation of the beam-slab interface as truck traffic goes over the bridge. Details of construction should be observed; for example, encasement of the top flanges of the steel girders would increase the probability of composite action. This investigation may very well prove to be inconclusive. In such a case the next step might be to load the bridge statically with a heavily loaded truck of known dimensions and weight and to measure deflections of the bridge at selected points. Comparison of the measured deflections with deflections calculated on the basis of both noncomposite and composite action should shed considerable light on the question of whether the bridge is behaving compositely or noncompositely. If composite action is found to exist, this behavior can be considered in the calculation of stresses at various load levels below that used in the test. However, the possibility of sudden slippage at loads greater than the test load makes the assumption of composite action at higher load levels questionable.

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