Design Anomalies in Concrete Deck–Steel Stringer Bridges

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The deterioration of highway bridge decks has been recognized as the most important factor governing the serviceability of bridges. Because of limited information provided in American Association of State Highway and Transportation Officials (AASHTO) specifications for the design of highway composite bridges to account for deterioration with time, a quantitative study of bridge deterioration factors is provided here. A description of the most important factors influencing bridge deterioration is presented. A review of the mechanism of deterioration and the status of current practice for each of the deterioration factors is provided to demonstrate the need to incorporate them into the AASHTO specifications. Finally, a stress comparison between a typical design of a highway bridge deck based on AASHTO specifications and the proposed method that accounts for four deterioration factors is presented, with a list of research needs to establish a complete understanding of the effects of bridge deterioration via suitable design formulas.

The National Bridge Inventory (NBI) study found that 23.5 percent of the nation's highway bridges are structurally deficient. These deficient bridges have been restricted to light vehicles. The most common deficiency observed was deck deterioration (*1*).

Although American Association of State Highway and Transportation Officials (AASHTO) specifications (*2*) contain adequate information about the flexural or shear design of highway composite bridges subjected to dead or live loads, little information is given about the factors that need to be included in the bridge design to account for deterioration with time.

The objective of this paper is to review the factors that influence the deterioration of concrete deck–steel stringer bridges, which have not received adequate attention in the current AASHTO highway bridge design specifications, and to investigate the accuracy of current design procedures and suggest some modifications.

A brief literature review of the performance of composite bridges follows. The description of the mechanism for each deterioration is suggested to provide a better understanding of bridge system behavior. Such an assessment may help develop priorities for the incorporation of different time-dependent deterioration mechanisms in design. Although specific influences of different failure mechanisms are not computed, stress levels for some mechanisms of deterioration are provided for a typical bridge to enable the reader to appreciate the significance of these mechanisms.

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FACTORS INFLUENCING DETERIORATION OF BRIDGE SUPERSTRUCTURES

Concrete deck–steel stringer bridges began to show an increasing degree of deterioration in the early 1960s. The main categories of deterioration are scaling, cracking, spalling, and delamination (*3*). The factors that seemed to play the most important role are

1. Corrosion of reinforcing steel,
2. Surface degradation (scaling, cracking, spalling, or rutting) due to moisture absorption and freeze-thaw cycles,
3. Loss of composite action,
4. Aging of concrete,
5. Tensile-stress inducement at the deck top,
6. Unequal expansion and contraction coefficients in concrete leading to thermal creep,
7. Stress inducement because of temperature gradients along the stringer depth,
8. Stress inducement at the interface between deck and stringers because of shrinkage and creep, and
9. Out-of-plane (impact) or in-plane (acceleration/deceleration) forces.

Much work has been performed on corrosion of steel reinforcement (*4–7*). The focus of this paper is on factors other than corrosion that affect deck deterioration. Results from the analysis of a typical composite bridge (Figure 1), including the effect of the most important deteriorating factors covered here, are compared with AASHTO results.

Loss of Composite Action Between Deck and Stringers

Mechanism of Deterioration

Composite concentration has been used extensively in highway bridge design because each material (concrete, steel) is employed to its best advantage (*8*). When the concrete is cast over steel stringers, shear studs are embedded into the concrete and can be considered to be in full contact with the surrounding concrete. Microcracks develop in the concrete surrounding the shear connectors. Microcracks may be caused by shrinkage or early creep of concrete or to corrosion of studs. Because the shear stud can undergo larger deformations resulting from progressive increases in cracking, the studs begin to crush the surrounding concrete and reduce its strength. Therefore, cyclic loads lead to initial crushing of concrete and possibly to a stud failure in fatigue.

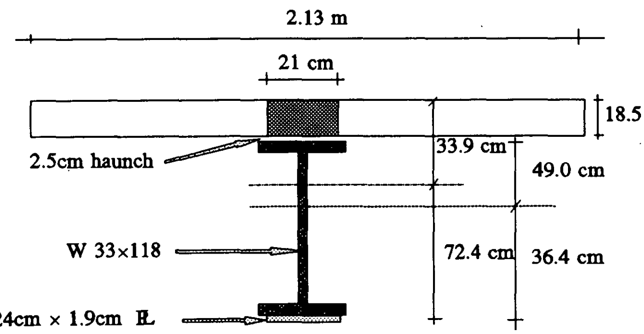

FIGURE 1 Design example of a composite concrete deck–steel stringer bridge (b_{ef} = 2.1 m, L = 17.4 m) (1 MPa = 145 psi).

The design of composite concrete and steel bridge superstructures using AASHTO specifications is based on the assumption of full composite action between deck and stringers. However, the slope of the curve of load versus slip decreases with time (*9*). The presence of slip violates the assumption of full composite action and reduces the bending stiffness. This stiffness reduction leads to additional deflection and induced stresses in the structure.

