Composite Construction for I-Beam Bridges

I. M. Viest, *Research Assistant Professor of Theoretical and Applied Mechanics*, and
C. P. Siess, *Research Associate Professor of Civil Engineering*
University of Illinois

This paper deals with the composite bridge consisting of longitudinal steel I beams supporting a reinforced-concrete slab connected to the beams in such a manner that the bridge acts similarly to a monolithic structure. Three subjects are treated: 1) the behavior of composite steel and concrete T beams, 2) the function and action of the shear connection between the concrete slab and the steel I beams, and 3) the behavior of composite I-beam bridges of both simple and continuous spans. Particular attention is given to the differences between composite and noncomposite construction. It is shown that the composite structure is tougher than its noncomposite counterpart but that this greater toughness will be realized fully only if the shear connection is capable of providing good interaction between the steel beams and the concrete slab at all stages of loading up to the ultimate capacity of the structure. Criteria for the design of such composite T beams and their shear connections are also discussed.

The material included in this paper is based primarily on the results of extensive analytical and experimental studies made at the University of Illinois in coöperation with the Illinois Division of Highways and the Bureau of Public Roads.

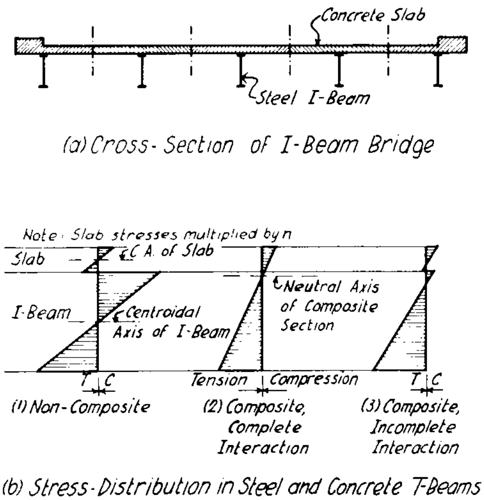
● ONE of the most-common types of highway bridges is the I-beam bridge with a reinforced-concrete slab as the roadway. Such a structure may be built with the slab either resting freely on the top flanges of the I-beams or connected rigidly to them. The latter type, the composite I-beam bridge, is a relatively recent development, and its popularity has been increasing steadily. The practical applications of composite construction have raised numerous problems which have been the object of several experimental investigations both in this country and abroad. Among the most extensive studies are those of Roš in Switzerland (*1*), Maier-Leibnitz (*2*) and Graf (*3, 4*) in Germany, Thomas and Short in England (*5*), and the studies made at the University of Illinois (*6, 7, 8, 9, 10*). References to other work on composite construction may be found in a selective bibliography included in *Bulletin 405* of the University of Illinois Engineering Experiment Station (*9*).

The results of experimental and analytical studies have been reported in detail in the references quoted above, but a general summary and discussion of the knowledge obtained through these studies is lacking. It is the purpose of this paper to fill this gap by presenting a general picture of the behavior of composite I-beam bridges. However, the scope of the paper is limited to a discussion of those effects inherent to composite construction; it does not include such general aspects of the behavior of I-beam bridges as the distribution of wheel loads. For a discussion of some of those problems the reader is referred to a paper by Siess and Veletsos (*11*).

A second objective of this paper is to discuss criteria for the design of composite I-beam bridges. Whereas behavior is a question of facts substantiated by experimental evidence, design criteria are necessarily a combination of facts and opinions, or in other words, an interpretation of the experimental evidence in the light of a design philosophy. For this reason, the parts of this paper dealing with criteria for design should be regarded as representing to some extent the opinions of the authors.

The paper is divided into three parts: 1) composite T beams, 2) shear connection, and 3) composite I-beam bridges. The effects of the composite action between the steel I beams and the concrete slab may be illustrated best by discussing the behavior of a composite T beam composed of a single I beam with an isolated section of the slab: the behavior of such structural members is

dealt with in the first part. The shear connection between the beam and the slab is an essential part of a composite structure, and it is discussed in the second part. Some aspects of the behavior of composite I-beam bridges depend on the interaction of all elements of the bridge; for example, the effect of composite action on the transverse distribution of load in both simple-span and continuous bridges and the ultimate load-carrying capacity of I-beam bridges belong in this category. Problems of this nature are dealt with in the third part.



(a) Cross-Section of I-Beam Bridge

(b) Stress-Distribution in Steel and Concrete T-Beams

Figure 1. Composite T beams.

COMPOSITE T BEAMS

The I-beam bridge shown schematically in cross section in Figure 1(a) may be thought of as being made up of several concrete-and-steel T beams, each consisting of one steel I beam and a portion of the slab. The deformations of the T beams are, of course, interdependent. If a load is applied to one T beam, a portion of the load is transferred by the slab to the remaining beams; as a result, all beams deform. Since the load-distributing action of the slab complicates the behavior of the bridge, it is simpler to consider at first only the effects of composite action on the behavior of an isolated composite T beam instead of the whole bridge.

Degree of Composite Action

Three types of T beams may be distinguished according to the amount of interaction between the slab and the beam: (1) noncomposite, (2) fully composite, and (3) partly composite. In a noncomposite beam the slab rests freely on the top of the I beam; that is, under the action of a load the bottom surface of the slab and the beam deform independently when loaded, and the distribution of flexural stress is similar to that shown schematically in Figure 1(b) for a noncomposite beam. The only interdependence between the deformations of the slab and the beam can be found in their deflections, which are approximately the same for both elements. The equal deflections make it possible to determine the proportion of the load carried by each of the two elements: the total load is distributed to the slab and to the beam roughly in proportion of their stiffnesses. Since the stiffness of the slab is small compared to that of the beam, the load carried by the slab of a noncomposite beam is only a small fraction of the total load carried by the I beam and the slab. For beams corresponding to those used in highway-bridge construction, the slab may carry about 5 to 15 percent of the total load, as long as the concrete of the slab does not crack in tension. However, ordinarily the slab of a noncomposite bridge cracks under the action of working loads and the stiffness of the slab is thus considerably reduced; this leads to a reduction of the contribution of the slab to the load-carrying capacity of the structure. Therefore, in the design of noncomposite I-beam bridges, it is reasonable to assume that all of the load is carried by the steel I beams alone.

In a fully composite T beam, the slab is connected rigidly to the I beam and therefore cannot slide along the beam. Consequently, the deformations of the bottom surface of the slab must be the same as the deformations of the top surface of the I beam, and the stress distribution is the same as in a monolithic structure (Fig. 1(b), composite beam). This integral action requires the transfer of horizontal shear between the slab and the beam by some sort of shear connection located at the contact surface between the slab and the I beam.

In a partly composite T-beam the slab is connected to the I-beam but the connection permits some slip between the two elements. Since the slip is smaller than in a noncomposite T beam, a partly composite T beam is an intermediate case between a noncomposite

and a fully composite beam. Consequently, the stress distribution for a partly composite T beam, shown in Figure 1(b), is also intermediate between those for noncomposite and fully composite beam. For a discussion of composite beams with incomplete interaction the reader is referred to a paper by Newmark, Siess, and Viest (*12*) and to the appendix of *Bulletin 396* of the University of Illinois Engineering Experiment Station (*8*).

The slab is usually connected to the I beam by a number of individual steel shear connectors welded to the beam and embedded in the concrete slab. When transmitting the horizontal shear from the beam to the slab, the shear connectors exert pressure on the surrounding concrete and the concrete deforms. Consequently some relative movement or slip occurs between the slab and the beam, and the interaction is not complete. However, it is possible to provide shear connectors of such strength and stiffness that the degree of interaction is very high, with the result that composite T beams having properly designed shear connectors may be designed as beams with complete interaction. For this reason the remainder of this paper deals primarily with fully composite beams.

