Development of the New AASHO Specifications For Composite Steel and Concrete Bridges

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The AASHO Committee on Bridges and Structures in 1955 prepared a new specification for composite bridges. Although the new specification retained the form and the basic design philosophy of its predecessor adopted in 1944, the contents were considerably expanded to incorporate the results of extensive experimental studies and practical experiences with the design and the completed structures. The scope of the specification was restricted to structures composed of steel beams with concrete deck slabs connected by shear connectors.

The major new provisions are concerned with the design of shear connectors, the design of negative moment sections, and the effects of creep of concrete on the stress conditions in a composite beam. Furthermore, it is specified that in the stress computations concrete should be considered ineffective in resisting tension, that the composite action should not be counted on until the concrete has attained 75 percent of the required 28-day strength, and a limit is placed on the l/d-ratio of the steel beams alone as well as of the composite beams.

Background information on which the new provisions are based is discussed. It is intended as an aid in familiarizing the design engineer with the specification and as a record for future specification writers.

● THE FIRST specifications for the design of composite highway bridges were published in 1944 as a part of the "Standard Specifications" of the American Association of State Highway Officials (1) following the construction of several composite structures and a few research studies (2, 3, 4, 5, 6, 7) in the 1930's and early 1940's. The specifications were applicable to structures composed of steel or timber beams and concrete slabs. The original version was retained without change in the 1949 and 1953 editions of the AASHO "Specifications" (8 9).

The period following publication of the 1944 specifications witnessed a rapid spread of composite bridges made of steel beams and concrete slabs, resulting in a general acceptance of this type of construction (10). The building of composite bridges was accompanied by a number of field tests (11, 12, 13, 14) and by systematic experimental investigations in the laboratory (15, 16, 17, 18, 19, 20, 21, 22). Most of the field tests were made on noncomposite bridges for the purpose of determining the degree of composite action furnished by natural bond. The laboratory studies included tests of both composite and noncomposite small-scale bridges, tests of small-scale and full-size composite beams, and tests of small-scale and full-size push-out specimens. The studies were concerned primarily with the general behavior of composite structures and with the characteristics of various types of shear connectors.

The original (1944) specifications outlined only the principles on which to base the design, whereas most design details and limits on applicability were left to the discretion of the designer. As widespread use leads to the development of a multitude of methods for securing composite action and to applications of the principles to more complex structures, questions were raised concerning the applicability of the specifications in some cases (23, 24). Thus, increased use and knowledge pointed out the need for a revision of the existing specifications.

The Committee on Bridges and Structures of the American Association of State Highway Officials recognized this need and appointed a subcommittee composed of Messrs.

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E. S. Elcock, Bridge Engineer, Kansas State Highway Commission; C. N. Crocker, Bridge Engineer, Georgia State Highway Department; and E. L. Erickson, Chief, Bridge Branch, Bureau of Public Roads, to study the available information and to prepare such revisions of the 1944 specifications as appeared desirable. The subcommittee submitted its recommendations to the parent committee in December 1955. The recommendations were discussed at the regional meetings of the Bridge Committee in June 1956 and the new specifications were adopted in the fall of the same year (25).

In recognition of the fact that most of the recent construction and studies were limited to bridges composed of steel beams and concrete slabs, the new specifications pertain only to composite construction in steel and concrete. Except for this one change all original provisions are retained, although most of them are in an expanded form. A number of new provisions also are included.

The new provisions are based to a large extent on research findings. It is the purpose of this paper to discuss the background information on which the new provisions are based, in order to familiarize the design engineer with the new specifications and to record the reasons for the new provisions. In most instances, detailed information is available in readily accessible literature; in such cases, the discussion is kept on a general level and the appropriate references are given. In a few cases in which the detailed information is not readily available, the material is presented in full.

NOTATION

The symbols and notation used in this paper are defined as follows:

C_m = M_{DL}/M_{LL} ;

C_{mc} = M_{DLc}/M_{LL} ;

C_{mi} = M_{DLs}/M_{LL} ;

C_s = S_c/S_s ;

C_v = V_{DLc}/V_{LL} ;

d = diameter of studs or of the round bars used in spirals, in inches;

f_s = allowable steel stress at working load;

f_y = yield point stress of the steel;

f'_c = compressive strength of 6- x 12-in. concrete cylinders, in psi;

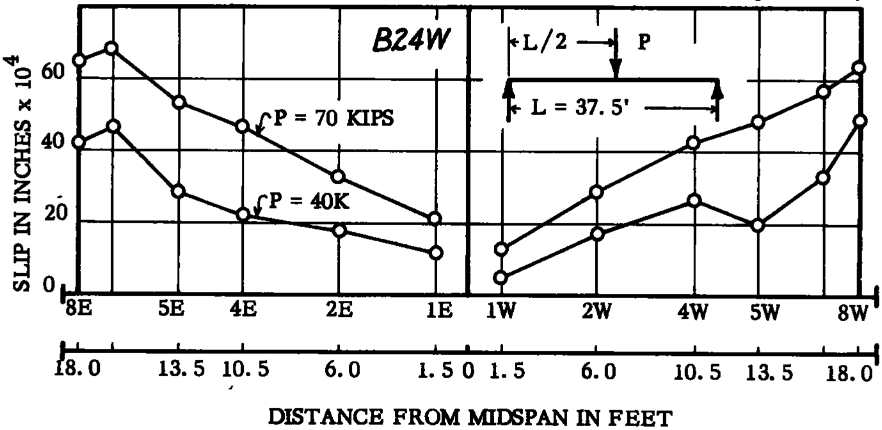

Figure 1. Slip distribution curves for a composite T-beam with mechanical shear connections.

F. S. = factor of safety for the design of shear connectors;

h = maximum thickness of the channel flange, in in.;

I = moment of inertia of the transformed composite beam section;

m = statical moment of the transformed compressive concrete area about the neutral axis of the composite section, or the statical moment of the area of reinforcement in the concrete for negative moment;

M_{DL} = maximum moment caused by the total dead load;

M_{LL} = maximum moment caused by the live load, including impact;

M_{DLc} = maximum moment caused by dead loads acting on composite section;

M_{DLs} = maximum moment caused by dead loads acting on the steel beam alone;

M_{ult} = flexural capacity of the composite beams;

N = number of live loads corresponding to the flexural capacity of the composite beam;

p = spacing of shear connectors along the beam;

Q = working load resistance value of one shear connector;

Q_{uc} = useful load capacity of one shear connector, in lb;

S = horizontal shear per linear inch at the junction of the slab and beam;

S_c = section modulus for the extreme tensile fiber of the composite beam at the maximum moment section;

S_s = section modulus for the extreme tensile fiber of the steel beam alone at the maximum moment section;

S_{pl} = M_{ult}/f_y;

t = thickness of the web of a channel shear connector, in in.;

V = total external shear due to superimposed loads applied after the concrete has attained 75 percent of its required 28-day strength;

V'_{des} = $V_{LL} + V_{DLc}$;

V_{DLc} = vertical shear caused by dead loads acting on the composite section;

V_{LL} = vertical shear caused by live loads plus impact;

V'_{ult} = $N\,V_{LL} + V_{DLc}$; and

w = length of a channel shear connector measured in a transverse direction on the flange of a beam, in in.