Status of Current Practice

Design of stud shear connectors in AASHTO or American Institute of Steel Construction (AISC) (*10*) specifications in composite structures is based on the principle that shear connectors have to withstand the ultimate load on the bridge (ultimate strength design) and resist the numerous applications of serviceability loads (*9*).

AASHTO specifications provide shear connector design by suggesting the check for ultimate strength (Article 10.38.5.1.2) and fatigue strength (Article 10.38.5.1.1) of the connectors. Oehlers (*9*) states that the fact that no interaction between monotonic strength of the connectors and fatigue loads is considered in the design can be explained by the absence of a design mechanism to allow the stud to experience fatigue damage. The shear connectors are also designed assuming full composite action between steel and concrete at service loads after a number of fatigue cycles. However, a considerable drop was reported in ultimate load of concrete decks with steel stringers after fatigue loading (*11,12*). The ultimate strength drop is between 51 and 73 percent of their expected static strengths. Oehlers (*9*) showed that the residual strength of the studs after a number of cycles also depends on the initial strength of the connectors. The semiempirical equations proposed by Oehlers are based on his experimental work; it is a great design tool.

Cracking or crushing of concrete around the studs and fatigue of the studs lead to a partial composite action that is not accounted for in the AASHTO specifications. Partial composite action in concrete and steel beams has been discussed by a number of authors (*8,13*) and has also been included in the AISC (*10*) design code (Article 1.11.4, 1988), but these code provisions lead to the use of fewer shear connectors in the ultimate strength design without consideration of the loss of stiffness of the structure from the loss of composite action. It has been observed that partial interaction of concrete deck and steel stringers increases deflection with higher concrete strength, lower steel strength, lower modulus of connector, lower ratio of steel area to concrete area, and low span-to-depth ratio (*14*). Knowles (*13*) introduced a simple methodology to account for partial composite action by establishing equilibrium equations of forces and moments acting on the partially composite section.

Zaremba (*15*) also investigated the partial composite action of composite members. Using a nonlinear load-slip relationship for shear connectors and equilibrium equations based on the strain distribution of a partial composite member, he developed a system of two differential equations and a computer-aided solution. He was able to evaluate the loss in stiffness at various load stages.

The fact that interfacial slip in composite structures can cause redistribution of strains and stresses under service or ultimate loads led Al-Amery and Roberts (*16*) to include the nonlinear material and shear connector behavior in the analysis of partially composite members. Their illustrative example presented the ability of the method to obtain reasonably accurate results for the load range, including failure.

All the findings described reveal that a percentage of composite action between the bridge deck and stringers decreases because of the decrease in contact of shear studs with the surrounding concrete. The stiffness degradation may not be severe for static service loads, but it can be detrimental when fatigue loading is involved. Sotiropoulos and GangaRao (*17*) have studied the effect of stiffness degradation associated with the loss of composite action for a typical composite bridge 7.3 m (24 ft) wide and 17.4 m (57 ft) long (Figure 1). Using Oehlers' (*9*) design equations, the interfacial slip between deck and stringers was calculated and the composite deck was designed including partial composite action. Results showed a 25 percent decrease in bending stiffness and 8 and 180 percent in stresses at the bottom and top of the steel stringer, respectively. Stress values for the particular design example are summarized in Table 1.

Thermal Stresses Due to Temperature Variation

Mechanism of Deterioration

Temperature rise or drop during a day or throughout a year induces, in many cases, longitudinal and transverse stresses that are often overlooked during the design of bridge superstructures. Malfunctioning expansion-contraction joints or a nonuniform temperature gradient through the deck depth lead to the development of stresses that can reach high levels. Because of concrete's poor heat conductivity and the increased depth of concrete decks, temperature differentials between upper and lower surfaces (or inner and outer walls in a box-girder section) are observed more often than in steel structures (*18*).

Most of the temperature-related problems in bridge superstructures have been observed in continuous spans (*19*). In precast, segmental concrete decks, absence of continuous reinforcement at the bottom of beams (at the intermediate pier location) may cause horizontal movement of each span because of the thermally induced curvature, leading to exertion of horizontal forces on the bearings and possible damage (*20*).

TABLE 1 Changes in Stresses Due to Effect of Deterioration

	σ_c^{top} (MPa)	σ_c^{bot} (MPa)	σ_s^{top} (MPa)	σ_s^{bot} (MPa)
AASHTO	-3.8	1.63	-20.86	116.41
Loss of Comp. Action	2.0	0.90	-37.46	9.05
Thermal stresses	-0.55	0.32	4.45	-3.72
Creep & Shrinkage	3.48	-0.09	-8.76	65.85
Transverse tensile stress	3.94	-	-	-

1 MPa = 145 psi

σ_c^{top} = stress at top of concrete deck

σ_c^{bot} = stress at bottom of concrete deck

σ_s^{top} = stress at top of steel stringer

σ_s^{bot} = stress at bottom of steel stringer

- indicates that data are not applicable

Radolli and Green (*18*) stated that temperature differences of 40°F (20°C) can give rise to thermal stresses equal in magnitude to maximum live-load stresses at the bottom fiber of the midspan section and up to three times larger than the live-load stresses at the top fiber. The combination of thermal stresses with dead- or live-load stresses or fatigue can create additional cracking in the concrete deck. It must be noted that cracking causes stress relief, and the reduction of the moment of inertia can lead to a reduction of thermal moments (*21*).