Behavior of Composite T Beams

The behavior of a composite T beam may be illustrated best by considering its load-deformation characteristics. Two load-strain curves are shown in Figure 2. Both curves are for the same T beam, except that the values for the lower line were computed for no interaction between the slab and the beam.

If a composite T beam is loaded with a continuously increasing load, the load-strain relationship is at first linear. After exceeding the yield point strain in the I beam, the strain increases at an increasing rate until the slab crushes. The crushing of the slab is accompanied by a permanent decrease of the load-carrying capacity of the T beam. Thus, the behavior of a composite T beam may be divided into three stages: 1) elastic stage, before yielding of the I beam, 2) inelastic stage, between first yielding of the I beam and crushing of the slab, and 3) after crushing of the slab.

During the first stage of loading the behavior of a composite T beam is elastic. The position of the neutral axis does not shift until yielding occurs in the steel beam. Ordinarily, the neutral axis of the composite cross section is either in the beam or slightly above the bottom of the slab. Thus the slab remains uncracked and the section properties may be computed on the basis of the gross area of the slab. If the behavior of a composite T beam is compared with that of a corresponding noncomposite T beam, it will be found that the deformations of the composite beam are smaller. For the beams of dimensions commonly encountered in highway-bridge con-

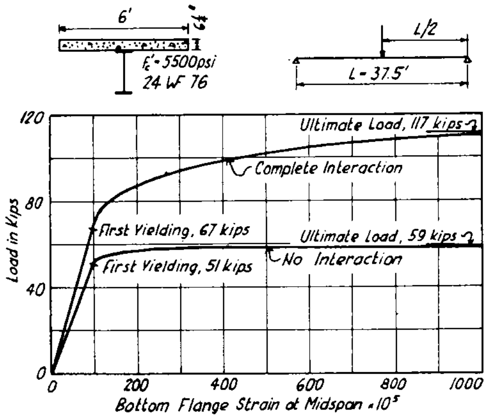


Figure 2. Effect of composite action on bottom-flange strains.

struction, the bottom flange stresses, which govern the design, are decreased by about 10 to 30 percent through composite action. The magnitude of the difference depends primarily on the ratio of the slab and beam areas; an increase in this ratio results in an increased effectiveness of the composite action in reducing the governing stresses. The midspan deflection of a simple-span, composite T beam is ordinarily 20 to 60 percent smaller than that of a similar noncomposite T beam. The effect of composite action on the governing stresses depends on the relative section moduli, whereas the effect on deflections depends on the relative moments of inertia. The section moduli are directly proportional to the moments of inertia and inversely proportional to the distance of the bottom flange of the I beam from the neutral axis. Since both the moment of inertia and the distance of the bottom flange from the neutral axis are larger for the composite section than for the noncomposite one, the effects of composite action tend to compensate and the difference between

the section moduli is smaller than the difference between the moments of inertia. Consequently, the effect of the composite action on the governing stresses is less marked than the effect on the deflections.

The first stage of behavior ends and the second stage begins when the steel beam starts to yield at the location of maximum stress, which is usually the bottom flange at midspan. Ordinarily, it is assumed that first yielding occurs when the critical stress reaches the yield-point value of the steel in the I beams of the web and the flanges. The magnitude of these stresses may be significantly large; for example, in the tests made at the University of Illinois (*9*) compressive stresses of over 30,000 psi. and tensile stresses of over 18,000 psi. have been observed in wide-flange I beams, 21 in. deep.

Whereas the residual stresses due to rolling are independent of whether a T beam is composite or noncomposite, the residual stresses due to welding the shear connectors and due to shrinkage of the slab are present only in com-

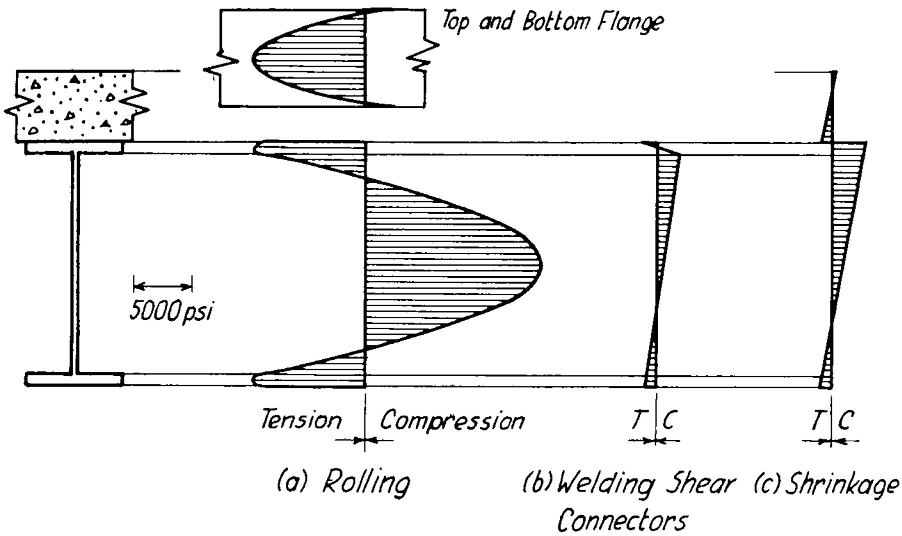


Figure 3. Residual stresses in composite T beams.

as determined from usual coupon tests or specifications. Actually this is not the case, since the first occurrence of yielding is influenced by residual stresses existing in the beam before application of the load. Three types of residual stresses, shown in Figure 3, may be present in a composite T beam; namely, those caused by 1) nonuniform cooling of the hot rolled I-beam section, 2) welding the shear connectors, and 3) shrinkage of the concrete slab.

The residual stresses due to rolling are present in all commercially rolled I beams. They are distributed nonuniformly throughout the section approximately in a manner similar to that shown in Figure 3(a); as a rule, the maximum compressive stresses are located at the middepth of the web and the maximum tensile stresses are located at the junctions

posite T beams. During welding of shear connectors, the upper flange of the I beam is heated above the annealing temperature at the locations of the welds. At first, the heating of the upper flange results in an elongation of the upper flange and an upward deflection of the I beam; however, since the temperature of the heated spots exceeds the annealing temperature, this elongation, as well as the accompanying upward deflection, are relieved during the welding operation. The subsequent cooling of the metal causes a contraction of the upper flange and a downward deflection of the I beam. Thus, the welding of shear connectors sets up residual stresses distributed approximately as shown in Figure 3(b). The stress in the bottom flange is tension. The magnitude of these stresses depends on the number of the shear connectors, the size and length of

the welds, the size of the I beam, and the welding procedure. Ordinarily these stresses will be small; in the tests mentioned above they were on the order of 1,000 psi.

Finally, the concrete slab shrinks and exerts pressure on the shear connectors. As the steel I beam resists these forces, compressive stresses are set up in the top flange and tensile stresses in the bottom flange, as in Figure 3(c). The magnitude of the residual stresses due to shrinkage depends on the magnitude of the unit shrinkage of the concrete, on the amount of relief afforded by creep of the concrete, and on the size of the beam and the slab; in the Illinois tests tensions of up to 3,000 psi. were measured in the bottom flange of the I beam.

It can be seen from this discussion that all three types of residual stresses set up tension in the bottom flnage. The magnitude of the total residual stress may be significantly large, but its exact value is uncertain. For this reason it is virtually impossible to predict the occurrence of first yielding with any reasonable degree of accuracy. Fortunately, as will be shown later, the load-deformation characteristics of a composite T beam during the initial phases of the second stage of behavior are such that the uncertainty regarding the exact load at which first yielding occurs may not be too important from a practical point of view.

