Case Study of Concrete Deck Behavior Without Top Reinforcing Bars

Li Cao and P. Benson Shing *University of Colorado, Boulder*
John Allen, *Allen Research & Development Corporation*
Dave Woodham, *Colorado Department of Transportation*

A major cause of the deterioration of bridge decks is the spalling and delamination caused by the corrosion of the top mat of reinforcing bars. Empirical evidence has indicated that the tensile bending stresses developed at the top of a bridge deck subjected to traffic loads are relatively low. As a result the need for top reinforcing bars for sustaining the negative bending moment induced by traffic loads is questionable. To explore the possibility of eliminating top reinforcing bars, and thereby reducing the vulnerability to corrosion, the performance of a four-span bridge deck is investigated. In the bridge studied one span has an experimental deck with no top reinforcement, whereas the remaining spans have both top and bottom reinforcements that conform to AASHTO specifications. The response of the bridge deck under a test truck was monitored with embedded strain gauges. It was found that the peak transverse tensile strains developed at the top of the deck were less than 30 percent of the cracking strain of the deck concrete. The behavior of the bridge deck under the test truck and other combinations of truck loads has also been investigated by means of elastic finite-element analysis. The results show that the tensile stresses developed at the top of the deck tend to be much less than the modulus of rupture of the deck concrete. The study confirms that a properly designed bridge deck does not require the top reinforcement for sustaining the negative bending moment induced by traffic loads.

The deterioration of bridges in the United States is a serious problem. As bridges age, repair and replacement needs accrue. It has been estimated that 41 percent of the nation's 578,000 bridges are either structurally deficient or functionally obsolete (*1*). An estimated investment of $51 billion is needed to bring all of the nation's bridges to an acceptable and safe standard by either rehabilitation or replacement. On the basis of national bridge inventory data obtained from the U.S. Department of Transportation, about one third of the nation's bridges have deficient decks, whereas 24 percent of the nation's bridge decks are structurally deficient. An effective means of preventing such deterioration is to eliminate top reinforcing bars from a deck. This can lead to substantial savings in construction, maintenance, and repair.

To explore this new design concept an experimental deck was designed and constructed without top reinforcement for an end span of a four-span bridge by the Colorado Department of Transportation. The main objective of the study was to determine the maximum tensile stresses that can be developed in such a deck in order to assess its durability in the absence of top reinforcement. The investigation consisted of the development of a linearly elastic finite-element model for evaluating the response of the deck under truck loads and the monitoring of the actual response of the bridge deck under a test truck as well as normal traffic loads. Results of the study have been documented in detail by Li et al. (*2,3*). This paper summarizes the design of the experimental deck, the field tests, and the experimental and numerical results.

Bridge Deck Design

Background

In North America most short- and medium-span bridges are constructed with slab-on-girder decks, in which a reinforced concrete slab is supported by several steel or precast, prestressed concrete girders. Generally, the design of reinforced concrete decks has been based upon the Westergaard theory (*4*), which assumes that a slab is continuous over fixed linear supports. The current AASHTO slab design provisions (*5*) are based on empirical rules derived from earlier adaptations of the Westergaard theory (*6,7*).

According to the conventional design method, bridge slabs over three or more girders are designed as continuous slabs, which are assumed to have the same positive and negative bending moments that are 80 percent of the simple span moment specified in the AASHTO code (*5*). As a result nearly the same quantities of steel have been used to resist the positive and negative bending moments in a slab. Until the late 1960s bridge decks had a top concrete cover of only 1.5 in. (38 mm) and a bottom cover of 1.0 in. (25 mm). In the early 1970s the top cover was increased to at least 2.0 in. (51 mm) or 2.5 in. (64 mm) for situations in which deicing chemicals were used. It was thought that the additional cover would significantly delay the penetration of chlorides to the reinforcing bars.

The increased top cover did not extend the lives of bridge decks dramatically. In many states where salt usage was prevalent, decks have been made more impervious to moisture and salt penetration. Concrete mixes with a high percentage of cement are known to be more impervious, so that their use has become standard practice. To protect the top reinforcing bars several barrier techniques, such as the use of membranes, dense concrete, or latex-modified concrete, have been developed. These techniques have moderate success, but the use of epoxy-coated bars has proven to be the most effective and widely accepted strategy.

In spite of attempts to prevent the deterioration of bridge decks due to the corrosion of reinforcing bars, deck cracking has worsened dramatically. In recent years the incidence of transverse cracking has increased. It has been observed that transverse cracks, which appear shortly after deck placement, often occur over the upper reinforcing bars, permitting increased exposure to chlorides from deicing chemicals.

The need for top transverse reinforcing steel has recently been questioned by Allen (*8*). Investigations of the behaviors of bridge decks by Beal (*9*) and Fang et al. (*10*) have shown that the negative bending moments in bridge decks and the resulting transverse tensile stresses at the top of a deck are usually very low, much less than the positive bending moments and the bottom tensile stresses. Analysis of their work and other empirical evidence by Allen (*8*) indicates that the deflection of girders tends to significantly reduce the transverse bending stresses at the top of a deck. Hence, the best way to prevent the corrosion of reinforcing bars is to eliminate the top mat of reinforcing steel. Without top reinforcing bars the predominant cause of bridge deck deterioration can be eradicated. To explore the new design concept of removing the top reinforcement, an experimental deck was designed and constructed by the Colorado Department of Transportation (CDOT), as discussed in the following section.

Design and Configuration of Prototype Bridge

The bridge selected for the project described here is located on Colorado State Route 224 over the South Platte River near Commerce City. It is 420 ft (128 m) long and 52 ft (15.85 m) wide. The superstructure consists of four equal continuous spans. The supporting girders are standard precast Colorado Type G-54 girders spaced at approximately 8.0 ft (2.44 m) on center. The thickness of the bridge deck is 8.0 in. (203 mm), which complies with the new design requirement adopted by CDOT. The configurations of the four-span bridge and typical girder sections are shown in Figure 1.