COMPARISON OF OLD AND NEW SPECIFICATIONS

The basic features of the 1944 specifications may be summarized as follows:

1. The design of a composite beam is carried out at the working load level and is controlled by allowable working stresses.
2. The proportioning and stress computations are based on the moment of inertia of the composite section. For beams built without temporary supports, it is assumed that the loads applied to the structure before the concrete of the slab has set are carried by the steel beams alone and only the loads applied after the concrete has set are carried by the composite section. On the other hand, for beams with effective temporary intermediate supports, all loads are assumed to be carried by the composite section.
3. The interaction between the slab and the beams is accomplished by mechanical shear connectors capable of resisting both uplift and horizontal shear. The mechanical shear connectors are designed to resist full horizonal shear due to loads carried by the composite section.

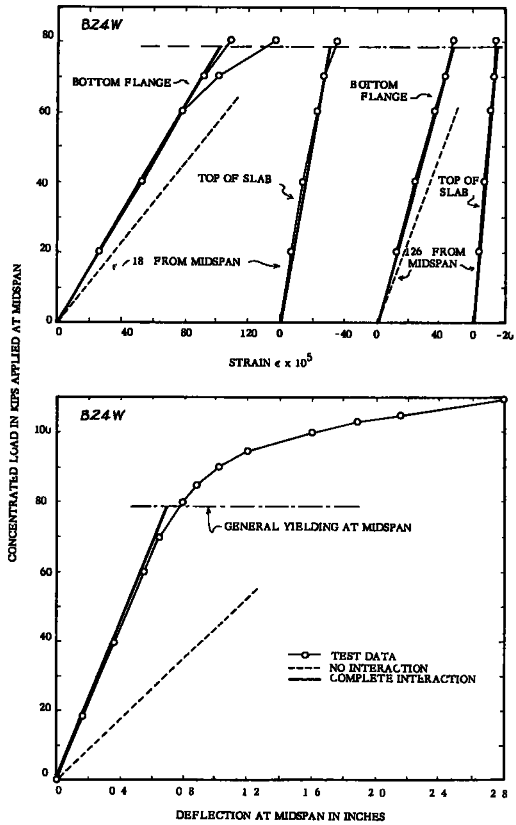

Figure 2. Load-strain and load-deflection curves for a composite T-beam with strong mechanical shear connectors.

Tests have shown conclusively that in a T-beam composed of a concrete slab and a steel beam interconnected only by mechanical shear connectors, the interaction between the slab and the beam is never entirely complete (26). Figure 1 shows the distribution of measured slips between the slab and the steel beam of a full-size composite T-beam loaded with a concentrated load at the center of a simple span (19). Although slip results in incomplete interaction, properly designed shear connectors permit only slips of such small magnitude that the decrease of interaction has practically no effect on either the governing stresses or the deflections. The measured load-strain and load-deflection curves in Figure 2 (19) are almost identical with the values computed for complete interaction, indicating almost no effect of the slips shown in Figure 1.

The moment of inertia method for the design of composite beams is based on the assumptions of complete interaction and elastic stress conditions. Thus, for the beams with properly designed shear connectors, design by the moment of inertia method at the working load level as required by the 1944 specifications is representative of actual conditions provided, of course, that the actual properties of the beam agree closely with those assumed in the design. The new specifications require the same method of design.

It is evident that when a bridge is built without temporary supports all loads applied to the structure before the concrete has hardened are carried by the steel beams alone, and that loads applied to the structure after the concrete has hardened are resisted by the composite section. On the other hand, when a bridge is built with temporary supports such that the steel beams are not stressed until the removal of shoring, all loads are carried by the composite section. The effects of shoring on the magnitude of stresses and load capacities are shown in Figure 3 (19), which gives theoretical and test curves for one composite beam designed and built without temporary supports (B24W), one composite beam designed without but built with temporary supports (B24S), and one composite beam designed and built with temporary supports (B21S). It can be seen that the use of temporary supports with no change of beam size (B24S vs B24W) decreases the stress at the design load and increases slightly the yield load, but has no effect on the ultimate load.

On the other hand, when two beams, one with and one without temporary supports (B21S vs B24W), are designed for the same stress at working load, the use of tempor-

