HIGHWAY RESEARCH BOARD
Bulletin 174

Studies in Composite Design of Steel Beam And Concrete Bridge Decks

National Academy of Sciences—
National Research Council
publication 534

The opinions and conclusions expressed in this publication are those of the authors and not necessarily those of the Highway Research Board

HIGHWAY RESEARCH BOARD
Bulletin 174

Studies in Composite Design of Steel Beam And Concrete Bridge Decks

PRESENTED AT THE
Thirty-Sixth Annual Meeting
January 7-11, 1957

1958

Washington, D. C.

Department of Design

T.E. Shelburne, Chairman
Director of Research, Virginia Department of Highways
University of Virginia, Charlottesville

COMMITTEE ON BRIDGES

G.S. Paxson, Chairman
Assistant State Highway Engineer
Oregon State Highway Commission, Salem

Raymond Archibald, 1411 4th Avenue Building, Seattle, Wash.
W.A. Back, Raymond Concrete Pile Company, New York, N.Y.
Russell E. Barnard, Armco Drainage and Metal Products, Inc., Middletown, Ohio
E.M. Cummings, Sales Engineer, Bethlehem Steel Corporation, Bethlehem, Pa.
E.L. Erickson, Chief, Bridge Division, Office of Engineering, Bureau of Public Roads
H. de R. Gibbons, The Union Metal Manufacturing Company, Canton, Ohio
T.B. Gunter, Jr., Assistant Chief Engineer-Bridges, North Carolina State Highway Commission, Raleigh
John J. Hogan, Consulting Structural Engineer, Portland Cement Association, New York, N.Y.
W.T. Lankford, Applied Research Laboratory, United States Steel Corporation, Monroeville, Pa.
M.N. Quade, Consulting Engineer, 51 Broadway, New York, N.Y.
William H. Rabe, First Assistant Engineer of Bridges, Ohio Department of Highways, Columbus
C.P. Siess, Department of Civil Engineering, University of Illinois, Urbana
J.A. Williams, Bridge Engineer, Missouri State Highway Department, Jefferson City

Contents

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

I. M. VIEST, Research Associate Professor of Theoretical and Applied Mechanics, University of Illinois,
R. S. FOUNTAIN, Consulting Engineer,* and
C. P. SIESS, Research Professor of Civil Engineering, University of Illinois

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 1/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.

*Formerly Senior Bridge Designer, State Highway Department of Georgia

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;

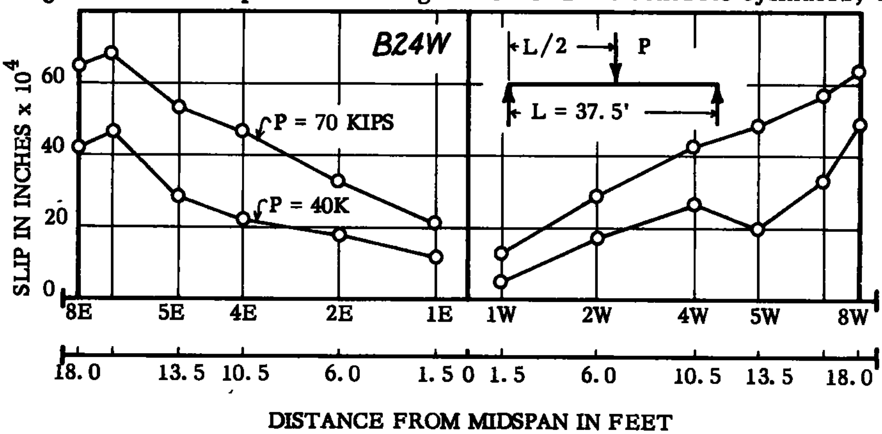


Figure 1. Slip distribution curves for a composite T-beam with mechanical shear connections.

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.

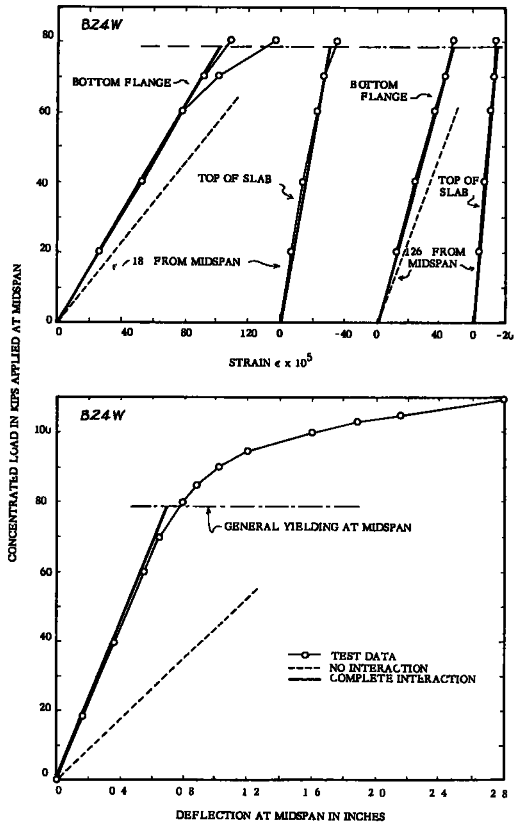


Figure 2. Load-strain and load-deflection curves for a composite T-beam with strong mechanical shear connectors.