In the four-span deck, the west end has an experimental deck, from which most of the top reinforcement has been omitted. It consists of the entire 104-ft (31.7-m) end span and 36 ft (10.97 m) of the adjacent interior span. The remaining deck has both top and bottom reinforcement that conforms to AASHTO specifications (*5*). The span at the east end serves as the control deck. The reinforcing steel details of the control and the experimental decks are shown in Figure 2. Both decks are instrumented with strain gauges.

In the midspan of the control deck, the top and bottom transverse reinforcement consists of No. 5 bars with a 5.5-in. (140-mm) center-to-center spacing. The top longitudinal reinforcement consists of No. 5 bars with an 18-in. (457-mm) center-to-center spacing, and the bottom longitudinal reinforcement consists of No. 5 bars with a 9.5-in. (241-mm) center-to-center spacing.

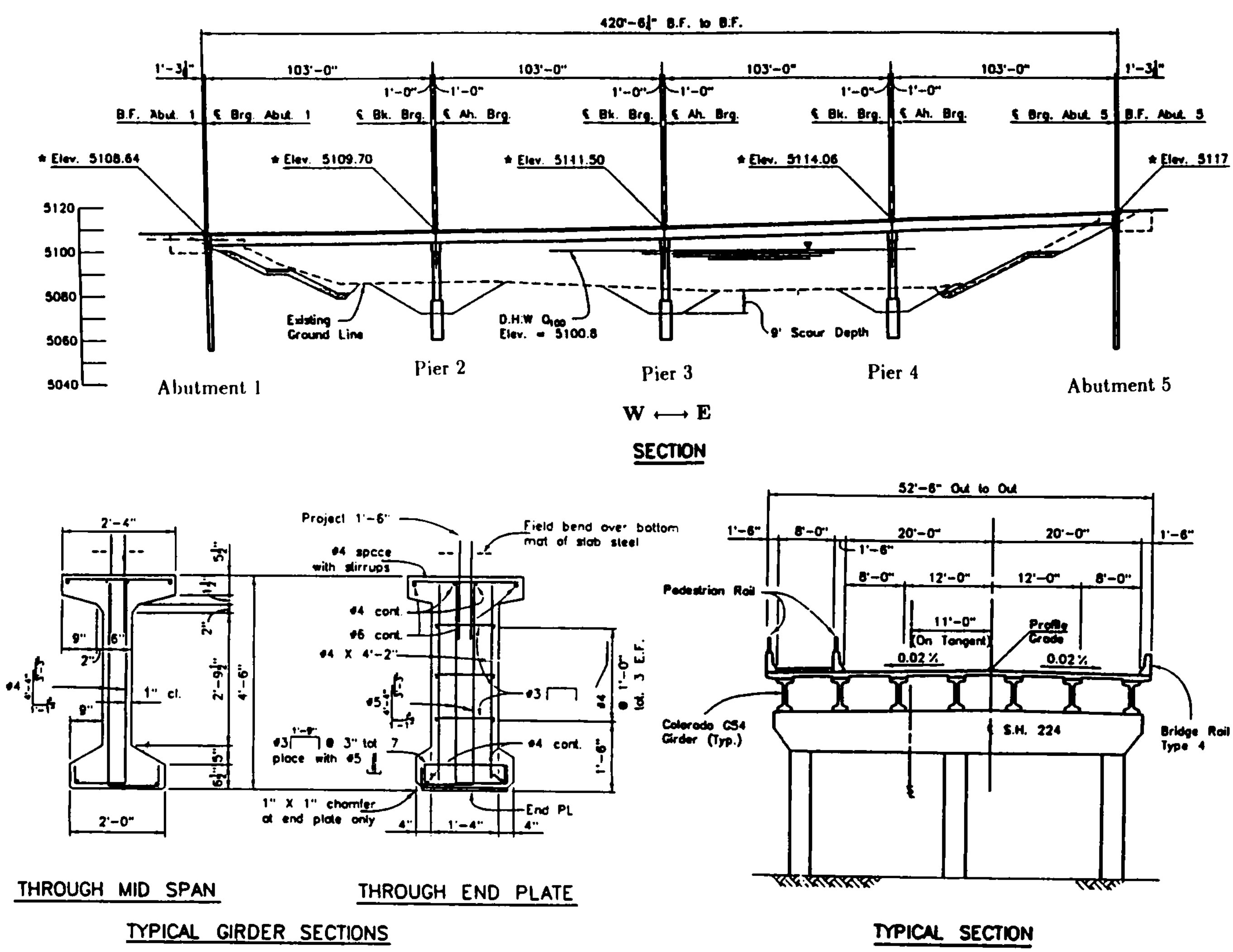


FIGURE 1 **Configuration of the bridge deck (1 in. = 25.4 mm).**

In the midspan of the experimental deck, the top reinforcement is removed. On the basis of an allowable stress of 24,000 lb/in.^{2} (165.36 MPa) for steel, this deck slab can sustain a positive transverse bending moment of 6,990 ft-lb/ft (31.1 kN-m/m). From the finite-element analysis of the bridge deck conducted by Li et al. (2), it has been found that the maximum transverse tensile stress that can be expected at the bottom of the deck is 550 lb/in.^{3} (3.80 MPa), with the deck thickness taken to be 7.5 in. (190 mm) on the basis of the preliminary design. This tensile stress corresponds to a positive bending moment of 5,160 ft-lb/ft (23.0 kN-m/m). According to AASHTO specifications (5), the design bending moment of the slab under an HS20 truck is 4,910 ft-lb/ft (22.0 kN-m/m), not including the continuity factor of 0.8. Hence, it has been concluded that the quantity of bottom transverse reinforcement adaopted here is sufficient to carry the design load, even if the slab is not continuous over the girders.