During the second stage of behavior, the steel beam continues to yield at the section of maximum moment, and the yielding spreads upward into the web toward the top flange and along the bottom flange toward both supports. Inelastic action is not confined to the steel alone; at some load during the second stage, the stresses in the concrete also enter the plastic range. For sections at which inelastic action has occurred, the stress distribution is nonlinear, and the shape of the stress-distribution diagram changes with load. Consequently, the location of the netural axis also changes, usually traveling upward as the load increases. Once the steel beam has begun to yield, the deformations increase at an increasing rate (Fig. 2). At first the rate of the increase of deformations is not much different from that observed during the first stage, but as the zone of yielding approaches the upper flange, a small increase of load is accompanied by very large increases in deformation. The load can be increased until the concrete of the slab fails by crushing. The load at crushing of the concrete, called in this paper the ultimate load, is substantially in excess of the load at first yielding and is reached only after deformations several times in excess of the elastic deformations take place. However, it is important to note in Figure 2 that a considerable portion of this reserve capacity beyond first yielding may be utilized while the plastic deformations are still only slightly in excess of the elastic deformations. It is for this reason that the uncertainty regarding the exact load at first yielding is not too significant.

The end of the second stage of behavior of a composite T beam is reached when the con-

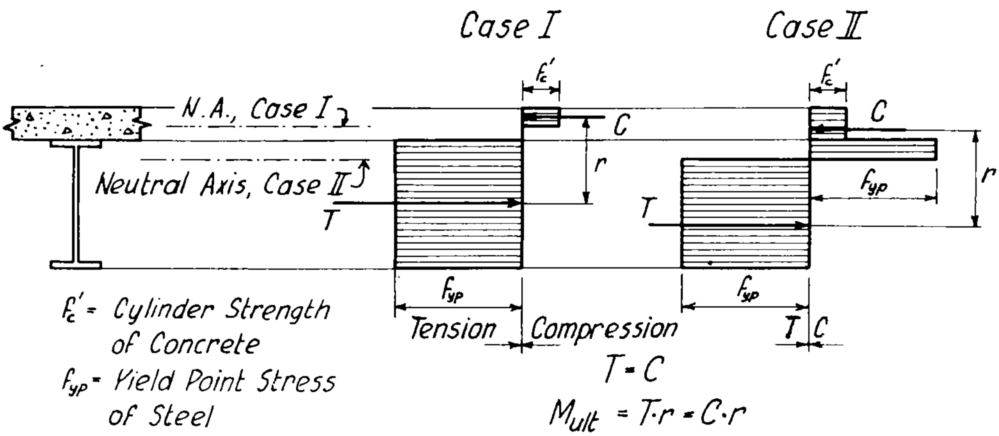


Figure 4. Idealized distribution of stress in composite T beams at ultimate load.

crete of the slab crushes. At this load the neutral axis of the composite section may be located either in the slab or in the I beam, as illustrated in Figure 4. The moment capacity at this load, called the ultimate moment capacity, may be evaluated with sufficient accuracy from the assumptions shown in Figure 4. When the neutral axis is loacted in the slab, Case I, it is sufficiently accurate to assume that the tensile stress in the steel beam is uniformly distributed and equal to the yield point stress of the steel, that the compressive stress in the slab is also uniformly distributed and equal to the strength of concrete determined by tests of 6- by 12-in. cylinders, that the concrete is not capable of carrying tension, and that the reinforcement of the slab does not contribute to the load-carrying capacity of the T beam. When the neutral axis is located in the I beam, Case II, it is sufficiently accurate to assume that both the tensile and the compressive stresses in the steel beam are uniformly distributed and equal to the yield point stress of the steel, that the compressive stress in the slab is distributed uniformly and equal to the cylinder strength of the concrete, and that the reinforcement of the slab does not contribute to the load-carrying capacity of the T beam. With these assumptions, the position of the neutral axis may be found from the equilibrium of horizontal forces, and the ultimate moment capacity may be found from the equilibrium of moments, as shown in Figure 4. Unlike the load at first yielding, the ultimate moment capacity, and therefore also the ultimate load are unaffected by the residual stresses.

For the particular composite beam shown in Figure 2, the ultimate load is 75 percent greater than the load at first yielding; that is, the reserve load capacity beyond first yielding is 43 percent of ultimate load. The similar reserve capacity for the noncomposite counterpart is only 14 percent. Furthermore, the ultimate load on the composite beam is reached only after deflections several times the maximum elastic deflections have occurred; in a noncomposite beam the ultimate load is reached at substantially smaller deflections. Although these differences in the behavior of ultimate load of a composite and a noncomposite beam will be smaller for beams with a relatively thin slab, it appears that the composite T beam is a considerably tougher structure than its noncomposite counterpart.

The third stage of behavior of a composite T beam begins when the slab crushes. After the slab has crushed, the load-carrying capacity of the T beam is decreased considerably, and from this point on the beam behaves in a manner similar to that of the same I beam without the slab.

Effective Slab Width

In computing the section properties of a composite T beam, it is usually assumed that the slab is fully effective in carrying the compressive stresses. This assumption is correct as long as the stresses are distributed uniformly throughout the full width of the slab. The question of the stress distribution in the slab of a composite T beam has been studied experimentally (*8*, *9*, *10*). In these tests the width of the slab of one specimen was approximately 11 times the slab thickness and one third of the span length, in two other specimens it was approximately 12 times the slab thickness and one sixth of the span length. In all three specimens the stress was distributed approximately uniformly over the full width of the slab.

Effect of Shoring the Beam

A composite T beam may be built in either of the two following ways: (1) the I beam is placed on the piers, forms for the slab are suspended from the I-beam, and the slab is cast and (2) the I beam is placed on the piers and is supported at intermediate points by shoring, forms are built for the slab, the slab is cast, and the shoring is removed only after the slab has acquired the prescribed strength. In the first case, that is, in a T beam built without shoring, the weight of both the beam and the slab is carried by the I beam alone; only the live loads and that portion of the dead load placed on the T beam after the slab has hardened are carried by the composite section. In the second case, i.e., in a T beam built with shoring, the entire dead load as well as the live load is carried by the composite section. Thus it may be expected that the behavior of these two types of composite T beams will differ. The AASHO Specifications for Highway Bridges (*13*) recognize this difference and for shored beams allow the assign-

ing of both the dead and live load to the composite section.

The relative behavior of shored and unshored composite T beams is illustrated in Figure 5. The three curves included in this figure represent the load-strain curves (1) for Beam 1 built with shores and shoring considered in the design, (2) for Beam 2 designed and built without shoring, and (3) for Beam 3 designed without shoring but built with shoring. Thus the dimensions of Beams 2 and 3 are identical, whereas Beam 1 is of smaller size. All data included in Figure 5 were computed for materials of similar properties without regard to the creep of concrete.

It can be seen from Figure 5 that the governing strain or stress at the design live load is equal for Beams 1 and 2, in spite of the smaller dimensions of Beam 1, and smaller for Beam 3 even though its dimensions are the same as those of Beam 2. Thus, at the design live load, the shoring seems to have a beneficial effect on the behavior of a composite T beam. However, the situation is different at first yielding and at the ultimate load. Of the three beams, Beam 1 yields at the lowest live load, Beam 2 at a substantially higher live load, and Beam 3 at a slightly higher live load than Beam 2. A comparison of live loads corresponding to the yield point strain for Beams 2 and 3 shows that some beneficial effect of shoring may be observed at first yielding, although percentage-wise this effect is much smaller than at the design stress. And finally, the ultimate load, not shown in Figure 5, is equal for Beams 2 and 3 but substantially smaller for Beam 1. Thus, shoring has no effect on the ultimate load.

The magnitude of the effects of shoring increases with increasing ratio of dead to live load. Since the dead-load-to-live-load ratio is larger for long spans than for short spans, the magnitude of the effects of shoring will generally increase with increasing span length. At the design load, the dead-to-live-load ratio may be as much as 1.0 or more; at first yielding, however, this ratio will be only a fraction of this value. Consequently, the effect of shoring at the yield load will be much smaller than at the design load. If then one beam is designed with shoring and another without shoring according to the provisions of the AASHO Specifications for Highway Bridges (*13*), the factor of safety against first yielding will be larger for the unshored than for the shored beam. The same is true for the factors of safety against ultimate failure, except that the difference between the factors of safety for the shored and the unshored beams is greater at ultimate than at yield loads.