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-

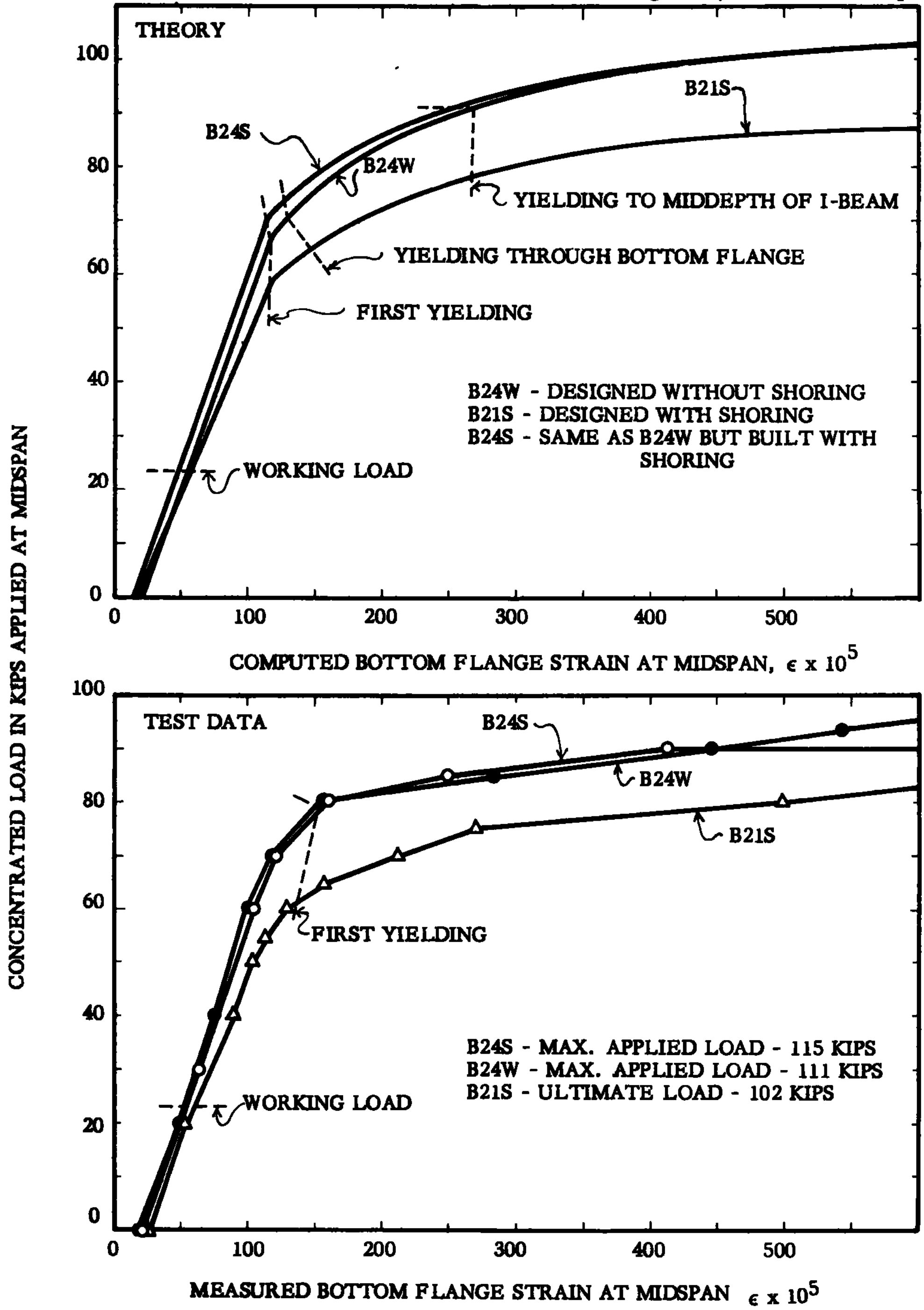


Figure 3. Effect of shoring on load-strain curves for composite T-beams.

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

$$S = \frac{Vm}{I} \qquad (1)$$

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.