Furthermore, from the finite-element analysis (2), the maximum transverse tensile stress at the top of the deck above the center of a girder is expected to be 286 lb/in.^{2} (2.0 MPa). The maximum top transverse tensile stress at a point 12 in. (305 mm) away from the center of a girder, which is approximately above the edge of the flange of a girder, is expected to be 140 lb/in.^{2} (1.0 MPa) for a slab 7.5 in. (190 mm) thick. This tensile stress is reduced to 123 lb/in.^{2} (0.85 MPa) for a slab 8.0 in. (200 mm) thick, which is way below the expected tensile strength of the deck concrete. Hence, it has been concluded that no top transverse reinforcement is required to sustain the negative bending moment.

In both the control and experimental decks, the same amount of top longitudinal continuity reinforcement was placed. This consisted of 56-ft (17.1-m)-long No. 9 bars spaced at 9 in. (229 mm) on center across the piers. The top cover on these bars was 3 in. (76 mm)

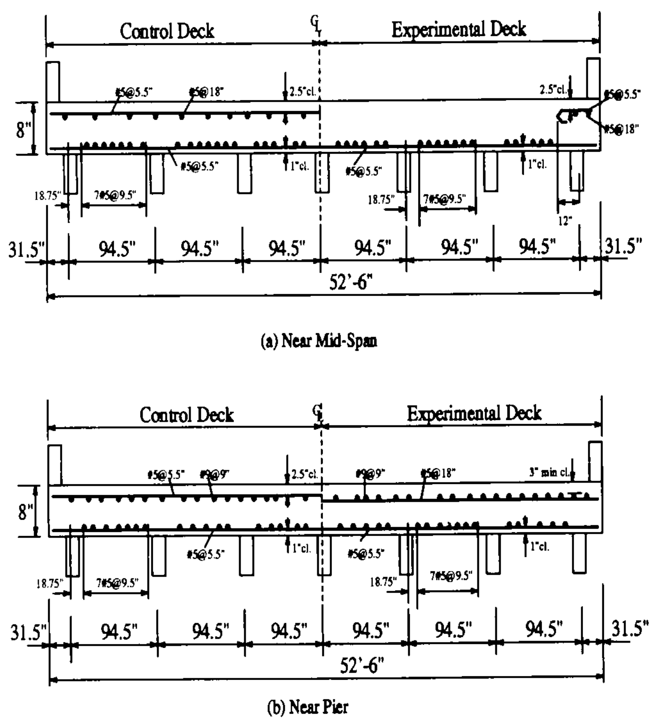


FIGURE 2 Reinforcing steel details for the bridge deck (not to scale; 1 in. = 25.4 mm).

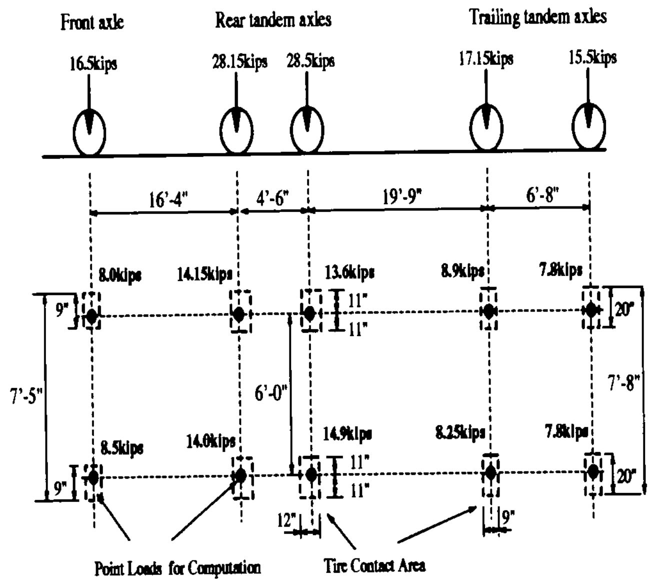


FIGURE 3 Test truck (1 kip = 4.45 kN, 1 in. = 25.4 mm).

in the experimental deck, so that these bars are at the same elevation as those in the control deck.

A small amount of polypropylene fiber (1.5 lb/yd^3 of concrete) was added to the deck concrete to help control cracking. The deck concrete was membrane cured. Based on 28-day laboratory-cured specimens, the average compressive strength and the modulus of rupture of the deck concrete are measured to be 5,740 and 590 lb/in.^{2} (39.6 and 4.1 MPa), respectively. The average compressive strength of the girder concrete is 8,500 lb/in.^{3} (58.6 MPa).