Status of Current Practice

As mentioned before, AASHTO specifications (*2*) do not consider temperature variations through girder depth. They only specify a rise or drop of mean temperature of the bridge superstructure in moderate or cold climates. A nonlinear temperature gradient has, however, been proposed in the draft copy of the load resistance factor design (LRFD) specifications by AASHTO (*22*); these design specifications are pending acceptance by the designers. The particular gradient can be used for concrete or steel superstructures, but no provision was made for composite bridges.

Evaluation of temperature distribution and thermally induced stresses has been a topic of interest for a number of years. A number of different temperature profiles along the deck depth have been proposed. Linear and nonlinear temperature gradients for a composite bridge have been proposed and are presented in Figure 1. The bilinear temperature distribution of the American Society of Civil Engineers (ASCE) (*23*) is selected to illustrate the example (*17*).

A deck subjected to nonlinear temperature distribution through depth experiences three effects: (a) uniform expansion or contraction (covered by AASHTO), (b) a curvature (hogging) that does not create any stresses in a simply supported case, and (c) Eigen stresses that are developed to compensate for the Navier-Bernoulli assumption of plane sections remaining plane, that is, linear strain distribution through depth. For continuous spans, additional stresses are accounted for in the calculations to compensate for the continuity of the span (continuity stresses). A formulation of one-dimensional heat transfer was used in most of the theoretical investigations to calculate thermal stresses for given temperature variation. The most popular and well-documented methods available in the literature are reported by Priestley (*24*) and Zuk (*25,26*). Both methods are simple static or equilibrium equations, and the former is used mainly for all concrete decks, whereas the latter is used in composite structures. Soliman and Kennedy (*27*) simplified Zuk's method and applied it in an illustrative example for a composite steel and concrete beam. Following the same procedure, but assuming ASCE's temperature distribution (Figure 2), the thermal stresses for the particular design example of the 7.3-m × 17.4-m (24-ft × 57-ft) composite bridge were presented by Sotiropoulos and GangaRao (*17*). The illustrative example

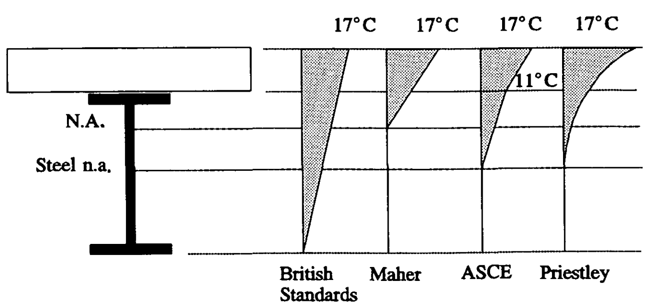

FIGURE 2 Proposed temperature gradients.

revealed that thermal stresses developed in the particular section under ASCE's proposed temperature distribution reached the level of 60 percent of the live-load stresses at the top of the steel beam in the example.

Calculations of thermal stresses developed under a nonlinear temperature gradient through the depth are well documented in the literature, and good correlation with experimental results verified their accuracy (*21*). Assuming a temperature gradient proposed by ASCE (*23*), the developed thermal stresses along a deck depth are calculated for simply supported or continuous spans and all concrete or composite decks. Temperature variation across the deck width is not considered in any of the methods mentioned; this variation can have significant effects on box-girder bridges.

Thermal Creep Due to Difference in Thermal Expansion and Contraction Coefficients of Concrete and Steel

Mechanism of Deterioration

Concrete, unlike steel, experiences unequal expansion and contraction when subjected to a temperature rise or drop of the same amount (ΔT). The phenomenon of unequal thermal expansion and contraction coefficients has been observed by a number of researchers, but in most cases no provisions are made during the design of concrete decks. Incompatibility of thermal coefficients between steel and concrete plays a major role in the performance of a deck, including deck growth problems.

The unequal thermal expansion and contraction coefficients in concrete have been recognized as contributing factors in deck deterioration (*28*). As has been reported by a number of researchers (*29–31*), thermal coefficients of concrete are different at different temperature ranges (above and below the freezing point of water) and even at the same temperature ranges (*30*). Variations in expansion and contraction coefficients of concrete and thermal coefficients between concrete and steel lead to inducement of residual thermal stresses and strain incompatibility. These factors in turn lead to deck cracking and "deck growth."