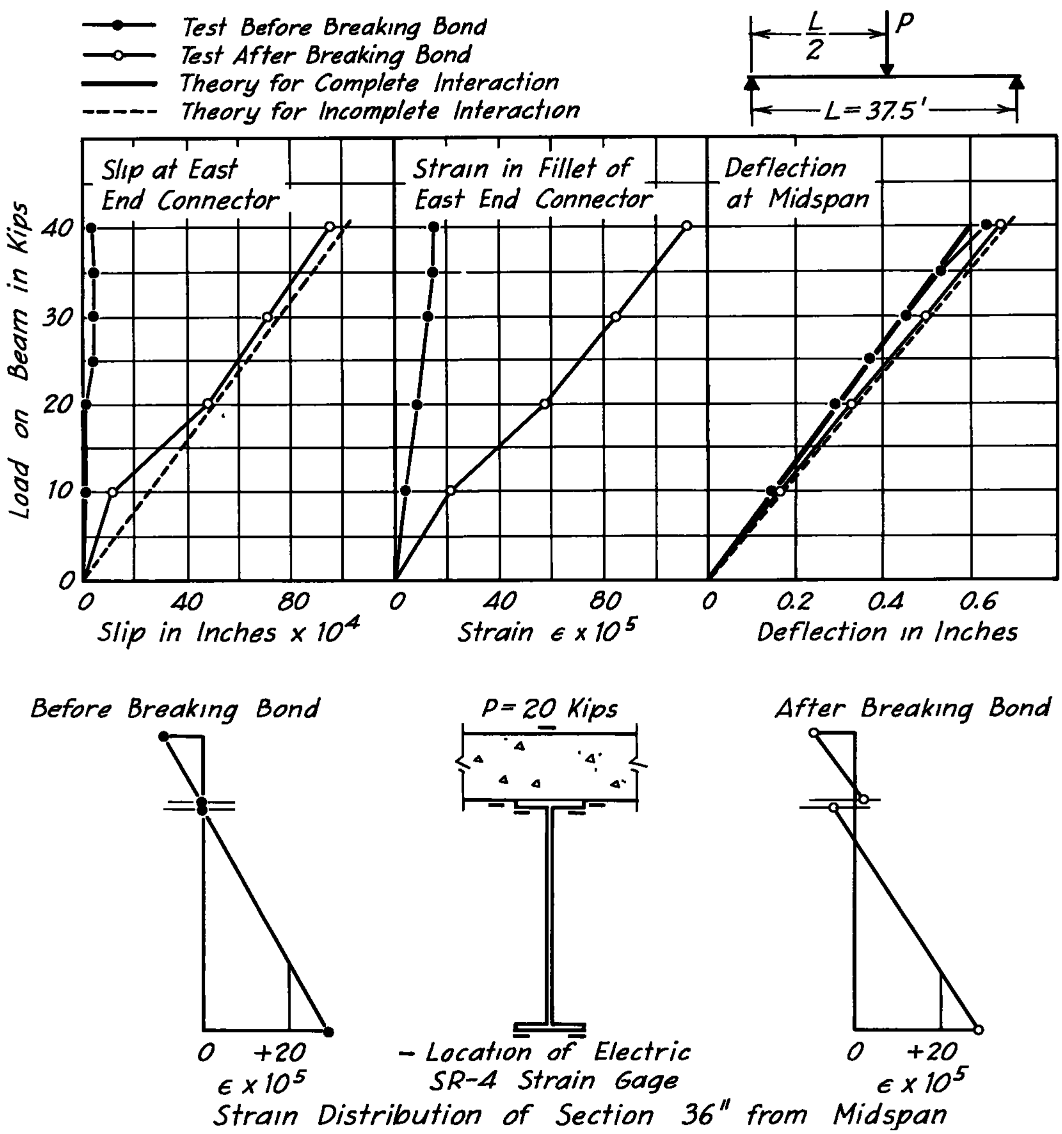


Figure 4. Effect of breaking bond in a beam with weak shear connectors.

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):

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

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.,

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

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

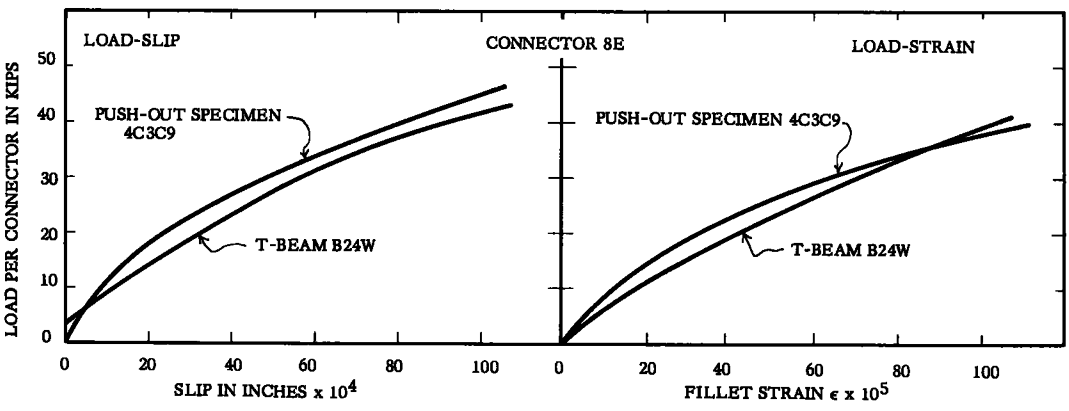


Figure 5. Characteristic curves for flexible channel connectors.

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

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

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)

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

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

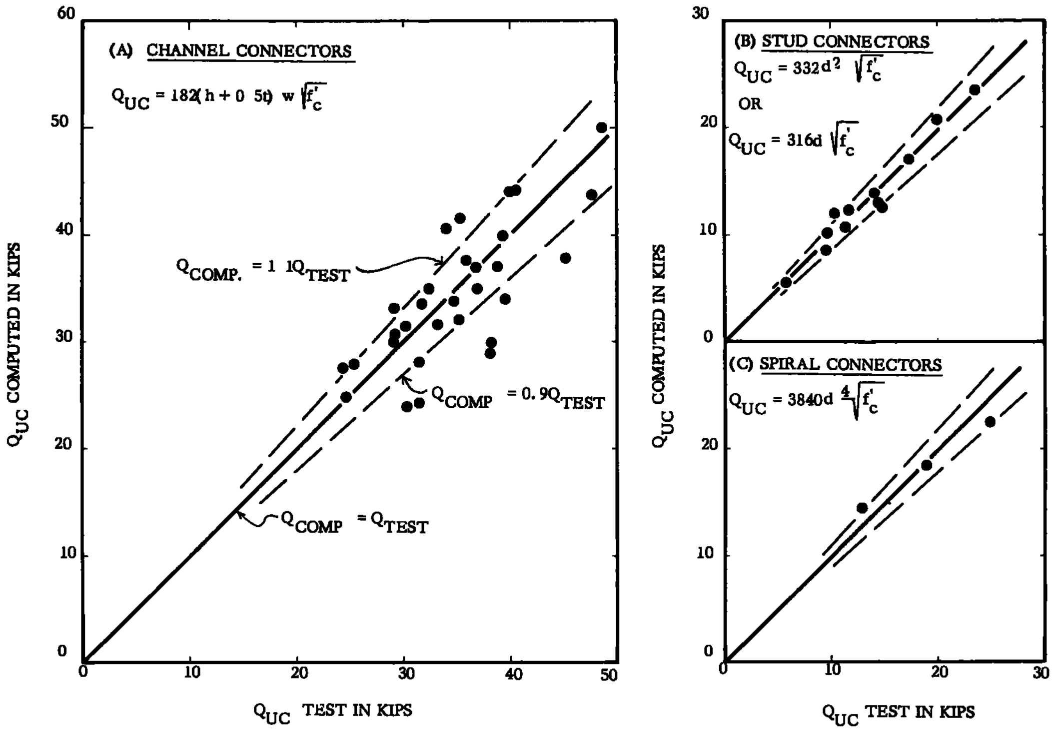


Figure 6. Comparison of computed and test values of useful capacities.

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.