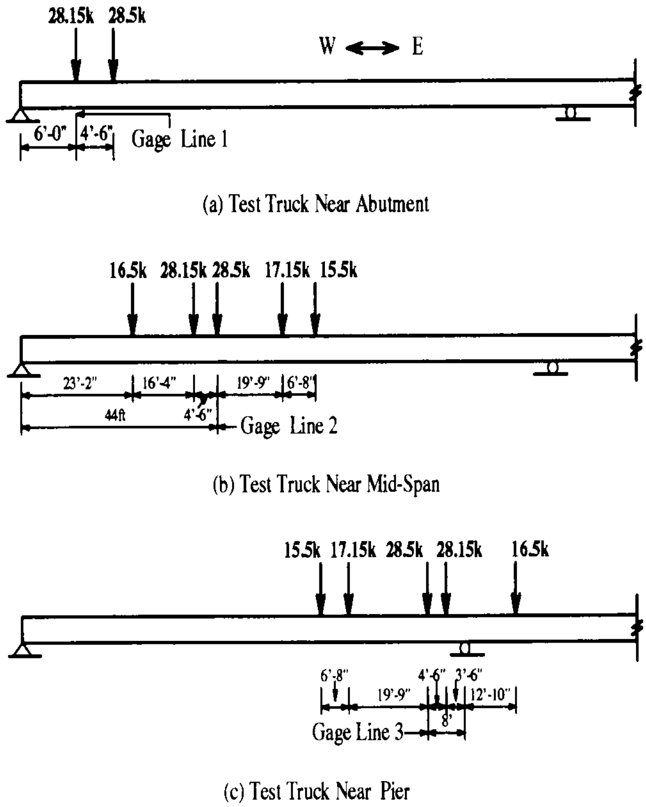
The bridge was constructed in two phases to facilitate the flow of traffic. The Phase 1 portion of the deck consists of a slab 34 ft (10.36 m) wide supported over five girders. It was cast in January 1993. The Phase 2 portion of the deck was cast in July 1993. After the bridge had been opened to traffic for 6 months, a series of load tests was conducted on a single day in September 1993, with the complete bridge temporarily closed to traffic.

Field Tests

Test Truck and Truck Load Positions

As shown in Figure 3, the test truck included a front axle transmitting a force of 16.5 kips (73 kN). The total force transmitted by the rear tandem axles of the test truck was 56.7 kips (252 kN), and the total force exerted by the trailing axles was 32.8 kips (146 kN). The total weight of the test truck was 106 kips (472 kN), which is 47 percent more than that of a conventional HS20 truck. The axle and wheel spacings of the test truck were similar to those of a standard HS20 truck.

To investigate the maximum tensile stresses that could be developed in the transverse direction at the top and bottom of the deck, it was decided that the test truck should be positioned at three different locations along the longitudinal direction of the bridge, as shown in Figure 4. The first truck position was close to the abutment in the experimental deck at the west end, resulting in rear tandem axle loads being approximately 8ft (2.44 m) away from the abutment. The deflection of the girders was small when the truck was at this position. The trailing axles and the front axle were not used in this load case, since it is expected that these axle loads would increase the girder deflections and thereby decrease the transverse tensile stresses at the top. The second truck position in the longitudinal direction was near the midspan of the experimental deck, with the resulting rear tandem axle loads being approximately 44 ft (13.41 m) away from the abutment. This induced differential deflections among the girders. The third truck load position in the longitudinal direction was in the vicinity of the pier in the experimental deck, with the resulting rear tandem axle loads being approximately 6 ft (1.83 m) away from the pier. The above positions are identified as Load Groups 1, 2, and 3, respectively. As illustrated in Figure 5, the wheels of the test truck were positioned at six to seven different locations along the transverse direction of the deck for each of these load groups.

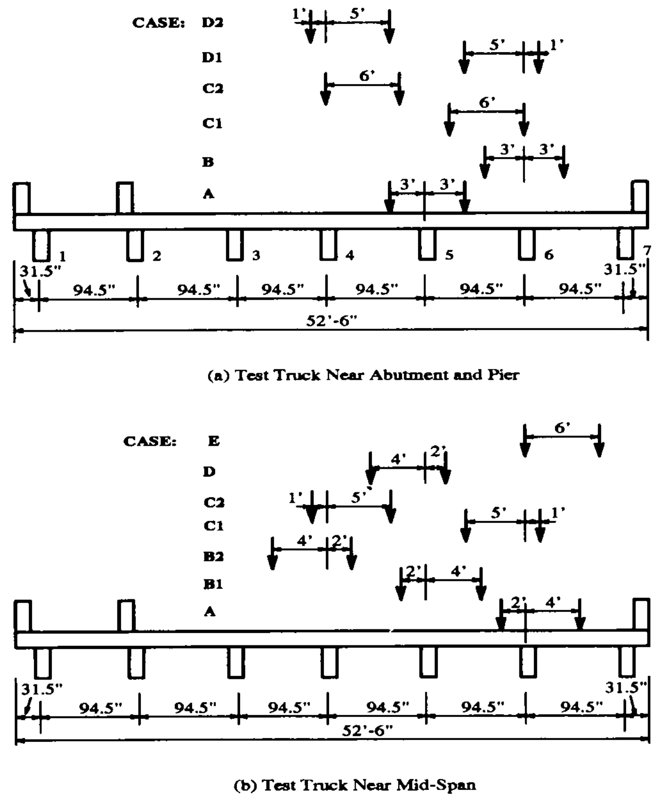


(a) Test Truck Near Abutment

(b) Test Truck Near Mid-Span

(c) Test Truck Near Pier

FIGURE 4 Longitudinal positions of test truck on the bridge (1 kip = 4.45 kN, 1 in. = 25.4 mm).



(a) Test Truck Near Abutment and Pier

(b) Test Truck Near Mid-Span

FIGURE 5 Test truck positions along transverse direction of the deck (1 in. = 25.4 mm).

The truck load positions were determined from the finite-element analysis (2). Each position results in a most severe stress condition when compared with the stress conditions at other positions in the vicinity. In addition to the three longitudinal positions, the test truck was also placed on the control deck. Load Groups 4 and 5 correspond to the midspan and abutment positions, respectively, on the control deck in the east span, which are similar to Load Groups 2 and 1, respectively.