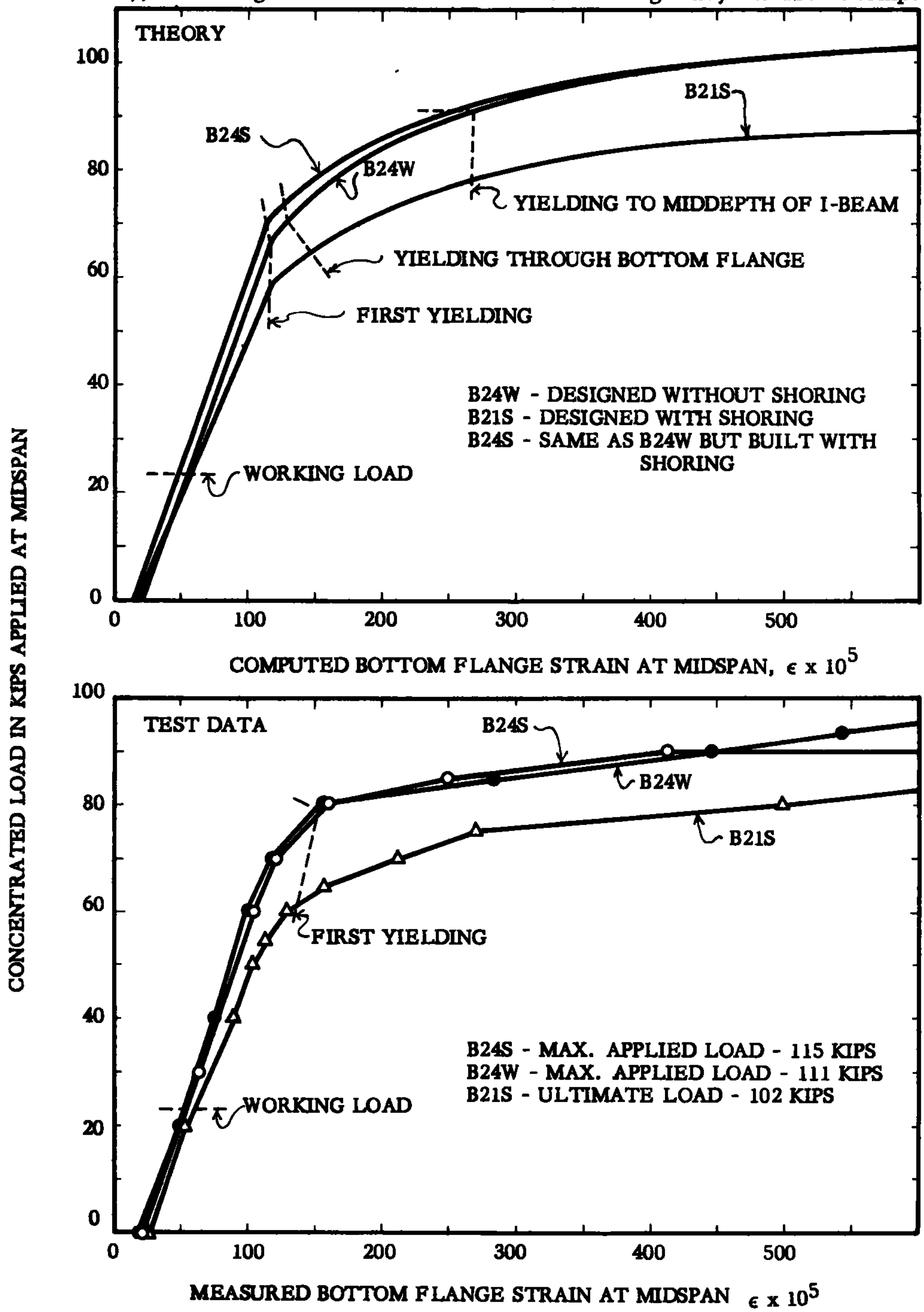

Figure 3. Effect of shoring on load-strain curves for composite T-beams.

ary supports decreases appreciably both the yield load and the ultimate load. Accordingly, if advantage is taken of shoring in design at working load level, the resulting factors of safety against both first yielding and ultimate load are smaller than for a beam designed without temporary supports. However, these factors of safety are still larger than those for noncomposite beams. The new specifications retain the old provision permitting advantage to be taken of shoring in the design of composite beams.

Carefully performed tests of new and old bridges without mechanical shear connectors have shown in some cases a definite absence (11, 12, 13), and in others a definite presence (11, 14) of bond between the slab and the top flanges of the steel beams; and have demonstrated the possible loss of bond during the lifetime of a bridge (11). Laboratory tests have shown that high over-loads cause large deformation stresses between the slab and the beam in those regions in which inelastic strains are produced (19).

Because such deformation stresses probably cannot be resisted by bond, it is unlikely that bond alone can guarantee composite action beyond the load corresponding to first yielding of the steel beams. The inability of bond to develop full ultimate load capacity of a composite beam was demonstrated indirectly by tests reported elsewhere (18, 19, 27). If the bond is broken in a structure without shear connectors, there is a substantial loss of composite action (11, 12, 13) accompanied by an increase of stresses and deflections. If the bond is broken in a structure with shear connectors, the full horizontal shear is transferred to the connectors. In a structure with weak shear connectors, such as the T-beam illustrated in Figure 4 (19), this may again lead to an appreciable loss of composite action. Accordingly, the new specifications retain the old provision requiring the design of mechanical shear connectors for full horizontal shear due to loads carried by composite section.

It can be seen that all three basic features of the 1944 specifications have been retained because they had been confirmed by laboratory and field tests. However, the same tests have shown also that these basic features are valid only if applied within certain limits. To insure that structures conform to these limitations and thus fulfill the assumptions made in design, several new requirements were added. They relate primarily to the design of shear connectors, the effect of permanent loading on steel stresses, and the conditions at the negative moment sections. Other new provisions are concerned with the effectiveness of concrete in tension, commencement of composite action, position of neutral axis, slenderness limitations, stability during erection, and expansion of concrete. All new requirements may be divided into four groups, as follows:

1. Shear connectors,
2. Negative moment sections,
3. Deformational stresses,
4. Other provisions.

Each group is discussed separately in the following.

SHEAR CONNECTORS

Design Load

Two load-slip and load-strain curves in Figure 5 compare the test results obtained for a flexible channel shear connector in a composite T-beam and in a push-out specimen (19). The slips and strains for both specimens and the loads for the push-out specimens are measured quantities. The loads for the T-beam data are the product of the computed horizontal shear and the shear connector spacing. The horizontal shear was computed as

$$S = \frac{Vm}{I} \tag{1}$$

where V is the vertical shear caused by the applied load, and m and I are the properties of the composite section. Only the envelope curves for the available test data are shown. It can be seen that the curves obtained from the tests of the two different specimens are in good agreement both qualitatively and quantatively, thus indicating that a combination of Eq. 1 with the properties of shear connectors obtained from tests of push-out specimens leads to a reliable design.