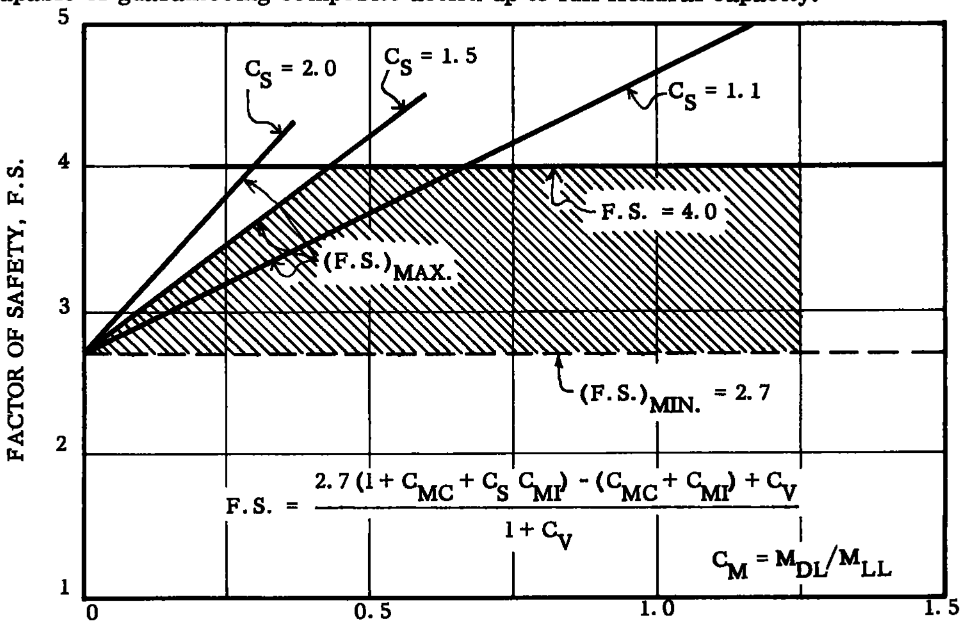


Figure 7. Variation of the factor of safety.

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:

$$Q = \frac{V'_{des} m}{I} p \tag{6a}$$

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,

$$Q_{uc} = \frac{V'_{ult} m}{I} p \tag{6b}$$

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

$$F.S. = \frac{Q_{uc}}{Q} = \frac{N\ V_{LL} + V_{DLc}}{V_{LL} + V_{DLc}} \tag{7a}$$

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.

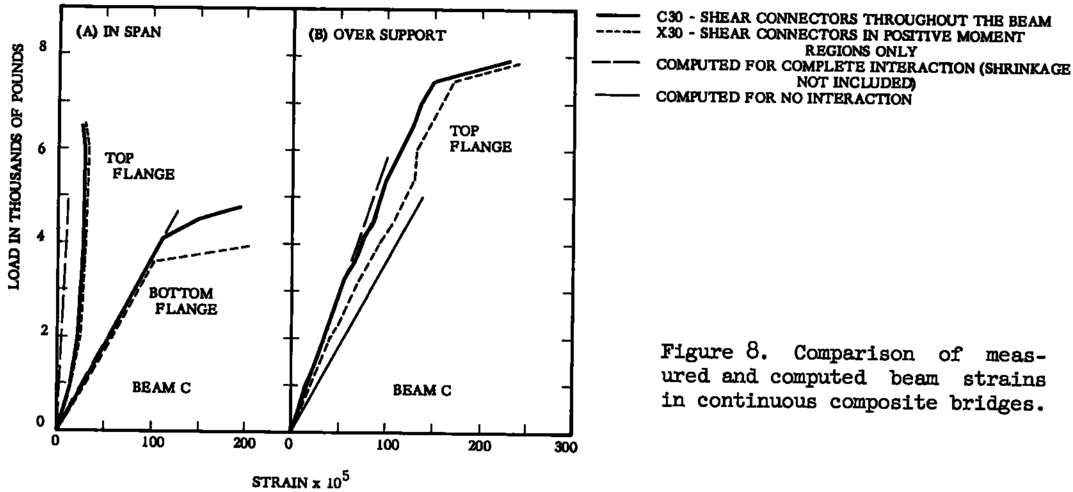


Figure 8. Comparison of measured and computed beam strains in continuous composite bridges.

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

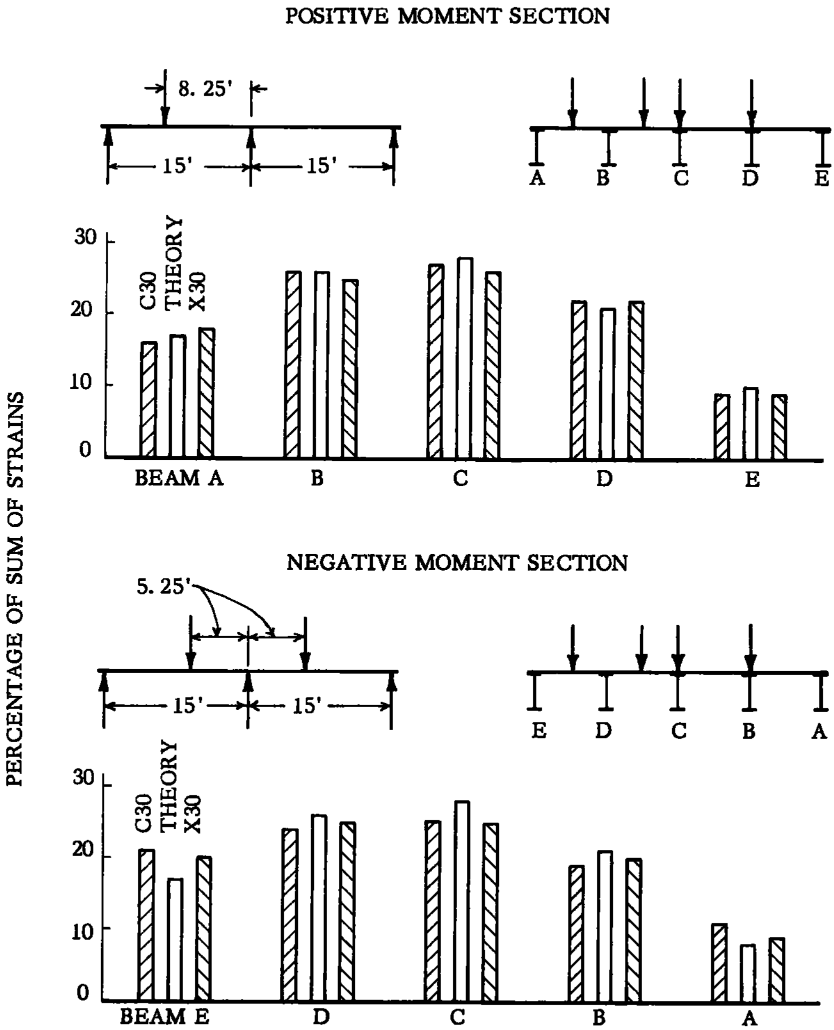


Figure 9. Transverse Distribution of Beam Strains: (a) Bridge C30, Shear Connectors Throughout Bridge; (b) Bridge X30, Shear Connectors in Positive Moment Regions Only; (c) Theory, Distribution for Equivalent Simple-Span Bridge.