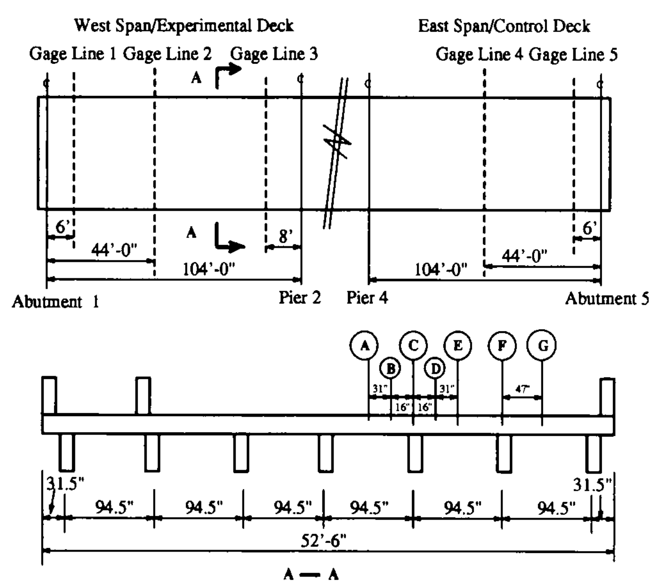


FIGURE 6 Locations of strain gauges in the bridge deck (1 in. = 25.4 mm).

Instrumentation

The response of the bridge deck under the test truck was monitored by strain gauges embedded at different locations in the deck. These locations are associated with the designated positions of the test truck discussed earlier. Five gauge lines are selected, as shown in Figure 6. The first three gauge lines are located in the experimental deck, and the other two are located in the control deck. In the experimental deck, the first and second gauge lines are 6 ft (1.83 m) and 44 ft (13.41 m) away from the abutment, respectively. The third gauge line is

TABLE 1 Maximum Strain Readings in the Transverse Direction at Top and Bottom of Slab

Gage Point	Gage Line 1	2	3	4	5
A	+3.1/+66.5	-52.3/+117.9	-53.3/+54.9	-/-	-/-
B	-/-31.5	-24.5/-	-/-	-/-	-/-
C	+20/-	+6.8/-	+19.2/-	+5.6/-	+13.8/-
D	+18.3/-	-/+50.7	-/-	-/-	-/-
E	-32.6/+76.7	-53.9/+173.8	-51.1/+73.4	-46.5/+133.2	-39.6/+30.2
F	+15.4/-	+13.0 /-	+18.7/-	-/-	+15.7/-
G	-14.8/+30.8	-/+176.2	-/-	-/-	-/-

Note: The plus and minus signs refer to the tensile and compressive strains, respectively. The locations of gage lines and gage points are illustrated in Fig. 6. The strain readings in each column are obtained under a load group which has the same number as the gage line.

8 ft (2.44 m) away from the pier. Gauge Lines 4 and 5 are located in the control deck.

There are seven gauge points (A through G) along each of the gauge lines, as shown in Figure 6. Each gauge point usually has top and bottom gauges, which are oriented in the transverse and longitudinal directions of the deck. The top and bottom gauges are about 1 in. (25 mm) away from the top and bottom surfaces of the deck, respectively. The strain gauges were welded on 21-in. (0.53-m)-long No. 4 bars that have anchoring hooks. These bars are embedded in concrete. The gauge mounting technique was verified with a reinforced concrete beam subjected to third-point loading (*3*).

Results of Field Tests

The response of the bridge deck to the test truck positioned at the different locations mentioned previously was measured by embedded strain gauges. The maximum strain readings at the top and bottom of the deck are summarized in Tables 1 and 2.

The modulus of rupture of the deck concrete was measured to be 590 lb/in.^{2} (4.1 MPa) with standard third-point loading tests, and the modulus of elasticity of the deck concrete was calculated to be 4,230 lb/in.^{2} (29 1500 MPa) on the basis of the formula of the American Concrete Institute (ACI) formula (*11*). Hence, the corresponding cracking strain of the deck concrete is estimated to be 140×10^{-6}. Based on the plane section assumption, the strains at the top and bottom surfaces of the deck can be determined with the strain measured at a gauge point. Since the distance from an embedded gauge to the top or bottom of the deck is about 1 to 2 in. (25 to 51 mm) and the thickness of the deck is 8 in. (203 mm), it is expected that the strains at the top and bottom surfaces of the deck will reach the cracking strain when the strain at an embedded gauge near the surface is about 70×10^{-6} to 105×10^{-6}.

It can be seen from Table 1 that when the test truck was close to the abutment the maximum transverse tensile strains at the top gauge positions of the deck along Gauge Line 1 were less than 20×10^{-6} and those at the bottom gauge positions of the deck were about 60×10^{-6} to 80×10^{-6}. When the test truck was near the midspan the transverse tensile strains at the bottom gauge positions of the deck along Gauge Line 2 became very large and were about 110×10^{-6} to 180×10^{-6}. At the same time the transverse tensile strains at the top gauge positions of the deck were less than 15×10^{-6}. Comparison of these readings with the readings obtained at Gauge Line 1 indicates that the deflection of the girders increases the transverse tensile stresses at the bottom of the deck and reduces those at the top. When the test truck was close to the pier, the transverse tensile strains at the top gauge positions of the deck along Gauge Line 3 were less than 20×10^{-6}, and those at the bottom gauge positions of the deck were about 50×10^{-6} to 80×10^{-6}.

It can be seen from Table 2 that the longitudinal tensile strains developed in the deck under the test truck were less than 28×10^{-6} for all three load groups. It is also noted from the test results that the behaviors of the experimental and control decks are similar.

Numerical Results

Finite-Element Models

To investigate the stress state of a bridge deck, the most refined approach is to use the finite-element method. A

TABLE 2 Maximum Strain Readings in the Longitudinal Direction at Top and Bottom of Slab

Gage Point	Gage Line 1	2	3	4	5
A	-/+0.4	-61.8/+1.0	-10.9/+10.7	-/-30.2	-/-
C	+6.2/-	-41.9/-	-/-	-/-	+10.0/-
E	-24.3/+27.5	-35.7/-23.4	-/-	-/-21.9	-/-
F	-17.3 /-	-51.7/-	-/-	-/-	-/-

number of different finite-element models have been used for the analysis of bridge decks (*10,12*). In the present study two layers of eight-node solid elements are used to model the concrete slab, and rigid beams are used to connect the nodes at the bottom of the slab to the centroids of the girders, which are represented by three-dimensional beam elements. The cross-sectional area and moment of inertia of each girder of the bridge are 630 in.^{2} (0.41 m^2) and 242,590 in.^{4} (10 097 360 cm^4), respectively. Finite-element program SAP90 (*13*) is used for the stress analysis. Nonconforming solid elements are used to eliminate possible shear locking.