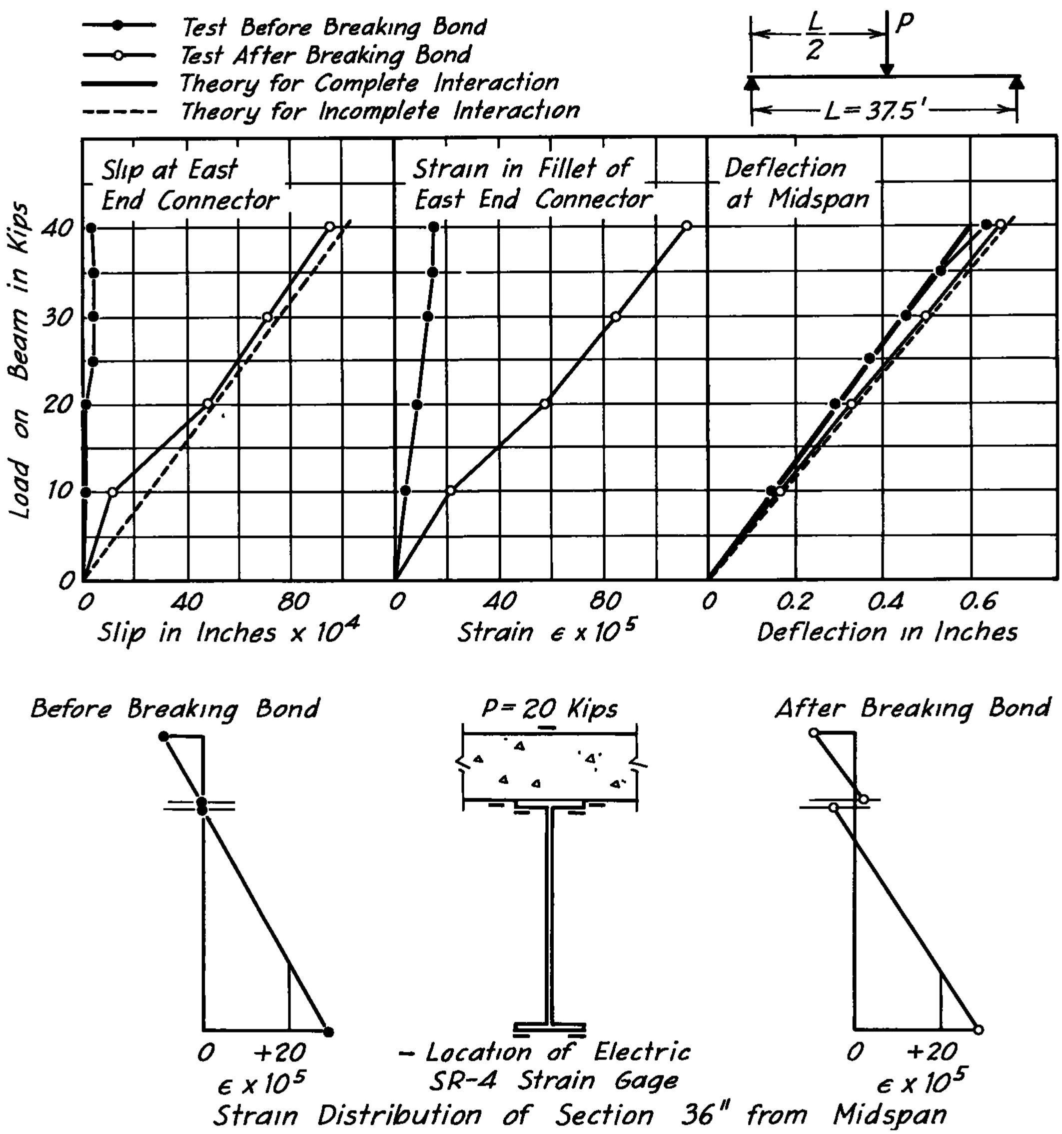

Figure 4. Effect of breaking bond in a beam with weak shear connectors.

The product of the horizontal shear, S, and the connector spacing, p, represents the load which has to be resisted by the shear connectors; that is, in terms of the working load design, the product must be equal to the working load resistance value, Q, of all connectors located at one transverse cross-section.

The shape of the load-slip and load-strain curves in Figure 5 is characteristic of those for mechanical shear connectors in general. It can be seen that both curves are non-linear. Accordingly, it is more convenient to compute the working load resistance value, Q, directly from the maximum load which the connector can carry while still performing its function satisfactorily rather than to base the resistance value on some allowable deformation or stress. Starting from this concept, the resistance value of one connector at working load may be expressed as

$$Q = \frac{Q_{uc}}{F.S.} \qquad (2)$$

where Q_{uc} is the useful (or critical) load capacity of the connector and F. S. is the desired factor of safety.

Useful Load, Q_{uc}

It has been pointed out that the assumption of complete interaction between the slab and the steel beams is approximately valid only if the shear connection permits almost no slip. If the slip exceeds a certain value, a significant decrease of interaction between the slab and the beam follows and causes an appreciable increase of both the stresses and deflections of the T-beam. Studies of various types of shear connectors have shown that the slip at failure of any type of connector is always considerably in excess of the tolerable magnitude.

Accordingly, the load on which the design should be based, (the useful load capacity of the connector, Q_{uc}) is always lower than the maximum load obtained from the push-out tests of connectors to destruction. For flexible types of connectors, the slip remains small until yielding takes place in the connector. For stiff connectors, the slip remains tolerably small until large inelastic deformations of the concrete take place. Accordingly, the useful load capacity is the load at which either the steel of the connector begins to yield or the inelastic deformations of the concrete begin to increase rapidly.

Equations for the useful load capacity of flexible channel, stud, and spiral connectors were derived from the results of tests of full-size push-out specimens. They are semi-empirical and empirical expressions based on tests covering almost complete practical ranges of the involved variables. The derivations may be found in the references given for each particular formula.

The capacity of a flexible channel connector made of structural grade steel may be computed from (28):

$$Q_{uc} = 182\,(h + 0.5t)\,w\,\sqrt{f'_c} \tag{3}$$

in which t, h and w are the channel dimensions and f'_c is the strength of concrete. Eq. 3 is compared with the available test data (19) in Figure 6a. The values "Q_{uc} test" are the test loads corresponding to a maximum measured connector strain of 0.0011. They represent the following ranges of variables: strength of concrete f'_c = 1,970 to 6,320 psi, channel web thickness t = 0.17 to 0.50 in., maximum channel flange thickness h = 0.377 to 0.944 in., channel length w = 4.0 to 8.0 in., and channel height = 3.0 to 5.0 in.

The capacity of stud connectors is given by (22):

For studs having diameter $d < 1$ in.,

$$Q_{uc} = 332\,d^2\,\sqrt{f'_c} \tag{4a}$$

$$S = \frac{Vm}{I}, \qquad Sp = Q, \qquad Q = \frac{Q_{uc}}{F.S.},$$

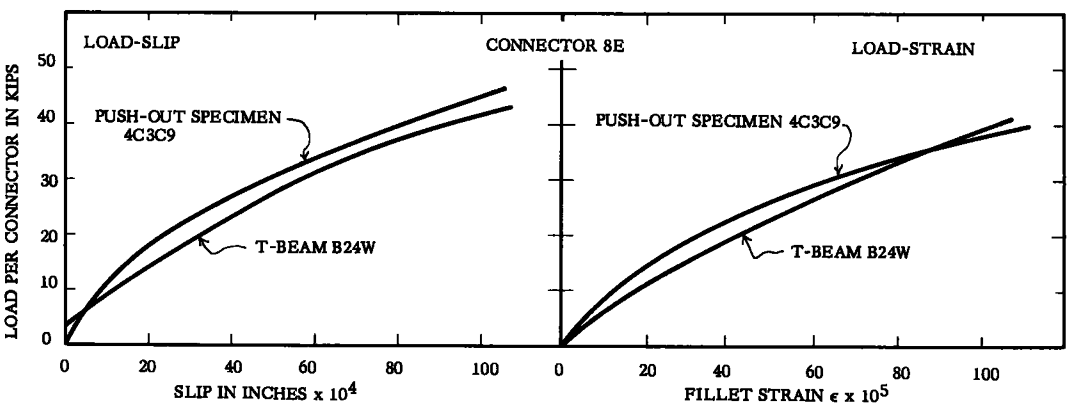

Figure 5. Characteristic curves for flexible channel connectors.

For studs having diameter d $\geqq$ 1 in.,

$$Q_{uc} = 316\, d \sqrt{f'_c} \qquad (4b)$$

Eqs. 4a and 4b apply to studs 4 in. or more in height. For studs 3 in. high, the values given by Eqs. 4a and 4b should be reduced by 15 percent. Comparisons of Eqs. 4a and 4b with the test data (22, 29) are shown in Figure 6b. The test data represent the following ranges of variables: concrete strength f'_c = 3,000 to 5,380 psi, stud diameter d = 0.5 to 1.25 in., stud length = 3 to 8 in., and stud spacing = 2 to 4 in.