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.

REFERENCES

1. American Association of State Highway Officials, "Standard Specifications for Highway Bridges," 4th Edition (1944).
2. Caughey, R. A., "Composite Beams of Concrete and Structural Steel," Proceedings, 41st Annual Meeting, Iowa Engineering Society, 1929, pp. 96-104.
3. Paxson, G. S., "Loading Tests on Steel Deck Plate Girder Bridge With Integral Concrete Floor," Oregon Highway Dept. Tech. Bull. No. 3, (1934).
4. Seiler, J. F., "New Type of Composite Beam," Wood Preserving News, 11: 11, 144 (1933).
5. Baldock, R. H., and McCullough, C. B., "Loading Tests on a New Composite Type Short-Span Highway Bridge Combining Concrete and Timber in Flexure," Oregon Highway Dept. Tech. Bull. No. 1 (1933).
6. Richart, F. E., and Williams, C. B., "Tests of Composite Timber and Concrete Beams," Bull. No. 343, Univ. of Illinois Engineering Experiment Station (1943).
7. Newmark, N. M., and Siess, C. P., "Design of Slab and Stringer Highway Bridges," Pub. Roads, 23: 7, 157 (1943).
8. American Association of State Highway Officials, "Standard Specifications for Highway Bridges," 5th Edition (1949).
9. American Association of State Highway Officials, "Standard Specifications for Highway Bridges," 6th Edition (1953).
10. White, A., "Summary of Composite Bridge Beam Questionnaire Survey," Jour. Amer. Conc. Inst. 52: 1013 (May 1956).
11. Fuller, A. H., "Skunk River Bridge Exhibits Composite Action After Twenty-Eight Years of Service," Civil Eng. 21: 7, 40 (1951). See also, A. H. Fuller, "Effect of Trucks Upon a Few Bridge Floors in Iowa in 1922 and 1928," HRB Research Report 14-B, pp. 1-9 (1952).

12. Van Eenam, N., "Live Load Stress Measurements on Fort Loudon Bridge," HRB Proceedings, 31: 36 (1952).

13. Foster, G. M., "Tests on Rolled Beam Bridge Using H20-S16 Loading," HRB Research Report 14-B, pp. 10-38 (1952).

14. Lin, T. Y., "Load Distribution Between Girders on San Leandro Creek Bridge," HRB Research Report 14-B, pp. 39-45 (1952).

15. Newmark, N. M., Siess, C. P., and Penman, R. R., "Studies of Slab and Beam Highway Bridges: I. Tests of Simple-Span Right I-Beam Bridges," Bull. No. 363, Univ. of Illinois Engineering Experiment Station (1946).

16. Newmark, N. M., Siess, C. P., and Peckham, W. M., "Studies of Slab and Beam Highway Bridges: II. Tests of Simple-Span Skew I-Beam Bridges," Bull. No. 375, Univ. of Illinois Engineering Experiment Station (1948).

17. Richart, F. E., Newmark, N. M., and Siess, C. P., "Highway Bridge Floors, A Symposium," Trans. ASCE, 114: 979 (1949).

18. Siess, C. P., Viest, I. M., and Newmark, N. M., "Studies of Slab and Beam Highway Bridges: III. Small-Scale Tests of Shear Connectors and Composite T-Beams," Bull. No. 396, Univ. of Illinois Engineering Experiment Station (1952).

19. Viest, I. M., Siess, C. P., Appleton, J. H., and Newmark, N. M., "Studies of Slab and Beam Highway Bridges: IV. Full-Scale Tests of Channel Shear Connectors and Composite T-Beams," Bull. No. 405, Univ. of Illinois Engineering Experiment Station (1952).

20. Siess, C. P., and Viest, I. M., "Studies of Slab and Beam Highway Bridges: V. Tests of Continuous Right I-Beam Bridges," Bull. No. 416, Univ. of Illinois Engineering Experiment Station (1953).

21. Viest, I. M., and Siess, C. P., "Composite Construction for I-Beam Bridges," HRB Proceedings, 32: 161 (1953).

22. Viest, I. M., "Investigation of Stud Shear Connectors for Composite Concrete and Steel T-Beams," Jour. Amer. Conc. Inst., 52: 875 (Apr. 1956).

23. Sherman, J., "Continuous Composite Steel and Concrete Beams," Trans. ASCE, 119: 810 (1954).

24. Viest, I. M., "A Study of Shear Connectors for Composite I-Beam Bridges," Proc. Southeastern As. of State Highway Officials, pp. 96-114 (Oct. 1955).

25. American Association of State Highway Officials, "Tentative Revisions, 1953 Edition, AASHO Bridge Specifications," (Dec. 1956).

26. Newmark, N. M., Siess, C. P., and Viest, I. M., "Tests and Analyses of Composite Beams with Incomplete Interaction," Proc. Soc. for Experimental Stress Analysis, 9: 1,75 (1951).

27. Lapsins, V., "A Comparative Study of Composite Action in Structural Steel and Concrete Beams," M.S. Thesis, Dept. of Civil Eng., State Univ. of Iowa (Aug. 1953).

28. Viest, I. M., and Siess, C. P., "Design of Channel Shear Connectors for Composite I-Beam Bridges," Pub. Roads, 28: 1,9 (Apr. 1954).

29. Viest, I. M., "Tests of Stud Shear Connectors, Parts I, II, III and IV," Engineering Test Data, Nelson Stud Welding, Lorain, Ohio.