Furthermore, since only a single end span of the four-span bridge is considered, the remaining three spans are modeled by equivalent beam elements only. Each equivalent beam has a rectangular section of 54 in. (1.37 m) height and 43.45 in. (1.1 m) in width whose moment of inertia is equal to that of a fully coupled composite T-beam section consisting of a girder and a slab. The effective width of the flange is equal to the center-to-center distance between the girders, in accordance with the recommendations of ACI (*11*).

In the finite-element model 50 solid elements are used in the transverse direction of the bridge deck, with 8 solid elements used between two girders. The mesh along the transverse direction remains the same for all three load groups. The mesh along the longitudinal direction is adjusted in accordance with the locations of the axle loads of the test truck. Twenty-four solid elements are used along the longitudinal direction in a single span. For all three load groups, a fine mesh is used in the vicinity of the rear tandem axle loads, where the maximum stresses are expected to occur. The meshes used for the stress analysis are shown in Figure 7.

In the finite-element analysis of the bridge deck, the elastic moduli for the deck and girder concrete are taken to be 4,230 ksi (29 150 MPa) and 5,260 ksi (36 240 MPa), respectively. The Poisson's ratio is assumed to be 0.2 for both the deck and girder concrete. There is a steel diaphragm consisting of a C15 × 33.9 channel at the middle of each span, which is modeled by bar elements connected to the girders. The elastic modulus of the bars is assumed to be 29,000 ksi (199 810 MPa). The bridge deck has an 8-degree angle of skew. However, because the angle of skew is small, it is ignored in the stress analysis. The wheel loads of the test truck are treated as concentrated point loads.

Comparison of Test and Numerical Results

The behavior of the bridge deck under the 19 load cases was analyzed with the finite-element models presented earlier. The corresponding normal stresses along the

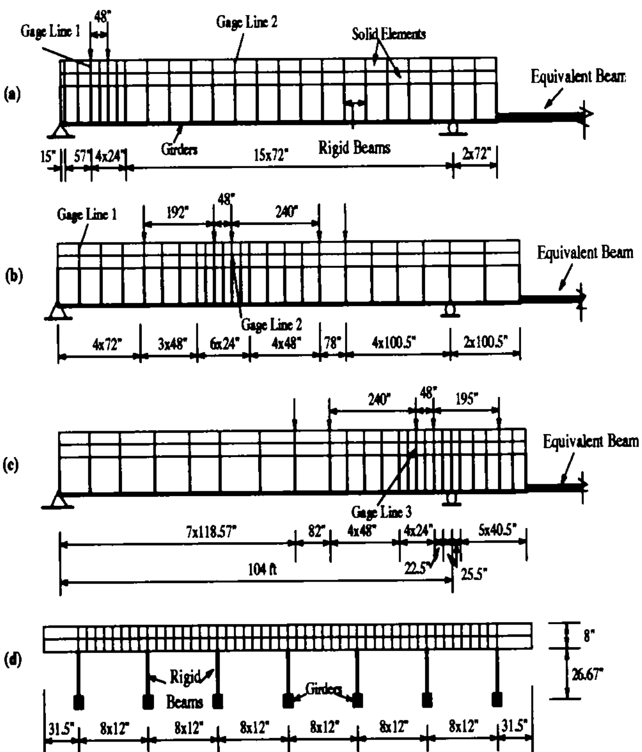


FIGURE 7 Finite-element meshes: (*a*) Longitudinal section for Load Group 1; (*b*) longitudinal section for Load Group 2; (*c*) longitudinal section for Load Group 3; (*d*) transverse section for all three load groups (1 in. = 25.4 mm).

transverse and longitudinal directions of the bridge deck have been determined.

Since two layers of eight-node solid elements have been used to model the bridge deck, the stresses have been computed at three nodal points along the depth of the slab. The stresses at the gauge locations have been evaluated from the nodal stresses with a linear interpolation, which happens to fit the nodal stresses very well. In spite of the small variations in gauge positions, it has been assumed that all strain gauges are 1.0 in. (25 mm) away from the top or bottom of the deck. Since the normal strains in both the longitudinal and transverse directions were measured at most of the gauge positions, the normal stresses in the deck have been calculated with a biaxial stress-strain relation, in which the modulus of elasticity and the Poisson's ratio of the deck concrete are the same as those used in the finite-element model.

The test and numerical results from selected load cases are compared in Figures 8 and 9. These correspond to Case A of Load Group 1 (Case 1A) and Case B1 of Load Group 2 (Case 2B1). The wheel load positions along the transverse direction of the deck are similar for these two cases, as shown in Figure 5. These

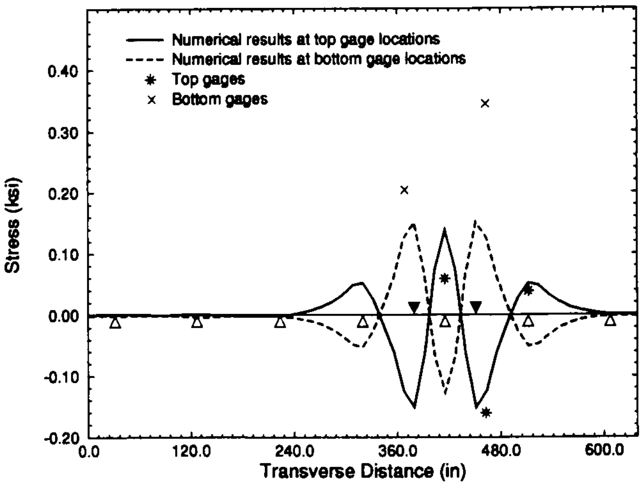


FIGURE 8 Normal stresses in transverse direction along Gauge Line 1 for test truck near abutment (Case 1A) (1 in. = 25.4 mm, 1 ksi = 6.89 MPa).