The curves in Figure 5 were computed without consideration of the creep of concrete. In a composite beam built without shoring, the creep presents no problem, since all or most of the sustained load, the dead load, is carried by the steel beam. The slab assists only in carrying the live load which, in a highway bridge, is usually of short duration.

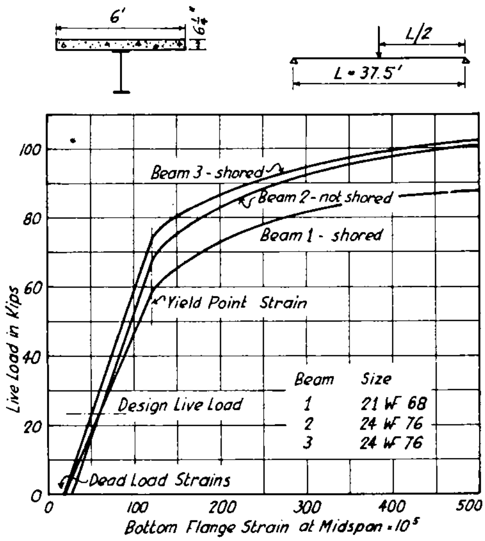


Figure 5. Effect of shoring on strains in composite T beams.

Thus, in such a beam the slab is not subjected to a permanent stress which will cause creep of the concrete. On the other hand, in a composite T beam built with shores, the slab is subjected to a permanent compression caused by the dead load. This permanent stress will result in creep, or more correctly, in relaxation of the concrete, which is followed by a redistribution of stress in the T beam. The change in the distribution of stress is shown schematically in Figure 6. It can be seen that the stress in the slab decreases, the neutral axis of the composite section moves downward toward the centroidal axis of the I beam, and the governing stress in the bottom flange of the I beam increases. In

other words, creep results in a decrease of the contribution of the slab to the resisting capacity of the section and, consequently, decreases the beneficial effect of shoring on the governing stresses. The decrease of the slab stresses due to creep is approximately proportional to the magnitude of the initial slab stresses. Consequently, the effects of creep will be relatively large in composite T beams with high initial permanent stresses in the slab.

It may be stated in summary that, although shoring has a favorable effect on the behavior of a composite T-beam, the effects are fairly large only at the design load; they are small at the yield load, and the ultimate load is not affected at all. Furthermore, the beneficial effect of shoring is partly offset by creep of the concrete. If, then, a design is based on working load conditions, the factors of safety both against first yielding and against ultimate

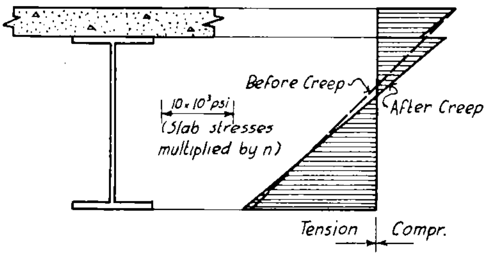


Figure 6. Effect of creep on distribution of stress

failure are smaller for the shored than for the unshored beams. If, on the other hand, shoring is used but its effects are disregarded in the design, the factor of safety against first yielding is slightly greater for the shored beam, while the factor of safety against ultimate failure is the same for both the unshored and shored beams.

Criteria for Design

It has been shown that a composite T beam is a tougher structure than a noncomposite T beam. However, the full benefit of composite action is realized only if the interaction is complete at all stages of loading, up to the ultimate load. Since the degree of interaction depends on the shear connection, the shear connection of a composite T beam should be made strong enough to transfer practically all of the horizontal shear from the slab to the I beam at all stages of loading up to the ultimate. How this can be done will be discussed in the next section.

A composite T beam with complete interaction may be designed as a homogeneous beam. In computing the moment of inertia of its cross section, the transformed area of the slab should be considered, that is the area should be divided by the modular ratio n. An example of the design of a composite T beam may be found in a paper by Newmark and Siess (*14*).

The choice of the effective width of the slab cooperating with the steel I beam depends on the spacing of the I beams in the bridge, the length of the span of the I beams, and the thickness of the slab. Although only a few tests are available on this subject, it seems that the provision of the AASHO Specifications for Highway Bridges (*13*) for the effective width of the slab of composite beams may reasonably be expected to apply. This provision requires that the effective width shall not be greater than: 1) the spacing of the I beams, 2) one fourth of the span length; or 3) twelve times the minimum thickness of the slab.

Composite beams built without shoring should be designed as noncomposite for dead load and as composite for live load. Composite beams built with shoring should ordinarily be designed in the same manner. In exceptional cases when the effect of shoring is large, both the dead and the live loads may be assigned to the composite section; however, it is important in such case that the design be made not for working stresses but for stresses at first yielding and that the effect of creep be taken into account. The creep may be accounted for by taking a modular ratio n equal to 20 to 30 in the computation of stresses due to dead load.

SHEAR CONNECTION

Purpose and Types

The concrete slab and the steel I beam of a composite T beam are interconnected by means of a shear connection. The purpose of the shear connection is twofold: 1) to prevent relative movement between the beam and the slab and 2) to transfer horizontal shear from the slab to the beam. The relative movement between the slab and the I beam may be either horizontal or vertical. Therefore

the shear connection should be capable of anchoring the slab in both directions.

The shear connection may be continuous or intermittent; an intermittent connection ties the slab to the beam at several locations spaced at regular or variable intervals. A continuous shear connection strong enough to transfer all of the horizontal shear and to prevent vertical separation of the slab from the beam at all loads up to the ultimate would be an ideal connection. However, according to existing knowledge, such an ideal connection cannot be built. On the other hand, as long as certain rules are observed, an intermittent connection is practicable which provides such a high degree of interaction that, for all practical purposes, it may be considered as perfectly rigid. Beams with such a shear connection may then be considered as composite beams with complete interaction.

The only kind of the continuous type of shear connection known at present is that provided by bond between the slab and the I beam. This type of connection is called, in this paper, a bond connection. An intermittent shear connection is provided by individual shear connectors and is called a mechanical shear connection. The functioning, advantages, and disadvantages of the mechanical connection and of the bond connection are discussed in the following sections.

Mechanical Shear Connection

A mechanical shear connection is composed of individual shear connectors attached to the top flange of the steel I beam at constant or variable spacing. It has been pointed out in the previous paragraphs that such a connection cannot provide absolutely perfect interaction between the slab and the beam but that it is a simple matter to design the shear connectors so strong that the degree of interaction is practically complete at all stages of loading up to and including the ultimate load.

The individual shear connectors are usually short pieces of steel, welded on the top surface of the steel I beam and embedded in the concrete of the slab. The shape of the connectors may easily be chosen such that the connectors provide mechanical anchorage in the horizontal as well as in the vertical direction. If, then, the individual connectors are spaced close together, the independent vertical movements of the slab are also substantially prevented.