The capacity of one pitch of spiral is given by (30)

$$Q_{uc} = 3{,}840\, d \sqrt[4]{f'_c} \qquad (5)$$

which was derived on the basis of extensive tests carried out in Switzerland including concrete strength f'_c spiral bar diameter d, spiral diameter, and pitch of spiral as variables. However, the report of the tests (30) does not include sufficient details to permit a reliable determination of the test value of the useful capacities for the individual specimens. Therefore, Eq. 5 is compared in Figure 6c only with the results of

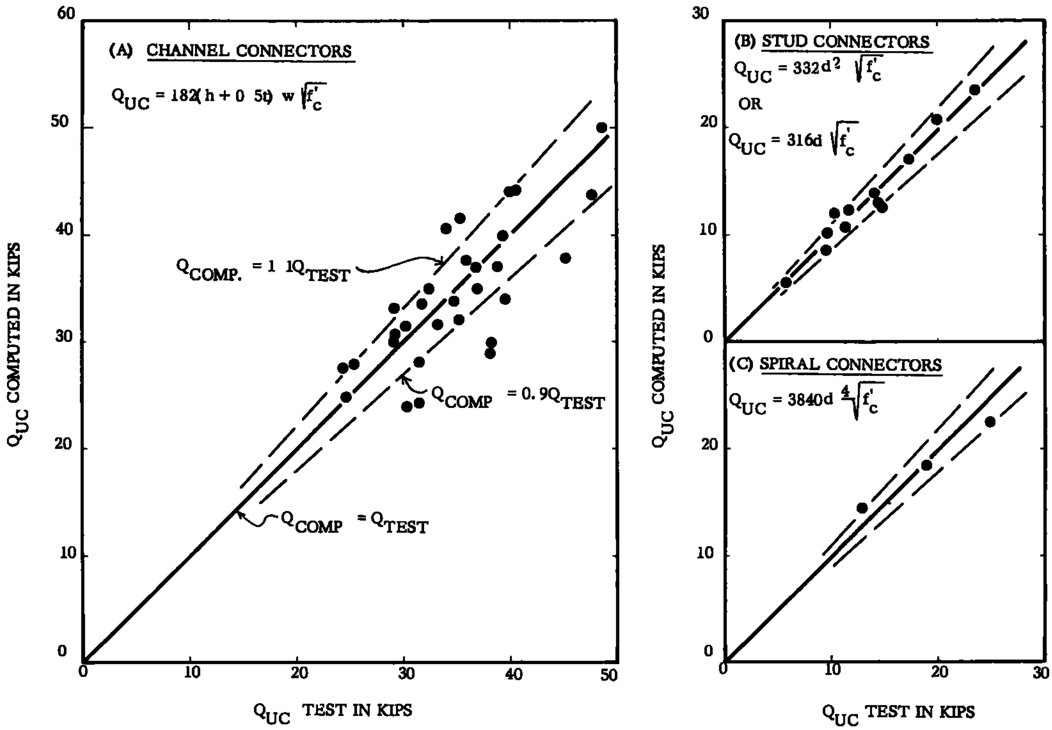

Figure 6. Comparison of computed and test values of useful capacities.

three recent tests (31) including the spiral bar diameter as the only variable. The bar diameters were ½, ⅝ and ¾ in.

Flexible channels, studs, and spirals are the only shear connectors for which test data are available for the useful loads. Although tests have been made on numerous other types of connectors, the published data do not contain the information necessary for determining the useful load carrying capacity.

Formula for Factor of Safety

A composite beam is not only stiffer than its noncomposite counterpart, but also has a considerably higher reserve strength beyond the yielding of the steel beam—provided the shear connection is capable of retaining a high degree of interaction up to the ultimate flexural capacity of the composite beam. Furthermore, the tests have shown that if the load on individual connectors computed with the aid of Eq. 1 does not exceed Q_{uc} at ultimate, the connection is sufficiently stiff to guarantee practically complete interaction both before and after yielding of the steel beam. On the other hand, if the connectors are designed to reach Q_{uc} at a lower load level, the degree of interaction may not be entirely satisfactory, even before yielding of the steel beams (24). It is desirable, therefore, to select a factor of safety for the design of shear connectors capable of guaranteeing composite action up to full flexural capacity.

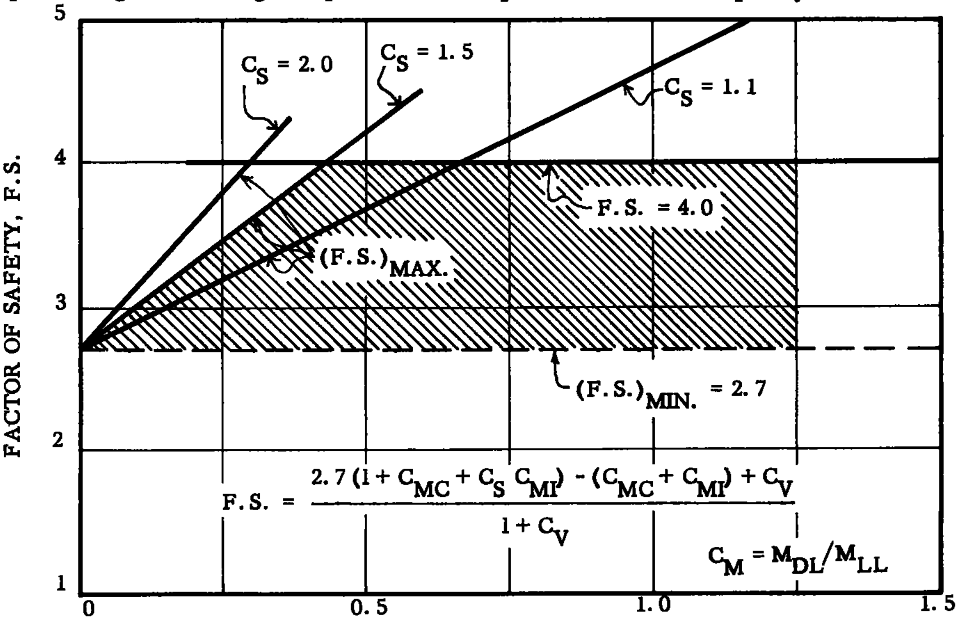

Figure 7. Variation of the factor of safety.

The load positions for the maximum moment governing the design of beams and for the maximum horizontal shear governing the design of shear connectors are ordinarily different because of moving loads. In such case, the full flexural capacity of the structure can be realized only if the shear connectors permit the load corresponding to flexural capacity to move into its critical position without impairing composite action in the process.