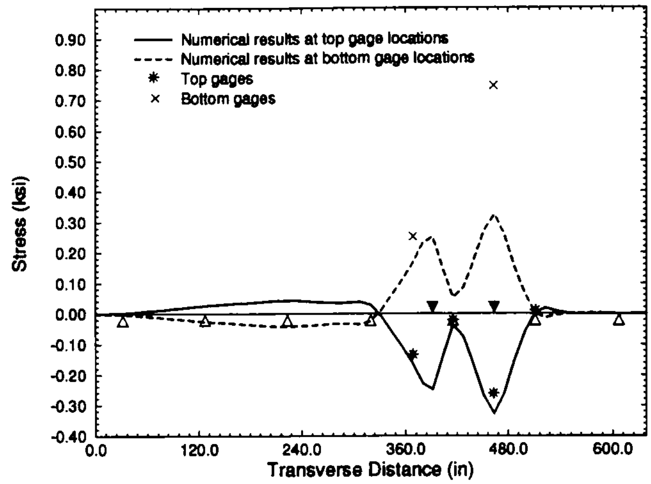


FIGURE 9 Normal stresses in transverse direction along Gauge Line 2 for test truck near midspan (Case 2B1) (1 in. = 25.4 mm, 1 ksi = 6.89 MPa).

two load cases demonstrate the effect of girder deflection on the normal stresses in the transverse direction of the deck. It can be seen from the figures that the numerical results are quite close to the test results for these two load cases. Nevertheless, the tensile stresses developed at the bottom of the deck in the field tests are about twice as large as the numerical results. This can be attributed to the cracking at the bottom of the deck, which is not accounted for in the analysis.

It can be seen from Figure 8 that when each of the wheel loads was near the midspan between two girders, the transverse normal stresses obtained from the tests at the top of the girders are only about 50 percent of the numerical results. This discrepancy is also found in other load cases in which the truck was close to a pier. This is probably caused by the flanges of the girders, which are not considered in the finite-element model. This effect is not significant when the truck loads are near the midspan, since in this case the transverse normal stresses at the top of the girders are significantly influenced by the deflection of the girders. In addition, it is found that the test and numerical results for truck loads near the pier are similar to those for truck loads close to the abutment.

It can be seen from Figure 9 that when the test truck was near the middle of the experimental deck, the transverse tensile stresses at the bottom of the deck were relatively high. This phenomenon can be observed from both the numerical and test results.

Analysis with Two Trucks

The prototype bridge has one lane of traffic in each direction. To simulate the most severe loading conditions that can be expected for the bridge, the response of the bridge deck under two test trucks was investigated. This response can be obtained with the superposition of the numerical results obtained with one test truck.

Figure 10 shows the combined effects of different load cases with the truck loads close to the abutment. The resulting maximum transverse tensile stress at the top of the deck is 230 lb/in.^{2} (1.59 MPa). Figure 11 shows the combined effects when the truck loads were near the midspan. In this case the maximum transverse tensile stress at the top is 100 lb/in.^{2} (0.69 MPa). The deck section at which the maximum tensile stress occurs is referred to as the critical section in the figures.

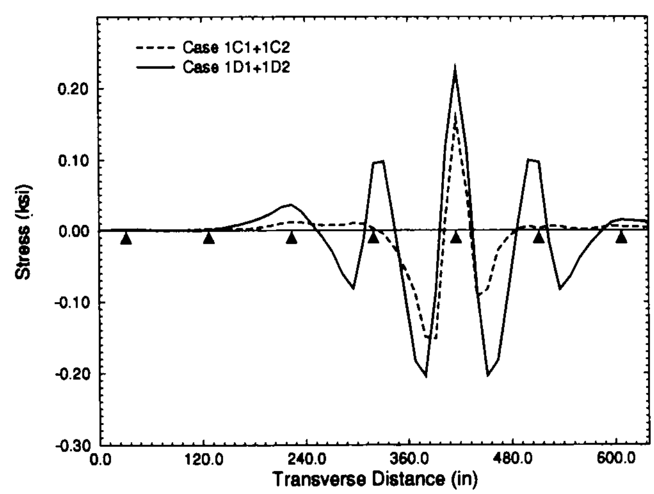


FIGURE 10 Top normal stresses in transverse direction along the critical section for two trucks near abutment (1 in. = 25.4 mm, 1 ksi = 6.89 MPa).

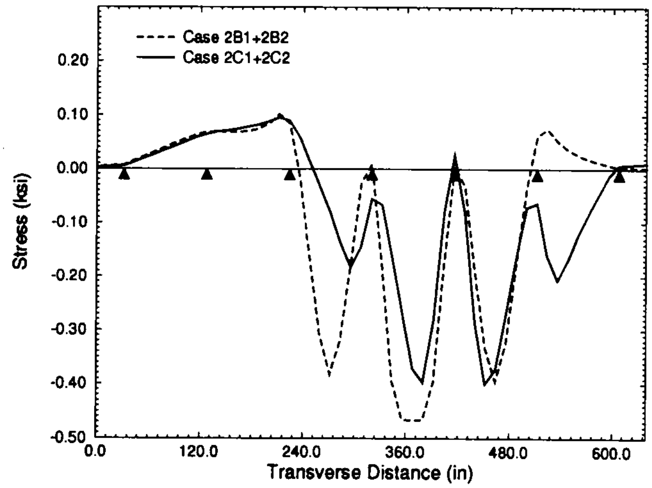


FIGURE 11 Top normal stresses in transverse direction along the critical section for two trucks near midspan (1 in. = 25.4 mm, 1 ksi = 6.89 MPa).