The numerous shear connectors proposed and used may be grouped into three types according to their behavior when loaded: 1) stiff connectors, 2) flexible connectors, and 3) bond connectors. Although the behavior and design of the three types of connectors differs, this difference will affect only the design of the individual shear connectors, but not the design of the shear connection as a whole or of the composite T beam. The design criteria for the shear connection should

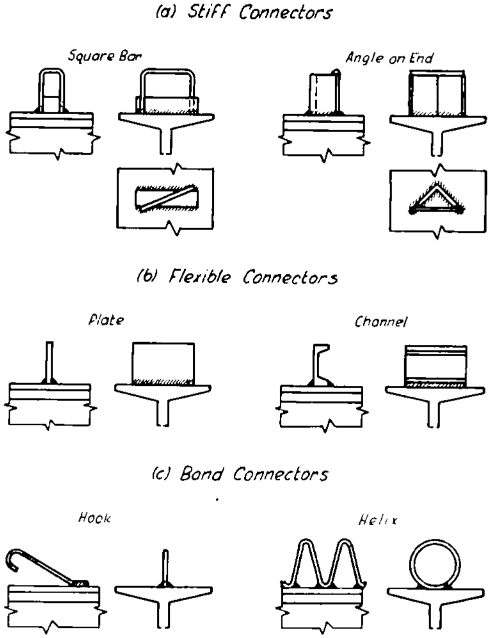


Figure 7. Types of mechanical shear connectors.

be the same, irrespective of the type of connectors used. Unfortunately, as a result of an insufficient knowledge of the behavior of the individual shear connectors, the various existing types are not designed on a common basis, and the resulting T beams do not have the same factors of safety against the failure of their shear connection. It is extremely difficult, therefore, to make an objective comparison of the efficiencies of the various types of connectors.

Two representative connectors of each type are shown in Figure 7. The stiff connectors are represented by a square steel bar with a

loop made of a round steel bar and by a rolled-steel angle placed on end with a steel bar welded across the upper end, the steel bars being provided for the purpose of vertical anchorage. The first of these two connectors has been used in Germany, and its behavior was studied by Graf (*3*). The second connector has been used in this country, and a few small-scale laboratory experiments with this type of connector (but without the bar for vertical anchorage) were carried out at the University of Illinois (*8*).

The flexible connectors are represented in Figure 7 by a steel plate and by a rolled-steel channel placed on one flange. Both of these connectors were tested at the University of Illinois; these tests included a few small-scale tests of the plate connectors (*8*) and extensive investigations of the behavior of the channel

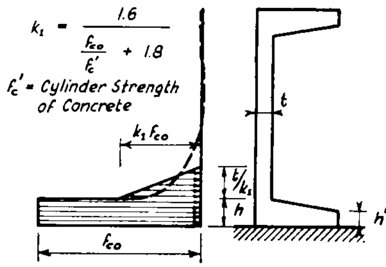


Figure 8. Flexible channel connector, pressure distribution.

connector (*8*, *9*). The channel connector is used extensively in this country.

The bond connectors are represented by a steel hook formed from a steel bar, and by a helix made from a round steel bar. The first type has been used in Europe and Australia and was tested by Roš (*1*) and Maier-Leibnitz (*2*). The helix was one of the earliest types of shear connectors developed and has been used extensively both in this country and in Europe. It has been investigated by Roš (*17*); however, these tests were limited in scope and the behavior of this type of connector is still not well understood.

Behavior of Mechanical Connectors

Flexible Connectors. A flexible shear connector is characterized by its low flexural stiffness. If loaded, flexural stresses of significant magnitude occur in the connector, the connector deflects, and the pressure exerted by the connector on the surrounding concrete is distributed nonuniformly. Since the connector is embedded in the concrete, its behavior is similar to that of a dowel embedded in an elastic medium and having one end fixed against rotation and the other floating freely in the surrounding medium. However, if various relative dimensions of the connector do not exceed certain limits, the height of the connector does not affect its behavior, and the behavior of the connector is similar to that of a long dowel fixed at one end and embedded in an elastic medium. Accordingly, the pressure on the concrete is highest at the fixed end, decreases rapidly with the distance from this end, and at some distance changes direction; in other words, it forms a wave of pressure alternating in direction and diminishing in magnitude. The stresses in the connector form a similar wave and are highest also at the fixed end. Furthermore, several types of flexible connectors, such as the channel connector, have a relatively stiff, nondeflecting part at the fixed end. If this is the case, the distribution of the pressure on the concrete is modified in that the maximum pressure at the fixed end is approximately constant throughout the stiff part of the connector and begins to decrease sharply at the junction of the stiff and flexible parts; the maximum stress in the connector occurs at this junction.

A good example of the flexible connector with a stiff flange and a flexible web is the channel shear connector shown in cross section in Figure 8. This type of connector has been the subject of extensive investigations at the University of Illinois (*8*, *9*), and its behavior and design have been discussed in a separate paper (*18*). The behavior of other types of flexible shear connectors is similar to that of a channel connector; however, the supporting experimental evidence for various other flexible connectors is insufficient to permit formulation of any definite criteria for design.

A flexible-channel shear connector is a short piece of rolled-steel channel with one flange welded to the top flange of the I beam. The welded flange is stiff and represents the stiff part of the connector, whereas the web is the flexible part. Connectors of practical sizes made of channels rolled in this country have such relative dimensions that the upper flange of the channel does not influence the

behavior of the connector. The actual distribution of the pressure exerted by this type of connector on the surrounding concrete is shown in Figure 8 as a dashed line. As described above, this bearing pressure is uniform adjacent to the stiff flange and nonuniform adjacent to the web. The maximum bearing pressure occurs adjacent to the stiff flange, while the maximum steel stress in the channel occurs at the junction of the stiff flange with the web, that is at the height h from the welded end (Fig. 8).

Since the purpose of the shear connector is to transfer horizontal shear without permitting any substantial movement between the slab and the I beam, the deflections of the connector, when loaded, must be small. It can be shown that this condition will be satisfied as long as the stresses in the channel do not exceed the yield point stress of the steel. For channels of usual dimensions and concretes of cylinder strength greater than about 2,000 psi., the pressure on the concrete is not critical although it may be quite large, even exceeding the cylinder strength of the concrete. The reason for this phenomenon lies in the state of stress in the concrete adjacent to the channel. Only a small portion of the slab area is subject to high stress; therefore the surrounding concrete offers a considerable restraint to the highly stressed portion and a triaxial state of stress is present. It is known that under such conditions concrete is able to withstand stresses several times greater than its unconfined compressive strength as determined from tests of control cylinders.

The critical stress in a flexible-channel shear connector is the maximum stress in the channel, which can be determined from semi-empirical design formulas presented elsewhere (*18*) or from the idealized pressure distribution diagram shown in Figure 8 as full line. In other flexible connectors having ratios of the height of the stiff part to the thickness of the flexible web similar to those encountered in channel connectors, the design will undoubtedly be governed also by the steel stress. The steel stress might govern also the design of flexible connectors having no stiff part. If, however, the flexible connector has a very high stiff portion, the pressure on the concrete may become the governing factor, as in a stiff connector.

Stiff Connectors. As the name of this type of mechanical shear connector indicates, the stiffness of a stiff connector is large; when loaded, the bending stresses in a stiff connector are insignificant, and the distribution of the pressure exerted by the connector on the surrounding concrete is approximately uniform. However, this uniform distribution exists only if the connector or any part of it does not deflect. In some connectors, a portion of the connector may deflect locally, although the connector as a whole does not deflect; for example, in an angle welded on one end (Fig. 7) the relatively thin wings may be locally distorted. Other connectors may be quite stiff, if short, but if their height is large in comparison with the dimensions of the cross section, bending and, therefore, a nonuniform distribution of pressure may result.

The stresses in a stiff shear connector are low; thus the governing factor is the magnitude of the pressure exerted on the concrete. If the design of the shear connection is based on the requirement that practically full composite action be retained up to the ultimate flexural capacity of the composite T beam, a pressure corresponding to a certain limiting deformation of the surrounding concrete might be the criterion for the design of a stiff connector. The upper limit of this governing pressure is that pressure which causes crushing of the concrete. The question of whether crushing of the concrete or some limiting deformation of the concrete should govern the design of stiff connectors has not been answered by the investigations of this type of connector. However, the work of Roš (*1*) and of Graf (*4*) suggests that an allowable working stress of about two thirds of the compressive strength of concrete cylinders is a satisfactory value for the design of stiff connectors.