Explanation of the expansion and contraction incompatibility of concrete is based on the composite nature of the material. The thermal coefficient of concrete depends on the quantity of the aggregate in the mix and the coefficient of the aggregate itself (*32*). Decrease of moisture and increase in age decrease the thermal coefficient (*29*). Thermal expansion or contraction is the result of the normal expansion and contraction of anhydrous materials and the hygrothermal expansion and contraction associated with the movement of internal moisture from capillaries or from gel pores (*31*). It has also been suggested (*32*) that if the thermal coefficients of aggregate and cement differ greatly, a large temperature change may introduce differential movement and a break in the bond between the aggregate and the surrounding paste. Therefore, if this differential movement occurs, cooling of concrete will not lead to recovery of the initial dimensions. The result will be a mismatch in expansion and contraction coefficients causing development of residual stresses.

Status of Current Practice

Although the mismatch of expansion and contraction coefficients of concrete has been observed, little has been done to correlate this effect with concrete deterioration. Limited experimental work has been conducted on specimens from actual bridge decks by GangaRao et al. (*28*), who studied, among other parameters, the magnitude of thermal creep and residual stress buildup in several concrete and concrete and steel specimens cast in laboratory conditions or cut from actual bridge decks under temperature and freeze-thaw cycles. Results verified the difference between thermal expansion and contraction coefficients that also varied in value at different temperature ranges (the contraction coefficient is always smaller than the expansion coefficient). The difference in expansion and contraction coefficients increased as the temperature range increased. Residual tensile strains were observed in both concrete and steel bars. The fact that their values differed in the same direction verified strain incompatibility between steel and concrete. The thermal strain buildup after 40 to 50 thermal cycles and after subtracting the "shrinkage compensation" was in the range of 400 microstrains (400×10^{-6} in./in.). It has also to be noted that plain concrete behaved differently from reinforced concrete. A good understanding of the behavior of concrete under freeze-thaw cycles awaits additional experimental data.

Shrinkage and Creep Effects in Steel and Concrete Composite Decks

Mechanism of Deterioration

As concrete is integrally connected with steel in composite structures, creep and shrinkage stresses have to be transferred from concrete to steel. The development of these stresses in concrete, resulting from the restraint set by the steel beam, reduces the efficiency of concrete in resisting loads (*33*). If the shrinkage stresses after reduction of the creep effect are greater than the allowable tensile stresses, cracks form in a concrete deck. As creep relief decreases with age, the tendency to crack becomes greater (*32*). The shrinkage and creep of concrete and steel structures result in an increase of deflection (*34*). Stresses induced by differential shrinkage may be as high as 50 percent of the dead load stresses in shorter bridges (*35*).

The magnitude of the shrinkage and creep stresses in a composite structure depends on the parameters that directly or indirectly affect the shrinkage and creep of the concrete itself. The typical parameters affecting shrinkage and creep are drying rate, size and grading of aggregate, water-to-cement ratio, relative humidity, externally applied stress level, strength of concrete, and so forth (*32*).

Status of Current Practice

Ghali and Favré (*36*) have researched the creep and shrinkage (time-dependent parameters) problems in all concrete structures. Their work provides theoretical calculations for shrinkage and creep stresses and deflections for composite steel and concrete structures. Bradford (*34*) has also presented a simplified method to calculate deflections resulting from creep and shrinkage of composite structures. The usual method of calculating the shrinkage and creep forces is the so-called "composite section technique," in which the differential shrinkage and creep forces are resisted by the composite section (*35*). An illustrative example and brief explanation of these proposed methods have been presented by Sotiropoulos and GangaRao (*17*).

AASHTO specifications do not neglect shrinkage and creep effects in concrete decks. The information given in the specifications is rather conservative for creep and not very explanatory for shrinkage. In particular, AASHTO specifications (Article 9.13.3.3, 1989) state that differential shrinkage of cast-in-place concrete over precast beams may influence the cracking load and the beam deflection profile. When these factors are particularly significant, the effect of differential shrinkage should be added to the effect of loads. In Article 10.38.1.4 (*2*), a multiplier of 3 is specified on calculated stresses of composite structures to account for creep. It has been noted that creep and shrinkage cannot be fully separated, because they take place simultaneously (*35*).

Creep and shrinkage stresses are large. The magnitude of tensile stresses developed at the top of the concrete deck for the design example mentioned before was in the area of 3.45 MPa (500 psi) (*17*). Combining this stress with the instantaneous stress resulting from dead load may lead to excessive tensile stresses and possibly to cracking. AASHTO's approach may not provide good results, especially for composite structures, because the differential shrinkage effect was not included in that calculation.

The methodology of the "composite section" can be considered adequate to address the differential creep and shrinkage between different concrete members in the same structure or composite concrete and steel decks. Cracked concrete sections as well as continuous spans can be studied by adding appropriate continuity stresses (*36*). All methodologies use the assumption of full composite action (no interfacial slip) between concrete deck and steel stringers, despite the tendency of the deck to lose some composite action with increasing number of load cycles. Therefore, the method of Ghali and Favré needs to be modified to include partial composite action. For simplification, incorporation of partial composite action in the calculations of shrinkage and creep stresses can be accomplished by using an "effective deck thickness," that is, the thickness of the deck that will result in the same effective bending stiffness of the composite beam after reduction resulting from partial composite action (*37*). The additional in-plane shear forces transmitted to the steel stringer through the connectors caused by shrinkage and creep, or accelerating or braking forces, need to be accounted for in the design of the connectors.