In summary, it has been found from the finite-element analysis that the maximum transverse tensile stress that can be developed at the top of the bridge deck is 230 lb/in.^{2} (1.59 MPa). If an impact factor of 0.3 is considered, this maximum transverse tensile stress is 300 lb/in. (2.06 MPa), which is much less than the modulus of rupture of the deck concrete.

A highway bridge is normally subjected to about 100,000 to 10 million cycles of repeated loadings during its lifetime (*14*). It has been observed from test results that the fatigue strength of plain concrete is about 60 percent of its rupture strength when concrete specimens were subjected to 10 million load cycles (*15–17*). If the bridge deck studied is going to be subjected to about 10 million load cycles, the tensile strength of the deck concrete is expected to be reduced from 590 lb/in.^{2} (4.1 MPa) to 355 lb/in. (2.45 MPa), which is still higher than the maximum tensile stresses expected at the top of the deck. Since the truck load used in the stress analysis of the deck was 47 percent heavier than a standard HS20 truck, the analysis is considered conservative.

Conclusions

The test results show that for all the load cases considered here, the transverse tensile strains at the top of the deck were always substantially lower than the cracking strain of the deck concrete. The behavior of the bridge deck under a test truck was analyzed by the finite-element method. The numerical results showed a good correlation with the test results. It was found from the analysis that even under more severe load cases with two test trucks, the normal tensile stresses at the top of the deck are less than the expected fatigue strength of concrete. Hence, the results of the present study strongly support the fact that top transverse reinforcement is not necessary for sustaining the negative bending moment induced by traffic loads. Nevertheless, further research on the control of temperature and shrinkage cracks in the absence of top reinforcement is warranted.

Nonlinear stress analysis of the bridge deck, considering the cracking of concrete, is under way. These analyses will provide a better understanding of the behavior of concrete bridge decks subjected to extreme traffic loads, as well as of the influence of shrinkage and temperature cracks on deck stresses. Furthermore, the bridge considered in the present case study has relatively stiff girders. Hence, further parametric studies should be conducted with the finite-element model to develop rational design guidelines that cover a range of girder flexibilities.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the CDOT for the present study.

REFERENCES

1. *The Status of the Nation's Highways and Bridges: Conditions and Performance—Highway Bridge Replacement and Rehabilitation Program.* FHWA, U.S. Department of Transportation, 1989.
2. Li Cao, J. H. Allen, and P. B. Shing. A Case Study of Elastic Concrete Deck Behavior in a Four-Span Prestressed Girder Bridge: Finite Element Analysis. *Report CDOT-DTD-CU-93-7,* Colorado Department of Transportation, Denver, Jan. 1993.
3. Li Cao, J. H. Allen, P. B. Shing, and D. Woodham. A Case Study of Concrete Deck Behavior in a Four-Span Prestressed Girder Bridge: Correlation of Field Tests and Numerical Results. *Report CDOT-CU-R-94-8,* Colorado Department of Transportation, Denver, April 1994.
4. Westergaard, H. M. Computations of Stresses in Bridge Slabs Due to Wheel Loads. *Public Roads,* March, 1930, pp. 1–23.
5. *Standard Specifications for Highway Bridges,* 15th ed. AASHTO, Washington, D.C., 1992.
6. *Standard Specifications for Highway Bridges,* 2nd ed. AASHTO, Washington, D.C., 1935.
7. *Minutes of AASHTO Bridge Subcommittee.* FHWA, AASHTO, Washington, D.C., 1957–1961.
8. Allen, J. H. Cracking, Serviceability and Strength of Concrete Bridge Decks. In *Transportation Research Record 1290,* TRB, National Research Council, Washington, D.C., 1991, pp. 152–171.

9. Beal, D. B. Load Capacity of Concrete Bridge Decks. *Journal of the Structural Division*, ASCE, Vol. 108, No. ST4, 1982, pp. 814–882.
10. Fang, I. K., J. A. Worley, N. H. Burns, and R. E. Klinger. Behavior of Isotropic R/C Bridge Decks on Steel Girders. *Journal of Structural Engineering*, ASCE, Vol. 116, No. 5, 1990, pp. 659–678.
11. *Building Code Requirements for Reinforced Concrete* (ACI 318-89). American Concrete Institute, 1989.
12. Mufti, A. A., et al. Fiber Reinforced Concrete Deck Slabs Without Steel Reinforcements. *Report 1-1991*, Department of Civil Engineering, Technical University of Nova Scotia, July 1991.
13. Wilson, E. L., et al. *SAP90 Program User's Manual.* Computers & Structures, Inc., 1989.
14. Hsu, T. T. C. Fatigue of Plain Concrete. *ACI Journal*, July-Aug., 1981, pp. 292–305.
15. Ballinger, C. A. Cumulative Fatigue Damage Characteristics of Plain Concrete. In *Highway Research Record 370*, HRB, National Research Council, Washington, D.C., 1971, pp. 48–60.
16. Tepers, R., and T. Kutti. Fatigue Strength of Plain, Ordinary, and Lightweight Concrete. *ACI Journal*, May, 1979, pp. 635–652.
17. Oh, B. H. Fatigue Analysis of Plain Concrete in Flexure. *Journal of Structural Engineering*, ASCE, Vol. 112, No. 2, 1986, pp. 273–288.

The opinions expressed in this report are those of the authors and do not necessarily represent those of the sponsor.