Most of the stiff connectors have such shape that they do not provide mechanical anchorage in the vertical direction. For this reason hooks or loops welded to the connector itself are provided for the purpose of vertical anchorage. According to Roš (*1*) and Graf (*4*) these hooks have little or no effect on the behavior of the connector at working loads, but increase the ultimate capacity of the connector. Both Roš and Graf propose that in the design of stiff connectors with additional vertical anchorages a certain portion of the total design load be assigned to these vertical anchorages.

Bond Connectors. This category includes connectors made of steel bars, usually round or square, formed either into individual hooks or loops, or into a continuous helix, or so-called spiral. Connectors of this type can transfer the horizontal shear by: 1) bearing through the stiff part of the bar at locations adjacent to the weld attaching the bar to the beam, 2) bearing of the flexible bar at sections away from the welds (at these locations the pressure is distributed nonuniformly), and 3) bond between the bar and the concrete.

The limiting factor for the stiff part is the pressure on the concrete which causes either crushing of the concrete or an excessive slip; the limiting factor for the flexible portion is probably yielding of the steel at the point of maximum stress; and the limiting value for the bond is either the bond strength or yielding of the steel in direct tension. For the connector, as a whole, the strength of the welds is an additional factor. Which of these factors, or combination of them, should govern the design of bond connectors has not been answered by the investigations of this type of connector (*1, 2, 17*).

Vertical Anchorage

It has been mentioned that the shear connection should prevent vertical as well as horizontal movement between the I beam and the slab. The purpose of this requirement is to insure the monolithic action of the composite I beam and to insure a proper functioning of the shear connection. If the shear connection is furnished totally or partly by bond on horizontal surfaces, it is obvious that vertical separation of the slab from the beam would destroy the bond and thus the shear connection. If mechanical shear connectors are used, their capacity may be greatly diminished if vertical separation can take place. Although the concrete under any shear connector is subjected to relatively high pressures even at working loads, these high pressures have no detrimental effect, because of the restraint offered to the highly stressed concrete by the surrounding less-stressed concrete and by the flange of the I beam. However, in case of vertical separation of the slab from the beam, the restraint offered by the upper flange of the I beam would be removed, and excessive deformation or crushing of concrete might result. Furthermore, vertical separation would cause redistribution of the pressures adjacent to the connector and would thus change the stress conditions in the connector itself.

Since the strength of bond in tension is very small, the bond cannot be relied on for vertical anchorage and mechanical vertical anchorage is necessary. Some connectors, such as a flexible-channel connector or a helix, are of such shape that they provide the needed vertical as well as horizontal anchorage (Fig. 7). Others, however, provide only the horizontal anchorage. In such case, a special vertical anchorage must be provided.

To insure monolithic action of a composite T beam, it is important that vertical separation be prevented as nearly as possible throughout the length of the beam. Thus it is important that the vertical anchorages be spaced at small intervals.

Bond Connection

When a concrete slab is cast on the top of a steel I beam, bond is established between the concrete and the steel at the contact surfaces during the hardening of the concrete. If such a beam is subjected to a load smaller than that at which the bond would break, it acts as a composite structure, since the bond provides the shear connection necessary for the interaction between the beam and the slab. Both laboratory and field tests have shown that bond, if present, is an effective shear connection (*8, 9, 15, 16*).

However, bond has some weaknesses which make it an unreliable shear connection. In the first place, although it is very effective at low loads, its strength is probably not sufficient to maintain full interaction up to the ultimate load. If a composite beam is loaded beyond first yielding of the steel I beam, large inelastic deformations take place in the beam and the magnitude of the horizontal shear increases in the vicinity of the sections that have yielded. Although no experimental evidence known to the writers is available, it seems unlikely that bond would be sufficient to transmit the shear in such regions. After the bond has broken in one place, a progressive bond failure may follow; consequently the shear connection may be lost throughout the full length of the beam as a result of local bond failures in regions that have yielded. Furthermore, bond is very weak in tension so that its ability to prevent relative movement of the

slab in the vertical direction is very limited. Therefore, shrinkage and warping of the slab as well as dynamic loading may destroy the bond even at working loads. Failures of bond have been observed both in the laboratory and in field tests (*8*, *9*, *15*, *16*).

Whatever the cause of bond failure, it is important to realize that once the bond is broken it is broken forever. If reliance is placed solely on bond, and the bond breaks, the T beam changes from a composite structure to a noncomposite one, with a consequent decrease in load-carrying capacity and increase in deflection. Since some of the factors which may cause the failure of bond cannot be controlled by the designer, bond should be considered as an unreliable shear connection.

Combination of Mechanical Connectors and Bond

It has been explained that bond is an unreliable shear connection, because it does not have sufficient strength in tension to prevent vertical separation of the slab from the beam and because it is not probable that it would be able to resist large deformational stresses after the steel beam has yielded at some sections and probably cannot insure composite action up to the ultimate load. However, if bond were combined with good mechanical vertical anchorage the first objection to reliance on bond for shear connection would be removed. Furthermore, it is possible, although not very probable, that if bond is combined with mechanical shear connectors, the bond might exist up to the ultimate load. If this were the case, considerably lighter shear connectors could be used than is customary at present. However, there is no experimental evidence available which would support this supposition. Therefore, until and unless such evidence is available, the mechanical shear connectors should be designed for full static load, as will be explained in the section on the criteria for the design of a shear connection.

Repeated Loading

The shear connection of a composite bridge beam must withstand a large number of repetitions of the design loads as well as a few overloads. Thus both static and fatigue strength must be considered in the design. The static strength of a shear connection and of individual shear connectors has been discussed in the preceding paragraphs. The fatigue strength of a mechanical shear connection is discussed in this section.

It has been shown by tests that mechanical shear connectors, as well as a shear connection composed of such connectors, have no endurance limit within a reasonable number of repetitions of the design load (*1*, *8*). Thus, the connection must be designed in such a manner that it will withstand a certain number of repetitions of the design load before a fatigue failure will probably occur. The design procedures proposed by various investigators usually provide a sufficient margin of safety against a fatigue failure (*1*, *18*).

An additional factor of safety against fatigue failure is provided by bond. Although it is known that bond is present in most composite structures, at least for some time, it is not considered in the design. However, as long as bond is present, the shear connectors will be required to transfer only a negligible proportion of the shear; it may be expected that as long as bond is present, the shear connectors will not fail in fatigue. This hypothesis was substantiated by the results of tests made at the University of Illinois (*8*). In these tests all specimens were constructed with flexible-channel shear connectors, and in most of the specimens the bond between the steel beam and the concrete slab was deliberately broken before the tests were begun so that all of the horizontal shear would be carried by the shear connectors. However, in a few specimens the bond was not broken before the fatigue test was begun but failed during the course of the tests, and in two specimens the bond remained unbroken throughout the tests. The specimens with initial bond withstood more repetitions of load than the corresponding specimens without bond, and the two specimens in which the bond did not break at all withstood over 2 million repetitions of load without any signs of damage. Since it is probable that bond exists in actual structures at least for some portion of the life of the structure, those design requirements which are based on consideration of fatigue may not be entirely applicable to shear connectors.

Criteria for Design

A composite T beam should be designed with mechanical shear connectors to provide

for the interaction between the beam and the slab. The shear connection should be strong enough to retain practically full interaction at all stages of loading, up to the ultimate flexural capacity of the composite T beam. In order to achieve this goal, the shear connection should be capable of transferring all of the horizontal shear resulting from the ultimate load without permitting any appreciable slip to take place between the slab and the beam.

The capacity of an individual shear connector is governed either by the maximum stress in the connector or by the maximum pressure on the concrete, depending on the type of connector. Some of the flexible connectors, such as channel connectors, are governed by the maximum steel stress. For such connectors the maximum steel stress at ultimate load should not exceed the yield point stress for steel.