In-Plane and Out-of-Plane Forces on Decks

Mechanism of Deterioration

The in-plane forces from accelerating or braking vehicles can lead to deterioration of the deck surface (concrete spalling, rutting). Local, friction-type loading on decks leads to an increase in stress on top of the deck. Such stress intensity may lead to breaking of the bond between the concrete's constituents or between the concrete and its reinforcement, causing spalling and delamination. Braking or accelerating forces also affect the composite action of composite structures because the in-plane forces are transmitted from the concrete deck to the steel stringers through the shear connectors. Existence of such loads can lead to additional local shear loading of the studs and subsequent loss of composite action with steel in the same manner as described earlier. Braking or accelerating forces cause redistribution of axle loads and can lead to an increase in impact force that may exceed the factors adopted by design codes (*38*).

Out-of-plane impact loads from the pounding of passing vehicles because of the rough riding surface of the concrete deck or even "launching" of trucks onto the deck because of unleveled approaches often cause deterioration of concrete decks. In addition to the induced vibrations in a local and global sense, high-intensity repetitive loading forms a punching type of load that can be detrimental close to the stringer locations. This out-of-plane fatigue load leads to deterioration of the integrity of concrete and causes spalling and rutting. Also, depending upon the bridge and vehicle characteristics, it may lead to excessive vibrations of the bridge as a whole and excessive stresses and deformations.

Status of Current Practice

Limited experimental or theoretical work has been published on the effect of local in-plane loads on bridge decks. The problem of impact or longitudinal forces on bridge decks has been dealt with in design codes or by individual researchers as a problem of vibration of the bridge superstructure by specifying equivalent static loads (*2*) or proposing rigorous analyses to predict the actual behavior of a bridge under these loading conditions.

AASHTO specifications define equivalent static loads to represent the longitudinal and impact forces exerted by the vehicular traffic. The design value for the force resulting from braking is taken as 5 percent of the live load in all lanes carrying traffic headed in the same direction (Article 3.9). The center of gravity of this force is assumed to be located 6 ft above the floor slab. Impact forces are considered through an impact allowance factor given by Article 3.8.2.

According to the Committee on Loads and Forces on Bridges (*23*), AASHTO specifications (*2*) requirements for minimum longitudinal forces are far less than the longitudinal load required by many other codes.

Modeling of braking forces and the consequent response on bridge decks was attempted by Gupta and Traill-Nash (*38*). They showed that the impact factor was 0.25 for a symmetric loading case and 0.33 for an eccentric case when AASHTO specified 0.27 for the particular span length. Differences were also reported by O'Connor and Chan (*39*) between the AASHTO specified impact factor and measured values from tests on composite steel and concrete bridges. O'Connor and Chan presented a method to evaluate the impact factor based on deflection or even strain readings. GangaRao (*40*) also presented a deterministic procedure, based on the orthotropic plate theory, to compute frequencies, deformations, and stresses. Analytical results for impact factor correlated well with measured data from a number of highway bridges in West Virginia. Harsh and Darwin (*41*) stated that traffic-induced vibrations did not appear to be detrimental to bond strength and compressive strength in bridge deck repairs if the concrete had low slump. The majority of recent studies have contradicted the current AASHTO impact factor (*40*). Maximum values of impact factor in the range of 0.80 were observed in a study of continuous bridge superstructures by Csagoly et al. (*42*).

As Schilling (*43*) noted, the impact factors used in fatigue design of bridges are different from those for nonfatigue design because they have to be based on average and not extreme loading conditions and have to account for the dynamic effects on the stress range rather than the peak stress. Analysis of the response of model bridges under four- and five-axle trucks presented an impact factor of 0.25. According to Schilling, the AASHTO formula can be used for fatigue calculations for simple-span and continuous-span bridges.

All the theories presented and research have revealed a number of differences between impact factors and longitudinal forces and AASHTO specifications. The differences are expected because of the dynamic nature of impact and braking forces. It is impossible for a design code to include all parameters that can affect the dynamic response of the deck.

The comparatively low design value for longitudinal force resulting from braking can be verified by a series of wheel-load tests on bridge decks. So far, wheel-load testing has been used for fatigue testing of decks. Such a test will be able to simulate the impact (pounding) nature of traffic loads and the horizontal braking or accelerating forces in the plane of the deck. Such a test can also be used to monitor the durability and wear resistance of the concrete deck surface against rutting, delamination, and spalling.

Tensile Stress Inducement on the Deck Top

Mechanism of Deterioration

Depending on the location of the concentrated loads that a typical vehicle exerts on the deck and relative transverse deck stiffness and longitudinal beam stiffness, eccentric loads on bridge decks may induce net tensile stresses on the concrete top. If the reinforcement is not adequate to withstand the induced tensile stresses, the concrete will crack. This phenomenon can be repeated and in time lead to fatigue failure of the deck.