The factor of safety may be determined by Eq. 2. The working load resistance value Q must be equal to the design load on one shear connector, and the useful load capacity Q_{uc} must be equal to the load acting on one shear connector at ultimate. Accordingly, using Eq. 1, the following relationship is valid for Q:

$$Q = \frac{V'_{des}\, m}{I}\, p \qquad \text{(6a)}$$

in which V'_{des} is the vertical shear resulting from the design live load and the dead load carried by the composite section $(V_{LL} + V_{DLc})$. According to the previous discussion, Eq. 1 may be used also at ultimate. Thus,

$$Q_{uc} = \frac{V'_{ult}\, m}{I}\, p \qquad \text{(6b)}$$

in which $V'_{ult} = N\, V_{LL} + V_{DLc}$ and N is the number of live loads corresponding to the flexural capacity of the structure. Thus, the factor of safety may be expressed as

$$\text{F.S.} = \frac{Q_{uc}}{Q} = \frac{N\, V_{LL} + V_{DLc}}{V_{LL} + V_{DLc}} \qquad \text{(7a)}$$

Dividing both the numerator and denominator by V_{LL} and letting $V_{DLc}/V_{LL} = C_v$ gives

$$F.S. = \frac{N + C_v}{1 + C_v} \tag{7b}$$

The number of live loads causing flexural failure may be obtained from the following in terms of the design moments and the moment capacity of the maximum moment section:

$$M_{ult} = N\ M_{LL} + M_{DLc} + M_{DLs} \tag{8a}$$

where M_{DLs} is the moment caused by the dead loads carried by the steel beams alone. After rearranging, and designating $M_{DLc}/M_{LL} = C_{mc}$, and $M_{DLs}/M_{LL} = C_{mi}$, the number of live loads may be expressed as

$$N = \frac{M_{ult}}{M_{LL}} - (C_{mc} + C_{mi}) \tag{8b}$$

But M_{LL} may be expressed in terms of the allowable stress and the section properties of the maximum moment section, because

$$f_s = \frac{M_{DLs}}{S_s} + \frac{M_{DLc} + M_{LL}}{S_c} \tag{9}$$

In which S_s is the section modulus for the extreme tensile fiber of the steel beam alone at the maximum moment section and S_c is the section modulus for the extreme tensile fiber of the composite beam at the maximum moment section. Rearranging and designating $S_c/S_s = C_s$,

$$M_{LL} = f_s S_c \frac{1}{1 + C_{mc} + C_{mi}\ C_s} \tag{10}$$

The ultimate moment, M_{ult}, is a function of the yield point stress of the steel of the beam. Designating $M_{ult} = f_y\ S_{pl}$, the number of live loads may be written as

$$N = \frac{f_y}{f_s}\ \frac{S_{pl}}{S_c}\ (1 + C_{mc} + C_s\ C_{mi}) - (C_{mc} + C_{mi}) \tag{11}$$

For structural grade steel, the stress ratio is specified at 1.83. It has been shown (28) that the ratio of the plastic section modulus, S_{pl}, to the elastic section modulus of the composite section, S_c, for composite sections made of rolled beams varies usually from 1.38 to 1.49. Accordingly, the product of the two ratios may be taken approximately as a numerical constant, 2.7. Thus, the maximum number of live loads which can cross a composite structure may be approximated as

$$N = 2.7\ (1 + C_{mc} + C_s\ C_{mi}) - (C_{mc} + C_{mi}) \tag{12}$$

The factor of safety required for the design of shear connectors may then be computed from

$$F.S. = \frac{2.7\ (1 + C_{mc} + C_s\ C_{mi}) - (C_{mc} + C_{mi}) + C_v}{1 + C_v} \tag{13}$$

It should be noted that the number of live loads which a composite beam can carry, and thus also the coefficients C_{mc}, C_{mi}, and C_s, should be computed at the section of maximum moment. In a continuous beam having several sections of maximum moments, the number of live loads, N, should be computed for each section of maximum moment and the smallest N-value should be used for computing the required factor of safety.

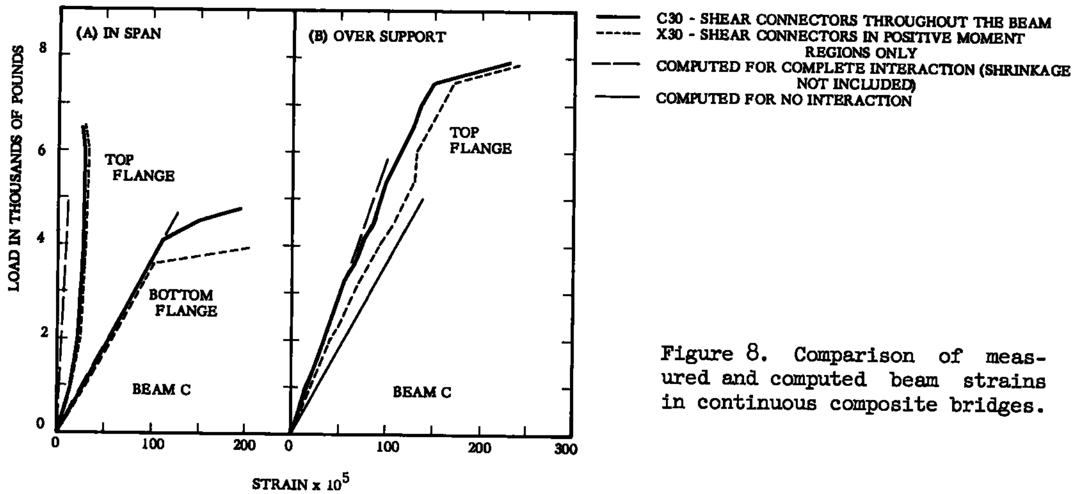

Figure 8. Comparison of measured and computed beam strains in continuous composite bridges.

Variation of Factor Safety

The factor of safety for the design of shear connectors varies with the ratios of composite and noncomposite dead load moments to live load moment, with the ratio of composite vertical shear to live load vertical shear, and with the ratio of the composite section modulus to the noncomposite section modulus. It can be shown that Eq. 13 gives the largest values when all dead loads are resisted by the steel beam alone; that is, for $C_{mc} = 0$ and $C_v = 0$. The corresponding factors of safety are plotted in Figure 7 as sloping full lines converging to 2.7 for $C_m = M_{DL}/M_{LL} = 0$, each line representing a different value of the section modulus ratio $C_s = S_c/S_s$. The ratio C_s is always larger than 1.0 and in highway bridges is unlikely to reach 2.0. Large values of C_s correspond to short bridges, thus to small values of C_m. The values most commonly encountered in the design of composite bridges are $C_m = 0.3$ to 1.0 and $C_s = 1.3$ to 1.1.

The smallest values of the factor of safety correspond to structures in which all dead load is carried by the composite section; that is, $C_{mi} = 0$. These minimum values vary from point to point on the bridge, depending on the shear ratio, C_v, at the particular section. In simple beams the factor of safety has the minimum value at the supports, where it is equal to 2.7. This value is shown in Figure 7 as a horizontal dashed line.