30. Voellmy, A., "Strength of Alpha Composite Sections Under Static and Dynamic Stresses," Porete Manufacturing Company, North Arlington, N. J.

31. Viest, I. M., "Tests of Spiral Shear Connectors," Engineering Test Data, Nelson Stud Welding, Lorain, Ohio.

32. Washa, G. W., "Volume Changes and Creep," Amer. Soc. for Testing Materials, Special Pub. No. 169, pp. 115-128 (1956).

Composite Beams with Stud Shear Connectors

BRUNO THÜRLIMANN, Research Associate Professor
Lehigh University, Bethlehem, Pennsylvania

Testing of a composite beam structure consisting of a 6-in. deck slab 32 ft long and 10 ft 11 in. wide and two 18WF50 beams 32 ft long, is described. The shear connectors on one of the beams were ¾-in. diameter studs with upset heads, welded to the top flange. On the other beam, ½-in. diameter studs with end hooks at right angle were used.

The loading program consisted of 1 million cycles at 100 percent, an additional 250,000 at 125 percent, and a further 250,000 at 150 percent of the design live load. Final failure was produced by static loading. Both types of shear connectors behaved satisfactorily under all testing conditions.

On the basis of these experimental results tentative design recommendations concerning the use of stud shear connectors are made. The desirability of further experimental studies is indicated, leading ultimately to less conservative design rules.

● USE of composite beams as structural members has become generally accepted for bridge and building construction (1, 2, 3). Essential parts of such beams are the shear connectors providing integral action of the steel beam and the concrete deck slab. They have the double function of transmitting the horizontal shear between slab and beam and of tying down the slab to the beam. Presently, a great variety of different types of shear connectors is successfully used.

The newest addition is stud shear connectors consisting of round studs automatically stud welded to the top flange of the steel beams (2). The simplicity of such connectors is striking in comparison to more elaborate systems already in use. In the present paper the results of tests on a full-scale specimen subjected to extensive repetitive loading and an ultimate static load are briefly reported. On the basis of these results and a corresponding analysis, tentative design recommendations are made.

In this connection it may be important to point out that the design of any type shear connector must necessarily be based on experiments (or experience). The action of the connectors is much too complicated to be accessible to an exact stress analysis. Even the loading of a connector is rather indeterminate, as a considerable amount of shear is transmitted by bond. In case the latter should be broken by slip, mechanical friction is still able to carry part of the shear.

Furthermore, the stress distribution in the connector itself is so highly complicated that any analysis must be regarded as an approximation. It may be well to remember that the design of other connections (for example, riveted or welded connections) is also fundamentally experimental. The allowable shear, bearing, and pitch values, or the allowable stresses of butt and fillet welds, were derived from experience and experiments.

Essentially two possibilities of failure must be considered:

1. In bridge structures the shear acting on the shear connectors is primarily produced by live loads; hence, it is repetitive, so that the possibility of a fatigue failure exists. Even in cases where the steel beams are temporarily supported during the construction of the slab, the dead load stresses in the slab, hence the shear between slab and beam, are greatly reduced by shrinkage and creep. It is therefore essential to investigate the fatigue behavior of a shear connector. Because the actual state of stress in and around a connector is practically inaccessible to an exact analysis, only testing of connectors in actual size and under actual action will furnish conclusive results.

2. Concerning the possibility of failure under static loading, it is desirable that failure not occur in the shear connectors but rather by general yielding of the steel beam. In the latter case no sudden failure will occur. Hence, the shear connection should be able to withstand the shear under ultimate load producing failure by yielding of the steel beam and eventual crushing of the concrete deck slab.

The tests were planned primarily to get experimental information on the behavior of $\frac{1}{2}$-in. and $\frac{3}{4}$-in. diameter stud shear connectors in fatigue. As no fatigue failure developed after a considerable number of overload cycles the ultimate load under static conditions was determined. It is believed that the experimental information is sufficient to recommend conservative design values for the $\frac{1}{2}$-in. studs.

TEST SPECIMEN, SET-UP, AND INSTRUMENTATION

The test specimen (Fig. 1) was built according to a design prepared by the Bureau of Public Roads. It consisted of two 32-ft long, 18WF50 beams (A-7 steel), placed 6 ft apart and a deck slab 6 in. thick and 10 ft 11 in. wide (specified concrete strength f'_c = 3,000 psi). Three crossbeams, 16WF40, placed 1 ft from each end and at mid-length were bolted to the longitudinal beams by high-tensile bolts. The reinforcement of the slab (deformed bars A-305 steel) is also indicated in Figure 1.

One of the beams, referred to as north beam, carried pairs of $\frac{3}{4}$-in. diameter K S M solid fluxed shear connectors with upset heads, 4 in. long, at a constant spacing of $11\frac{1}{2}$-in. The south beam was provided with rows of three $\frac{1}{2}$-in. diameter K S M solid fluxed L-connectors having a height of $1\frac{7}{8}$ in. and a short hook bent at right angle. The spacing was kept constant at 14 in. The stud connectors were welded by the manufacturer's personnel with their equipment.

No special preparation was given the beam before welding. The slab was poured without temporary support of the steel beams, that they carried the entire dead load. Figure 2 shows the slab during pouring.

For testing, the new installation for testing of large assemblies under static and fatigue loading (5) at Fritz Engineering Laboratory, Lehigh University, was used. The specimen was simply supported over a span of 30 ft with supports under each steel beam. Identical loads were applied at mid-span by two hydraulic jacks acting directly above the two steel beams. For cyclic loading the jacks were connected to an Amsler pulsator producing a cyclic, sinusoidally varying load at 250 cycles per minute. In the static test for ultimate load the Amsler pendulum dynamometer applied and measured the pressure in the jacks. The test set-up is shown schematically in Figure 3. Figure 4 shows a picture of the specimen under cyclic loading.