Status of Current Practice

Current AASHTO specifications (*2*) lead to the bridge deck design for maximum moment when the design truck has one of its two 142-kN (32-kip) axles over the transverse center line of the bridge (HS20-44 design truck). Similar shear design of the deck uses the location of the truck as close to the extreme stringer as possible. No reference is made in the specifications to the possibility of stress reversals across the width of a deck under eccentric loads.

Using a simplified methodology of a beam on elastic springs developed by Kallomalos (*44*), an approximation can be made of load distribution factors across the deck width. Establishing the force interactions between deck and stringers, a moment diagram of the deck can be obtained resulting from the load by one axle of the design AASHTO truck and the area of the deck in which stress reversals exist can be located. Theoretical results on a timber deck 7.3 m × 12.2 m (24 ft × 40 ft) with a thickness of 23.5 cm (9.25 in.) presented a tensile stress at the top of the timber deck of the order of 3.45 MPa (500 psi) for an eccentric position of the design truck. The beam-on-elastic-springs model was analyzed by finite elements (*17*) and by representing the concrete deck of the design example (Figure 1) by a strip 1.52 m (60 in.) wide and the stringers by elastic springs. The resulting maximum tensile stress at the top of the deck for an eccentric position of the AASHTO truck load was 3.94 MPa (571 psi). The magnitude of the tensile forces in this design example may not seem adequate to cause concrete failure, but these tensile forces in conjunction with fatigue cycling can lead to fatigue failure of the deck.

Finite element analysis, including composite and noncomposite action, of concrete bridge decks on steel stringers (*45*) determined that for a bridge 8.5 m × 18.3 m (28 ft × 60 ft) with an 18-cm (7-in.) concrete slab, tensile stresses in the range of 1.4 MPa (200 psi) developed at the top of the deck away from the truck position. The designed deck has almost the same dimensions as the one that is adopted in the example (*17*). Noncomposite action between stringers and deck presented a more dispersed distribution of tensile stresses in the center half of the span, whereas when full composite action was assumed, tensile stresses developed along lines coincident to the unloaded stringer positions.

Eccentric fatigue tests of concrete slabs on steel girders need to be performed to monitor the induced tensile stresses and the possible crack formation at the concrete top as well. It is necessary to examine whether the conventional orthotropic reinforcement ratios recommended by AASHTO for deck slabs can provide adequate strength because of this particular extreme loading condition. Batchelor (*46*) found that deck slabs with such reinforcement presented large reserves of strength against fatigue failure. Csagoly and Lybas (*47*) also stated that 0.3 percent isotropic reinforcement is adequate for serviceability fatigue and ultimate capacity. Before any of these recommendations are adopted, additional tests should be performed on decks to determine their service life.

RESEARCH NEEDS

As stated in the preceding section, deficiencies exist in the current AASHTO specifications covering the design of highway bridge decks to withstand or tolerate deterioration. Simple numerical calculations presented the effects of various factors on the performance of the deck and the entire bridge superstructure (*17*).

As the evidence of the growing need to improve deck life becomes paramount, a rank ordering of research needs in this area of bridge design is important; such a listing is suggested here.

1. Loss of composite action constitutes the most urgent aspect that needs to be considered in design computations because it directly affects the stiffness of composite structures and no provisions are made in the specifications about the loss of composite action with time. Considerable progress has been made in predicting the loss of composite action, but additional research is needed to unify all the proposed methods to predict the decrease of bending stiffness with time.

2. Thermal effects on bridge superstructures can be significant under extreme temperature conditions, but more emphasis needs to be placed on understanding the variations of thermal coefficients of concrete leading to thermal creep. As far as thermal stress evaluations due to nonlinear temperature gradients through the depth of the deck are concerned, transportation officials need to include this aspect as an important item for the new AASHTO specifications because design methodologies have a way of gaining general acceptance.

3. Creep and shrinkage effects on composite decks urgently need attention because of the magnitude of the developed stresses. Knowledge and experience gained from the study of creep and shrinkage of plain reinforced concrete are expanded to understand and calculate the resulting stresses on concrete decks and steel beams of composite decks. The significance of this factor is based on the fact that even though creep and shrinkage originate in the concrete deck, their action affects both the deck and stringers.

4. The AASHTO specifications for transverse reinforcement of concrete decks should be revised on the basis of more accurate transverse load distribution formulas. Inducement of tensile stresses at the concrete top (under eccentric loads) should be studied from the point of view of serviceability because tensile stresses may cause cracking or fatigue to the deck.

5. The effects of out-of-plane and in-plane loads on concrete decks should be studied as the main factors of material disintegration in a local and global sense.

A summary of the results of the preliminary analysis (*17*) of a typical 7.3-m × 17.4-m (24-ft × 57-ft) concrete and steel bridge with a deck of 18.5 cm (7.25 in.) is presented (Table 1) to illustrate the need to modify current AASHTO specifications.