In most bridges a part of the dead load is carried by the steel beams alone and a part, such as sidewalks, curbs, railings, wearing surface, etc., is carried by the composite section. The corresponding factors of safety are located in Figure 7 somewhere between the horizontal dashed line and the sloping full lines. It can be seen that ordinarily the factor of safety is not likely to exceed the value of 4.0 shown in Figure 7 as a horizontal full line; it will ordinarily vary within the shaded area.

Article 3.9.5 of the new specifications permits the use of either Eq. 13 or a constant F.S. = 4.0.

Limitations

The purpose of the shear connectors is to make the slab and the steel beams of the bridge act as a unit. An ideal connection would connect the two elements at every point of contact. Mechanical shear connectors cannot fulfill this ideal condition, but can approach it if spaced closely together. If, on the other hand, the spacing of mechanical connectors is large, the connection permits differential deformations of the slab and the beam in the vertical direction (19). It is considered desirable, therefore, to impose an upper limit on the longitudinal spacing of connectors. Article 3.9.5 sets the maximum spacing at 24 in.

The concrete next to the shear connectors is subjected to high stresses, even at working loads. It is able to resist these stresses without damage because the stresses are localized on a small portion of the slab area, and because the highly stressed concrete is confined on all sides. To provide sufficient confinement, it is necessary to place the connectors some distance away from the edge of the I-beam (19). Article 3.9.2 requires a minimum distance of 1 in. between the edge of the I-beam and the edge of the shear connector.

Article 3.9.2 requires also a minimum of 1 in. of concrete cover over the tops of the shear connectors. This provision is intended primarily to insure easy placing and finishing of the concrete slab.

NEGATIVE MOMENT SECTIONS

It is well known that the tensile strength of concrete is only about one-tenth of its compressive strength and that relatively small tensile stresses can cause cracking. Tensile stresses in the slab can result either from shrinkage or from loading and can occur whether the bridge is designed as composite or noncomposite. Shrinkage cracks can occur anywhere on the bridge; cracks due to loading occur in the negative moment

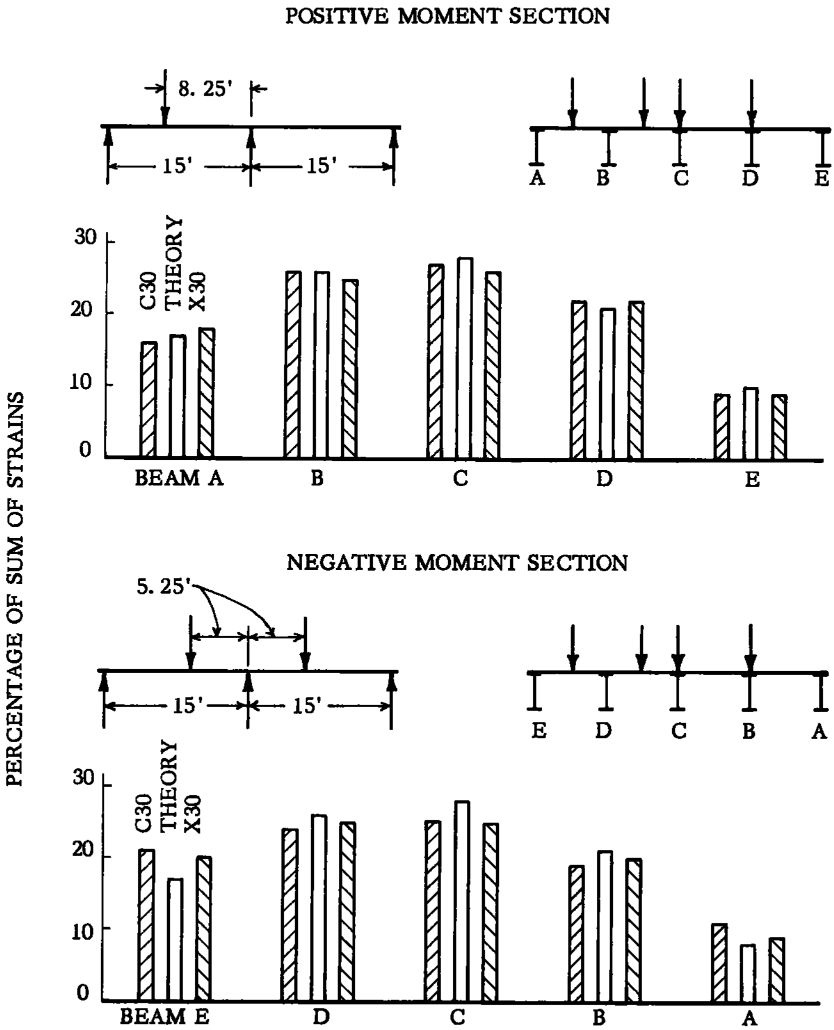

Figure 9. Transverse Distribution of Beam Strains: (a) Bridge C30, Shear Connectors Throughout Bridge; (b) Bridge X30, Shear Connectors in Positive Moment Regions Only; (c) Theory, Distribution for Equivalent Simple-Span Bridge.

regions. If shrinkage cracking occurs in the span of a composite bridge, the cracks close on application of load.

The effect of shrinkage cracking on beam stresses in the span is shown in Figure 8a (20). The stresses in the top flange are increased as a result of cracking, but the bottom flange stresses remain almost unchanged. Thus, tensile cracking of the slab in the span has the same qualitative effects on the beam stresses as the shrinkage stresses discussed in a later portion of this paper. On the other hand, tensile cracking of the slab in the negative moment regions of a composite bridge, regardless of whether it is caused by shrinkage or load, renders the concrete slab ineffective in resisting any stresses because, on application of load, such cracks only tend to open wider. Tensile cracking of an ordinary reinforced concrete slab in the negative moment regions of a continuous bridge is unavoidable. Thus, in the negative moment regions only the slab reinforcement can act compositely with the steel beams.

A continuous composite bridge may be built with shear connectors either in the positive moment regions or throughout the length of the bridge. As long as the slab is continuous throughout the length of the bridge (no expansion joints), the slab reinforcement in the negative moment regions interacts with the steel beams in both types of construction, but to a different degree. This is shown in Figure 8b, in which the stresses measured in the top beam flange over the support of both types of continuous model bridges are compared with values computed for full and for no interaction between the slab reinforcement and the steel beam. It can be seen that in the bridge with shear connectors throughout the beam (full line), the slab reinforcement was fully effective; in the bridge with shear connectors in the positive moment regions only (dotted line), the slab reinforcement was only partly effective (about half way between full and no interaction). Obviously, the slab of the second bridge had no choice but to elongate over the support because it was anchored to the beam at the points of contraflexure.