The load on an individual connector is equal to the product of the spacing of the connectors and the horizontal shear at the contact surfaces of the beam and the slab. The capacity of the individual connector should be equal to the load acting on the connector when the T beam is loaded for maximum shear with that load which causes the beam to fail in flexure (the load corresponding to the ultimate flexural capacity of the beam).

A design procedure based on the criteria outlined above is an ultimate design procedure and as such requires a knowledge of the ultimate capacity of the T beam. Since the present design procedures for I-beam bridges are based on working load rather than on ultimate load conditions, it may be desirable to design shear connectors also for working load. This may be done by the use of certain conversion factors as has been illustrated in a paper on the design of channel shear connectors (*18*) or by the use of allowable stresses with sufficiently large factors of safety as was proposed for the design of stiff connectors by Roš (*1*) and Graf (*4*).

COMPOSITE I-BEAM BRIDGES

Simple-Span Bridges

The distribution of load to the girders in an I-beam bridge depends on the relative stiffness of the I beams to the slab (*11*). If the stiffness of the I beams is high relatively to that of the slab, the transverse distribution of the load is small. In the elastic range of stresses the stiffness of a beam section may be expressed as the product of the moment of inertia and the modulus of elasticity; since the moment of inertia of a composite section is substantially higher than the moment of inertia of a noncomposite section of the same dimensions and since the stiffness of the slab is approximately the same for both structures, the relative stiffness is substantially higher for the composite structure. Thus, in the composite bridge there will be less transverse distribution of load than in a similar noncomposite bridge, as long as the stresses are elastic.

It has been shown in the discussion of composite T beams that their reserve capacity beyond first yielding is substantially greater than that of their noncomposite counterparts. Accordingly, the composite action results in an increase of the reserve capacity of an I-beam bridge. Furthermore, the reserve capacity of the bridge beyond first yielding is influenced also by the transverse redistribution of the load; if the redistribution in one bridge is such that all T beams are stressed to their full capacity, whereas in another similar bridge the failure occurs after only a few T beams have reached their full capacity, the reserve capacity is higher in the first bridge than in the second. Laboratory tests of model bridges made at the University of Illinois have demonstrated that in a noncomposite structure the I beams may fail by torsional buckling, since the torsional restraint offered to the top flange of the I beams is only that offered by friction between the slab and the beam. Thus, this type of bridge may fail long before a substantial part of the load has been redistributed to the remaining beams. In a composite structure, however, the shear connectors offer a substantial torsional restraint to the I beams; as a result, a premature torsional failure is not probable, and a larger portion of the ultimate capacity of all T beams is developed before the ultimate capacity of the bridge has been reached.

The greater reserve capacity and greater toughness of the composite I-beam bridge may be considered to offset the unfavorable effect of the greater beam stiffness on the wheel-load distribution of elastic conditions. Although

theoretically the maximum beam moments at working loads should be about 8 percent greater for a composite I-beam bridge than for a similar noncomposite structure (*10*), the AASHO Specifications for Highway Bridges permit the same wheel-load distribution to be used in both cases. This may be considered as tacit recognition of the greater toughness and reserve strength of the composite bridge.

Continuous Bridges

In composite, simple-span I-beam bridges the concrete slab is in compression. In a continuous bridge, however, some portions of the bridge are subjected to negative moments and the slab in these regions is in tension. Since concrete is inherently strong in compression but weak in tension, it is important to know how this difference in the properties of concrete influences the behavior of a continuous bridge in the regions of negative moments. Furthermore, the question arises as to whether the behavior in the positive moment region of a continuous bridge is the same as the behavior of a simple-span bridge and how the distribution of moments in the negative moment regions compares with the distribution of moments in the positive moment regions.

A continuous, composite I-beam bridge may be built with shear connectors provided either throughout the full length of the bridge or only in the positive moment region. If shear connectors are provided throughout the full length of the bridge, interaction between the slab and the I beam will be present in the regions of both positive and negative moments. However, in the negative-moment regions, the slab of a composite bridge is stressed in tension, and since the concrete can resist only tensions of magnitudes considerably smaller than those which are caused by the working loads, the slab will crack and the reinforcement of the slab will be the only element acting compositely with the I beams in the regions of negative moments.

If the shear connectors are provided only in the positive-moment regions, there is no composite action between the slab and the beams in the regions of negative moment. Nevertheless, the slab still offers some degree of restraint to the I beams, since it is anchored to the beams in the regions of positive moment. In this case, as in a bridge with shear connectors in the negative-moment regions, the slab is in tension; thus, the concrete cracks and only the slab reinforcement contributes resisting capacity to the negative moments. The stresses in the reinforcement at the section of critical negative moment are smaller than in a similar bridge with shear connectors throughout; thus, the assistance offered by the slab in resisting the critical negative moment will be smaller if the shear connectors are provided only in the positive-moment regions. However, since the area of the slab reinforcement is usually quite small in comparison with the area of the I beams, the resisting capacity of both types of composite bridges to negative moments will be only a few percent higher than that of the I beams alone. Thus from the standpoint of composite action it makes little difference whether the shear connectors are provided or left out in the negative-moment regions.

A possible reason for providing shear connectors throughout the entire length of the bridge might be for the purpose of vertical tie-down of the slab. However, it should be noted that the lengths of the negative-moment regions are relatively short, and furthermore, in these regions the curvature of the deflected structure is the reverse of that at midspan, so the slab tends to exert pressure on the I beams instead of pulling away when loaded.

The slab of a composite bridge is restrained against shrinkage deformations, and the danger of cracking due to shrinkage is present. If such cracking occurs in the positive-moment regions, the cracks close under the action of loads, since the slab is in compression. Thus cracking will affect the behavior of the bridge only at low loads. However, where the slab is in tension, the shrinkage cracks open under the action of loads and may be objectionable. Although such cracks will not influence the behavior of the bridge at working loads (the concrete of the slab does not contribute to the composite action), they may decrease the resistance of the slab to punching, as will be described later. However, it makes little difference whether shear connectors are present in the negative-moment regions or not, since the slab is restrained against shrinkage in both cases.

In the negative-moment regions of a composite, continuous I-beam bridge, the upper

stress shown in Figure 4. How many of the several beams in a bridge reach the fully plastic stage depends on the type and position of loading applied to the bridge. If a live load is applied across the full width of the bridge, the ultimate capacity corresponds to a fully plastic state of stress in all beams. However, if only a few concentrated loads are placed on the bridge, especially if the loads are placed eccentrically with respect to the longitudinal axis of symmetry of the bridge, some beams will reach the fully plastic state of stress, while others may not even have yielded when the failure occurs. In continuous bridges the situation at ultimate load is more complicated, because in such structures the ultimate capacity is reached only after yielding occurs at two or more sections. The phenomena of the ultimate capacity of I-beam bridges through general yielding are not well understood, and additional experimental and theoretical studies of this question are needed.

From the standpoint of design, the minimum ultimate capacity is important; that is, the lowest load at which one of the possible failures occurs. However, in order to be able to predict this minimum ultimate load, it is necessary to understand all types of possible failures. If such knowledge were attained, it would be possible to base the design of composite I-beam bridges on the ultimate instead of on the working load behavior. A design based on the ultimate capacity would have two advantages as compared to present methods based on the elastic behavior. First, the designer would have a clearer picture of the actual factor of safety of the structure. Second, it is probable that a design procedure based on ultimate capacity would be simpler than one based on the elastic analysis.

Criteria for Design

Composite I-beam bridges should be designed for complete interaction between the slab and the steel beams. This requires that the shear connections be strong enough to insure complete interaction up to the ultimate capacity of the bridge. How this goal may be achieved has been discussed in other parts of this paper.

Mechanical shear connectors are a necessity in the region of positive moment, i.e., throughout the full length of simple span bridges and between the points of contraflexure in continuous bridges. They may be omitted in the regions of negative moment in continuous bridges.