CONCLUSIONS

This study presents the major factors affecting bridge deck deterioration. Special emphasis is placed on items not fully covered in the current AASHTO design specifications. A description of the deteriorating mechanisms of each factor is given, with a list of advancements in theory or experiment toward a better understanding of each phenomenon, as reported in the literature. Specific additional research is recommended. This work presents areas in the design of highway bridges that need to be revised by AASHTO. The results from the design example presented are preliminary. Similar calculations for superstructures with different dimensions need to be studied. The paper emphasizes the need to account for the deterioration of bridge superstructures during design.

ACKNOWLEDGMENT

The authors would like to acknowledge the help of Barry Dickson of the Constructed Facilities Center of West Virginia University during the review of this paper.

REFERENCES

1. K. F. Dunker and B. G. Rabbat. Highway Bridge Type and Performance Patterns. *Journal of Performance of Constructed Facilities*, ASCE, Vol. 4, No. 3, 1990, pp. 161–173.
2. *Standard Specifications for Highway Bridges*, 14th ed. AASHTO, Washington, D.C., 1989.
3. R. K. Banks. Bridge Decks: Their Problems and Solutions. *Public Works*, Vol. 117, No. 12, 1986, pp. 26–28.
4. J. T. N. Atkinson and H. VanDroffelaar. *Corrosion and Its Control: An Introduction to the Subject.* National Association of Corrosion Engineers, Houston, Tex., 1982.
5. C. H. Hare et al. *NCHRP, Synthesis of Highway Practice 136: Protective Coatings for Bridge Steel.* TRB, National Research Council, Washington, D.C., Dec. 1987.
6. G. J. Malasheskie et al. *Bridge Deck Protective Systems.* Final Report RP85-17. Pennsylvania Department of Transportation, Harrisburg, July 1988.
7. D. Stark. *Evaluation of the Performance of Galvanized Reinforcement in Concrete Bridge Decks.* Final Report. Portland Cement Association, Skokie, Ill., May 1982.
8. J. P. Cook. *Composite Construction Methods.* John Wiley & Sons, New York, 1977.
9. D. J. Oehlers. Deterioration in Strength of Stud Connectors in Composite Bridge Beams. *Journal of Structural Engineering*, ASCE, Vol. 116, No. 12, 1990, pp. 641–653.
10. *Manual of Steel Construction*, 8th ed. American Institute of Steel Construction, Inc., Chicago, Ill., 1988.
11. C. G. Coughlan. *The Stiffness of Stud Shear Connections in Composite Beams Subjected to Static and Fatigue Loading.* M.S.E. thesis. National University of Ireland at Cork, 1984.
12. D. J. Oehlers and L. Foley. The Fatigue Strength of Stud Shear Connections in Composite Beams. *Proceedings of the Institute of Civil Engineers*, Vol. 79, No. 2, 1985, pp. 349–364.
13. P. R. Knowles. *Composite Steel and Concrete Construction.* John Wiley & Sons, London, 1973.
14. R. P. Johnson and I. M. May. Partial-Interaction Design of Composite Beams. *Structural Engineer*, Vol. 53, No. 8, Aug. 1975, pp. 305–311.
15. C. J. Zaremba. Strength of Steel Frames Using Partial Composite Girders. *Journal of Structural Engineering*, ASCE, Vol. 114, No. 8, 1988, pp. 1741–1760.
16. R. I. M. Al-Amery and T. M. Roberts. Nonlinear Finite Difference Analysis of Composite Beams with Partial Interaction. *Computers & Structures*, Vol. 35, No. 1, 1990, pp. 81–87.
17. S. N. Sotiropoulos and H. V. S. GangaRao. *Design Anomalies in Highway Bridge Decks.* Interim Report. Constructed Facilities Center, West Virginia University, Morgantown, 1992.
18. M. Radolli and R. Green. Thermal Stresses in Concrete Bridge Superstructures Under Summer Conditions. In *Transportation Research Record 547*, TRB, National Research Council, Washington, D.C., 1975, p. 23.
19. R. A. Imbsen, D. E. Vandershaf, R. A. Schamber, and R. V. Nutt. Thermal Effects in Concrete Bridge Superstructures. In *NCHRP* Report 276, TRB, National Research Council, Washington, D.C., 1985, p. 99.
20. M. Elbadry and A. Ghali. Thermal Stresses and Cracking of Concrete Bridges. *ACI Journal*, No. 83-90, Nov./Dec. 1986, pp. 1001–1009.
21. S. J. Thurston, M. J. N. Priestley, and N. Cooke. Influence of Cracking on Thermal Response of Reinforced Concrete Bridges. *Concrete International*, Vol. 6, No. 8, 1984, pp. 36–43.
22. Modjeski and Masters. *Development of Comprehensive Bridge Specifications and Commentary.* 3rd Draft LRFD Specifications and Commentary. Prepared for National Cooperative Highway

Research Program, TRB, National Research Council, Washington, D.C., April 1992.