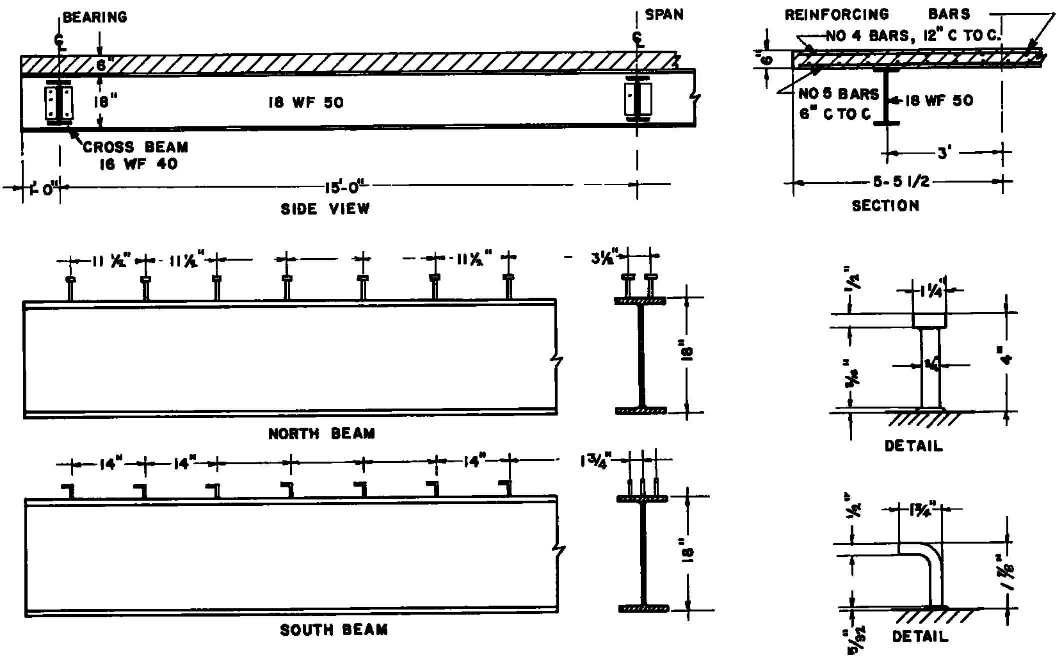


Figure 1. Test specimen design detail.

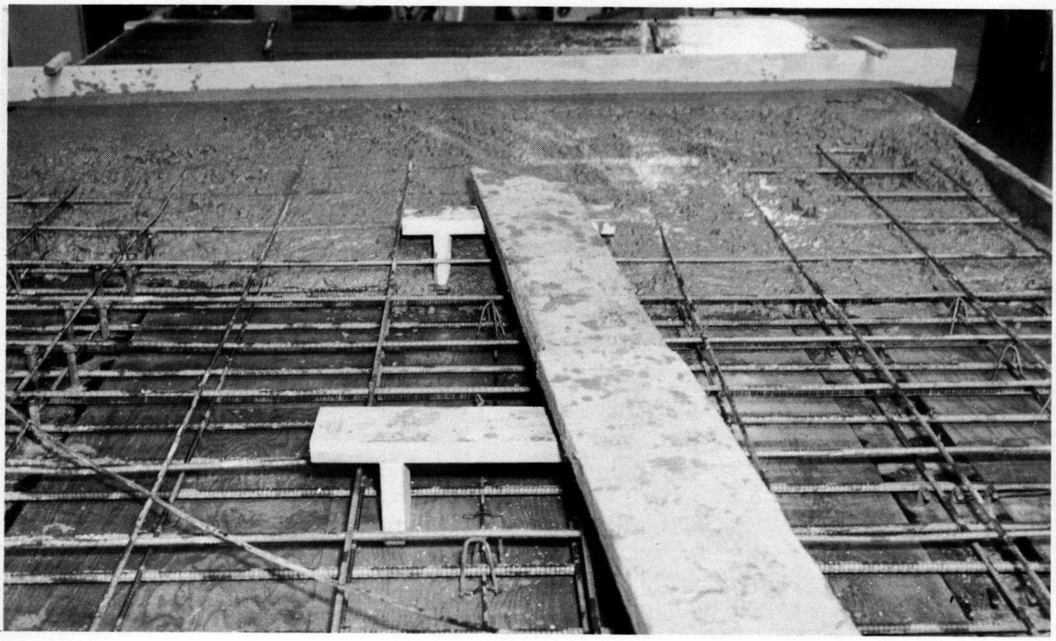


Figure 2.

The instrumentation was intentionally concentrated on a few essential measurements. Ames dials were mounted at the four ends of the beams to measure the relative movement slip between beam and slab. Deflection measurements were taken by means of a level instrument reading 0.001-in. scales attached to the steel beams over the supports and at mid-span.

On each steel beam two SR-4 electrical strain gages were mounted 9 in. from the center (in order not to interfere with the connection of the cross member). One gage was placed in the middle of the bottom flange, the other 16 in. above the bottom flange on the outside of the web (Fig. 5). Readings of the gages were recorded graphically by a "Brush" oscillograph.

PROGRAM, TESTS, AND RESULTS

The test program comprised the following parts:

1. 1,000,000 load cycles alternating between a minimum of 3,700 and a maximum of 33,800 lb (total for both jacks). The maximum load corresponded to the design live load.
2. 300,000 cycles between 4,000 and 42,200 lb. The maximum load corresponded to 125 percent of the design live load.
3. 250,000 cycles between 6,000 and 50,700 lb, the latter being equal to 150 percent of the design live load.
4. Destruction test under static loading.
5. Auxiliary tests for determining the tensile strength of coupons from the 18WF50 beams, the tensile and shear strength of the stud shear connectors, and the cylinder strength of the concrete in the deck slab.

The results of the auxiliary tests are summarized in Tables A to D in the Appendix. The static yield stress indicated in Table A was determined on the tensile coupons at practically zero loading rate within the yield level between the yield point and strain hardening. This value is much more representative for computation of the ultimate load than the upper yield point obtained at a considerable loading rate (for example in mill tests). The double shear tests on the studs reported in Table D produced shear failure on the side opposite to the weld, hence showing a strength of the weld superior to the base material.