The portion of the wheel load assigned to one beam may be the same for a continuous bridge as for an equivalent simple-span bridge and the same for the negative-moment regions as for the positive-moment regions.

In designing a continuous, composite I-beam bridge, composite action should be considered in the regions of positive moment but may be neglected in the regions of negative moment. Actually, some composite action will be present also in the negative-moment regions, but since the reinforcement is the only component of the slab which contributes to the resisting capacity of the section in these regions, disregarding composite action in the region of negative moments will have no appreciable effect on the behavior of the resulting structure.

SUMMARY

This paper has been concerned with composite construction for I-beam highway bridges. It is based principally on numerous experimental and theoretical studies.

It has been shown that a composite bridge structure consisting of several steel I beams and a reinforced-concrete slab rigidly connected to the steel beams is generally superior to a similar structure having no connection between the beams and the slab. It has also been shown that, in order to secure all of the advantages of composite action between the beams and the slab, the shear connection should be strong enough to provide a practically complete interaction at all stages of loading, up to the ultimate load, and must be effective in preventing movements in both the horizontal and vertical directions.

The shear connection should be provided by mechanical shear connectors. Mechanical shear connectors provide a reliable anchorage, and although they always permit some relative movement between the beams and the slab, properly designed mechanical shear connectors insure practically complete interaction at all stages of loading. Natural bond between the slab and the I beams is also capable of transmitting horizontal shear but is an unreliable shear connection, since it is weak in tension and can easily be broken if the slab tends to

separate from the beams. Furthermore, it is questionable whether bond would remain unbroken if the beam were loaded up to its ultimate capacity.

Stiff, flexible, and bond types of mechanical connectors are used at present. The behavior of these three types of connectors differs, but all of them are capable of providing an adequate connection. Design procedures for several particular connectors have been proposed and may be found in the literature.

Since the bond is unreliable and may break during the life of the structure, it is recommended that mechanical shear connectors be designed for the full horizontal shear.

In the design of continuous, composite I-beam bridges with shear connectors, the designer may choose to place the shear connectors throughout the full length of the bridge or only in the regions of positive moments. It has been pointed out that the presence or absence of shear connectors in the regions of negative moments has no appreciable influence on the static behavior of a composite bridge, but that from the standpoint of the fatigue strength of the I beams, it may be undesirable to attach the shear connectors to the I beams in the negative-moment regions.

The elastic distribution of moments to the individual beams is approximately the same at the sections of maximum positive and negative moments of a continuous bridge; it is also approximately the same as in a simple-span bridge of equal cross section but with a span length equal to the distance between the points of contraflexure. Thus, the beams of a continuous, composite I-beam bridge may be designed for the same proportion of a wheel load as for a simple-span bridge.

In general, existing knowledge of the behavior of composite T beams, their shear connection, and of composite I-beam bridges is sufficient to permit safe and economical structures to be designed and built. Nevertheless, further advances in the design of structures of this type will require further studies. The number of types of shear connectors whose behavior is well known quantitatively is small and further research in this direction would give the designer a wider choice. And if it is desired to make it possible to predict the ultimate load-carrying capacity of the composite I-beam bridges, additional experimental and theoretical studies are needed.

ACKNOWLEDGMENTS

The major portion of the experimental and theoretical evidence upon which this paper has been based was gained through an investigation of slab-and-beam highway bridges conducted at the University of Illinois in coöperation with the Division of Highways of the State of Illinois and the Bureau of Public Roads of the U. S. Department of Commerce. All phases of this investigation were under the general direction of N. M. Newmark, research professor of structural engineering. Most of the studies of composite bridges and T beams and the studies of shear connections were planned and directed by C. P. Siess. The senior author of this paper conducted or interpreted most of the tests of composite T beams and of the flexible-channel shear connectors. However, considerable credit must go also to the many others who took part in the extensive work required by both the experimental and the theoretical studies.

REFERENCES

(1) M. Roš, "Träger in Verbund-Bauweise," *Report No. 149*, Swiss Federal Mat. Test Lab. (EMPA), Zurich, 1944.

(2) H. Maier-Leibnitz, "Versuche über das Zusammenwirken von I-Trägern mit Eisenbetondecken," *Bautechnik* (Berlin), Vol. 19, pp. 265–270 (1941).

(3) O. Graf, "Versuche über den Verschiebewiderstand von Dübeln für Verbundträger," *Bauingenieur* (Berlin), Vol. 25, pp. 297–303 (1950).

(4) O. Graf, "Über Versuche mit Verbundträgern," *Abhandlungen aus dem Stahlbau*, Deutschen Stahlbau—Verband (Köln), No. 10, pp. 74–90 (1951).

(5) F. G. Thomas and A. Short, "A Laboratory Investigation of Some Bridge-Deck Systems," *Proceedings*, Institution of Civil Engineers (London), Part I, Vol. 1, pp. 125–187 (1952).

(6) N. M. Newmark and C. P. Siess, "Studies of Slab and Beam Highway Bridges—Part I: Tests of Simple-Span Right I-Beam Bridges," *Bulletin No. 363*, University of Illinois Engineering Experiment Station, 1946.

(7) N. M. Newmark, C. P. Siess and W. M. Peckham, "Studies of Slab and Beam Highway Bridges—Part II: Tests of

Simple-Span Skew I-beam Bridges," *Bulletin No. 375*, Univ. of Ill. Eng. Exp. Sta., 1948.

(8) C. P. SIESS, I. M. VIEST AND N. M. NEWMARK, "Studies of Slab and Beam Highway Bridges—Part III: Small-Scale Tests of Shear Connectors and Composite T-Beams," *Bulletin No. 396*, Univ. of Ill. Eng. Exp. Sta., 1952.

(9) I. M. VIEST, C. P. SIESS, J. H. APPLETON AND N. M. NEWMARK, "Studies of Slab and Beam Highway Bridges—Part IV: Full-Scale Tests of Channel Shear Connectors and Composite T-Beams," *Bulletin No. 405*, Univ. of Ill. Eng. Exp. Sta., 1952.

(10) F. E. RICHART, N. M. NEWMARK AND C. P. SEISS, "Highway Bridge Floors," *Transactions*, American Society of Civil Engineers, Vol. 114, pp. 979–1072 (1949), (Also *Reprint No. 45*, Univ of Ill. Eng. Exp. Sta., 1949).

(11) C. P. SIESS AND A. S. VELETSOS, "Distribution of Loads to Girders in Slab and Girder Bridges: Theoretical Analyses and Their Relation to Field Tests," *Research Report No. 14B*, Highway Research Board, 1953, pp. 58–74.

(12) N. M. NEWMARK, C. P. SIESS AND I. M. VIEST, "Tests and Analysis of Composite Beams with Incomplete Interaction," *Proceedings*, Society for Experimental Stress Analysis, Vol. 9, No. 1, pp. 75–92 (1951).

(13) THE AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, "Standard Specifications for Highway Bridges." Washington, D. C., 1949.

(14) N. M. NEWMARK AND C. P. SIESS, "Design of Slab and Stringer Highway Bridges," *Public Roads*, Vol. 23, pp. 157–164 (1943).

(15) A. H. FULLER AND E. W. BLUMENSCHEIN, "Skunk River Bridge Exhibits Composite Action After Twenty-Eight Years of Service," *Civil Engineering*, Vol. 21, pp. 40–43 (1951).

(16) N. VAN EENAM, "Live-Load Stress Measurements on Fort Loudon Bridge, Final Report," PROCEEDINGS, Highway Research Board, 1952, pp. 36–57.

(17) M. Roš, "Les constructions acier-béton, système Alpha," *L'Ossature metallique* (Bruxelles), Vol. 3, pp. 195–208 (1934).

(18) I. M. VIEST AND C. P. SIESS, "Design of Channel Shear Connectors for Composite I-Beam Bridges," prepared for publication in *Public Roads*.