23. American Society of Civil Engineers, Committee on Loads and Forces on Bridges of the Committee on Bridges of the Structural Division. Recommended Design Loads for Bridges. *Journal of the Structural Division*, ASCE, Vol. 107, No. ST7, 1981, pp. 1161–1213.
24. N. M. J. Priestley. Design of Concrete Bridges for Temperature Gradients. *ACI Journal*, No. 75-23, May 1978, pp. 209–217.
25. W. Zuk. Thermal and Shrinkage Stresses in Composite Beams. *ACI Journal*, Vol. 58, 1961.
26. W. Zuk. Simplified Design Check of Thermal Stresses in Composite Highway Bridges. In *Highway Research Record 103*, HRB, National Research Council, Washington, D.C., 1965, pp. 10–13.
27. M. Soliman and J. B. Kennedy. Simplified Method for Estimating Thermal Stresses in Composite Bridges. In *Transportation Research Record 1072*, TRB, National Research Council, Washington, D.C., 1986, p. 23.
28. H. V. S. GangaRao, P. R. Raju, and H. K. Thippeswamy. *Thermal Studies on Concrete Cubes, Cylinders, Decks and Filled Grid Steel*, Research Report FHWA-WV-86. West Virginia University, Morgantown, April 1992.
29. C. Berwanger and F. A. Sarker. *Effect of Temperature and Age on Thermal Expansion and Modulus of Elasticity of Concrete*. ACI Publication, SP-39. Oct. 1973, pp. 1–22.
30. G. E. Monfore and A. E. Lentz. Physical Properties of Concrete at Very Low Temperatures. *Journal of Portland Cement Association*, May 1962, pp. 33–39.
31. J. A. Rhodes. Thermal Properties. In *Significance of Tests and Properties of Concrete and Concrete-Making Materials*, American Society of Testing and Materials, STP 169B, Committee C-9 on Concrete and Concrete Aggregates, Philadelphia, 1978.
32. A. M. Neville. *Properties of Concrete*. Wiley & Sons, New York, 1973.
33. S. G. Arzoumanidis, R. G. Burg, and J. Shmid. Creep and Shrinkage in Composite Cable-Stayed Bridges. In *Transportation Research Record 1290*, TRB, National Research Council, Washington, D.C., 1991, pp. 20–27.
34. M. A. Bradford. Deflections of Composite Steel-Concrete Beams Subject to Creep and Shrinkage. *ACI Structural Journal*, No. 99-S63, Sept.–Oct. 1991, pp. 610–614.
35. D. R. Schelling. Minimization of Composite Dead Load Effects. *Journal of the Structural Division*, ASCE, Vol. 108, No. 11, 1982, pp. 2591–2608.
36. A. Ghali and R. Favré. *Concrete Structures: Stresses & Deformations*. Chapman and Hall, Ltd., New York, 1986.
37. S. N. Sotiropoulos. *Static Response of Bridge Superstructures Made of Fiber Reinforced Plastic*. M.S. thesis. West Virginia University, Morgantown, 1991.
38. R. K. Gupta and R. W. Traill-Nash. Vehicle Braking on Highway Bridges. *Journal of Engineering Mechanics*, ASCE, Vol. 106, No. 4, Aug. 1980, pp. 641–658.
39. C. O'Connor and T. H. T. Chan. Dynamic Wheel Loads from Bridge Strains. *Journal of Structural Engineering*, Vol. 114, No. 8, 1988, pp. 1703–1723.
40. H. V. S. GangaRao. Impact Factors for Highway Bridges. *Vehicle, Tire, Pavement Interface*. ASTM STP 1164 (J. J. Henry and J. C. Wambold, eds.), American Society of Testing and Materials, Philadelphia, Pa., 1992, pp. 155–166.
41. S. Harsh and D. Darwin. Traffic-Induced Vibrations and Bridge Deck Repairs. *Concrete International*, Vol. 8, No. 5, 1986, pp. 36–42.
42. P. F. Csagoly, T. I. Cambell, and A. C. Agarawal. *Bridge Vibrational Study*, Research Report 181. Ontario Ministry of Transportation and Communication, Canada, 1972.
43. C. G. Schilling. Impact Factors for Fatigue Design. *Journal of Structural Engineering*, Vol. 108, No. 9, 1982, pp. 2034–2044.
44. D. A. Kallomalos. *Behavior of Timber Deck-Steel Stringer Bridges Subjected to Gravity Loads*. M.S. thesis. West Virginia University, Morgantown, 1989.
45. R. Lopez-Anido. *Analysis and Evaluation of Top Deck Transverse Tensile Stresses in Concrete Slab-Steel Beam Bridges*. Constructed Facilities Center, West Virginia University, Morgantown, Feb. 1992.
46. B. deV. Batchelor. An Investigation of the Fatigue Strength of Deck Slabs of Composite Steel/Concrete Bridges. In *Transportation Research Record 644*, TRB, National Research Council, Washington, D.C., 1978, pp. 153–161.
47. P. F. Csagoly and J. M. Lybas. Advanced Design Method for Concrete Bridge Deck Slabs. *Concrete International*, Vol. 11, No. 5, 1989, pp. 53–63.

Publication of this paper sponsored by Committee on General Structures.