The third important test finding concerning the action of continuous composite bridges is illustrated in Figures 8 and 9 (20). Figure 9 shows the transverse distribution of strains in composite model bridges C30 and X30 at the section of maximum positive moment and over the support. It can be seen that the distribution of strains, and thus also of moments, at the two locations is nearly the same. The test data are compared also with the values computed for midspan of a simple span bridge having the same cross-section and having a span length equal to the distance from the outside support to the dead load point of contraflexure of the continuous bridge.

The theoretical values in Figure 8 were computed with the aid of the conventional elastic analysis for a two-span continuous bridge taking the stiffness in the positive moment regions as that of the composite section made up of the steel beams and the concrete slab; and the stiffness in the negative moment region as that of the composite section made up of the steel beams and the slab reinforcement. It can be seen in Figures 8 and 9 that the test data are in good agreement with the computed values. Furthermore, a good agreement also was found between this approximate analysis and a more exact theoretical analysis (20). Thus, the conclusion may be drawn that the use of an elastic analysis in combination with the usual load distribution factors is justified, and no special provisions are needed for the design of continuous composite bridges.

Based on the considerations previously discussed, Article 3.9.2 prescribes that if shear connectors are provided in the negative moment regions the slab reinforcement may be considered as contributing to the moment resisting capacity of the section. If, however, shear connectors are provided only in the positive moment regions, the steel beams must be designed to resist the full negative moment.

DEFORMATIONAL STRESSES

Concrete is inherently a dimensionally unstable material. Depending on its composition and on atmospheric conditions, concrete may either shrink or expand. If loaded, concrete continues to deform with time—it creeps. In a composite beam, concrete is tied to the steel so that every deformation of the slab induces deformation of the beam. The deformation of the steel beam leads to deformational stresses both in the beam and in the slab. Thus, all three phenomena (creep, shrinkage, and expansion of concrete) affect the stress conditions in a composite beam. Because experimental evidence relating to the effects of concrete instability on the stresses in composite beams is scarce, recourse must be made to analyses utilizing the general knowledge of volume changes of concrete (32).

In the positive moment sections of bridges in which all or some portion of the dead load is carried by the composite section, the slab is subjected to permanent compressive stresses. The resulting creep increases the compressive stresses in the top flanges of the steel beams and the tensile stresses in the bottom flanges, and decreases the compressive stresses in the slab. Theoretically, the magnitude of the creep stresses in any particular beam may be computed if the creep characteristics of the slab concrete are known. Such analysis is complicated and, in view of the variability of the creep characteristics of concrete, of questionable value. A simpler procedure is to increase the value of the modular ratio in the computation of the section properties of the composite beams (17). A multiplication factor of 3 on the usual modular ratio is required by Article 3.9.1 for consideration of creep effects.

It should be noted in connection with creep that it decreases the loads acting on shear connectors. Thus, the critical loading of shear connectors occurs immediately after construction before a substantial creep can take place.

During the period of a few months after construction, the concrete of the slab usually is subject to shrinkage. Shrinkage has a similar effect on the stress conditions in a composite beam as creep caused by compressive stresses; it places the top flanges of the steel beams in compression, the bottom flanges and slab in tension.

If the slab were free to shrink and had no longitudinal reinforcement, a unit shrinkage of 0.0003 to 0.0006 in. per in. would be expected to occur (32). The longitudinal reinforcement in the slab decreases the amount of shrinkage, and creep due to the tensile stresses resulting from the restraint offered by the steel beams tends to reduce further the amount of shrinkage deformation in a composite beam. A few measurements on full-size composite T-beams made in the laboratory (19) have indicated a net unit shrinkage of about 0.0002. Computations of shrinkage stresses for a unit shrinkage of 0.0002 have indicated that the resulting increase of stress may reach about 1,000 psi in the tension flange and about 4,500 psi in the compression flange. The new specifications do not require consideration of shrinkage in the design because the 25 percent overstress permitted by Article 3.4.1 for Group IV loading, including shrinkage, is not likely to be exceeded.

It has been stated in the discussion of the provisions for the design of negative moment sections that shrinkage may cause cracking of the slab. Shrinkage cracking over the support does not require any special consideration because it is assumed in the design that concrete cannot carry tension. Studies of typical composite cross-sections indicate that the effect of shrinkage cracking on stresses at the positive moment sections may be expected to be of similar order of magnitude to those caused by shrinkage without cracking of the slab.

It is known that concrete made of some aggregates expands. The effects of expansion of concrete on the stresses in a composite beam may be considered as reversed shrinkage if the expansive characteristics of the concrete are known.

OTHER PROVISIONS

The following new provisions are based partly on known behavior of structural materials, partly on experiences gained in design, and partly on experience with existing composite bridges.

Recommended Position of Neutral Axis. In most existing composite bridges, the neutral axis of the composite section is located below the top flange of the steel beam. Such structures are known to have given satisfactory performance. If, however, the neutral axis is located in the slab, the resulting tensile cracking of the slab may possibly have some detrimental effect on the composite action. The specifications, therefore, state a preference for designs with neutral axis in the steel section.

Concrete Ineffective in Tension. Because of its low tensile strength, concrete is usually not counted on to resist tension. Article 3.9.1 stipulates the ineffectiveness of concrete in tension as a general requirement also for the design of composite structures except for deflection computations.

Commencement of Composite Action. Loading of concrete at an early age may result in excessive creep. To prevent excessive creep deformations, it is specified for design purposes that interaction between the slab and the steel beams becomes effective when the concrete has attained 75 percent of the required 28-day strength.

Slenderness Limitations. Experiences with composite bridges have shown excessive vibrations of some slender structures. In an attempt to prevent such vibrations, it is recommended in Article 3. 9. 6 to limit the slenderness ratio of the steel beams to 30 and the slenderness ratio of the over-all section to 25. This limitation is comparable to a similar requirement for steel structures.

Stability During Erection. In composite bridges with unsymmetrical steel sections, the design stresses in the top flange are often close to the full allowable value of 18,000 psi in compression. If the neutral axis of the composite section is located in or close to the top flange and the structure is designed and built without temporary supports, most of the design stress is due to the weight of the steel beam and the slab. Because the effective lateral support furnished by the slab is absent until the slab concrete has hardened, it is necessary to investigate the stability of the steel beams.

CONCLUDING REMARKS

It may be stated in conclusion that the 1956 revision of the "AASHO Specifications" for the design of composite bridges follows the same basic principles as those contained in the original (1944) version. However, the revised specifications contain numerous new requirements which should result in more uniform designs. Detailed requirements for the design of shear connectors, based on extensive tests carried out during the past decade, represent a major change. New requirements for the design of negative moment sections and for consideration of creep are based both on experimental evidence and on general knowledge of the behavior of reinforced concrete. A number of other new provisions and recommendations are the result of experiences with composite bridges in service and with the design of composite structures.

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