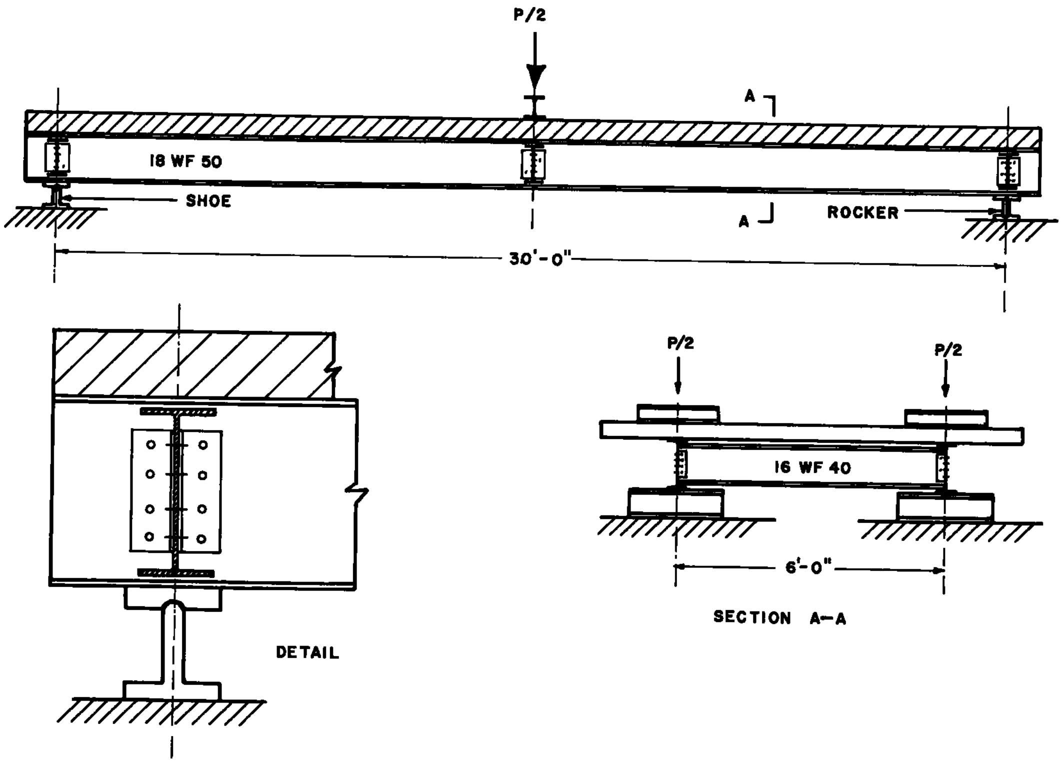


Figure 3. Schematic test set-up.

TABLE 1

INFLUENCE OF REPETITIVE LOADING ON DEFLECTIONS AND SLIP

TEST NO	AGE OF SPECIMEN (IN DAYS)	LOADING	LOADS (IN POUNDS)		C_L DEFLECTION (IN INCHES)		END SLIPS (IN INCHES X 10^4)			
							NORTH BEAM 3/4 INCH STUDS		SOUTH BEAM 1/2 INCH STUDS	
			MINIMUM	MAXIMUM	MEASURED	THEORETICAL(a)	EAST END	WEST END	EAST END	WEST END
1	30	Static	—	33,800	0 235	0 222	0	0	0	2 b
2	33 to 35	1,002,000 Cycles	3,700	33,800	—	—	—	—	—	—
3	36	Static	—	33,800	0 215	0 222	0	0	2	0
4	36	Static	—	44,000	0 280	0 289	0	0	4 b	3
5	36 to 37	296,500 Cycles	4,000	42,200	—	—	—	—	—	—
6	37	Static	—	44,000	0 270	0 289	0	0	2	3
7	41	Static	—	52,000	—	—	0	0	4	6
8	41 to 42	256,800 Cycles	6,000	50,700	—	0 341	—	—	—	—
9	47	Static	—	49,800	0 330	0 327	0	0	4 c	10 c

Notes (a) With $E_s = 30 \cdot 10^6$ psi, $n = 10$

(b) Not recovered

(c) Recovery not checked

The fatigue tests on the composite beam specimen are summarized in Table 1. The load, P, was applied in equal parts, $\frac{P}{2}$, over each steel beam at mid-span (Fig. 3). In order to study the influence of the repetitive loading on the behavior of the specimen—and especially on the deflection and possible slip between steel beam and concrete deck—static loading tests were interposed between the different phases of the fatigue loading program. A total of nine tests were performed in the following sequence (see Table 1):

Test 1 was a static loading test up to the design live load, P = 33,800 lb, producing together with the dead load stresses a computed maximum fiber stress of 18,000 psi. The measured deflection at mid-span and the slip at the four ends of the beams are given in Table 1. The difference in deflection between the north and south beam was less than 0.003 in. and never exceeded this value in the subsequent static tests (tests 3, 4, 6, 7, and 9). As the loads applied to the beams were identical at any time, it can be concluded that no interaction took place between the north and south beams.

Test 2 consisted of 1,002,000 cycles between a minimum load, P_{min} = 3,700 lb, and a maximum load of P_{max} = 33,800 lb. These loads were the actual effective loads, taking into account the influence of the inertia forces. It amounted under the given conditions to 16 percent of the load amplitude. During testing no unusual observations were made.

The subsequent static test (test 3) indicated that 1,000,000 repetitions of full live load did not break the bond or produce any significant slip. The decrease in deflection from tests 1 to test 3 is rather remarkable, indicating an increase in the bending stiffness of the specimen. It is probably attributable to the increase of the modulus of elasticity of the concrete within the six days between test 1 and 3.

An additional 296,500 cycles at 125 percent live load (test 6) and 256,800 cycles at 150 percent live load (test 8) produced neither fatigue failure nor important slip. Concerning a comparison between the slip data for the north beam with ¾-in. studs, showing absolutely no slip, and the south beam with ½-in. studs, exhibiting a maximum slip of 0.001 in., it is shown subsequently that the shear area provided by the studs on the south beam was considerably less than that on the north beam.

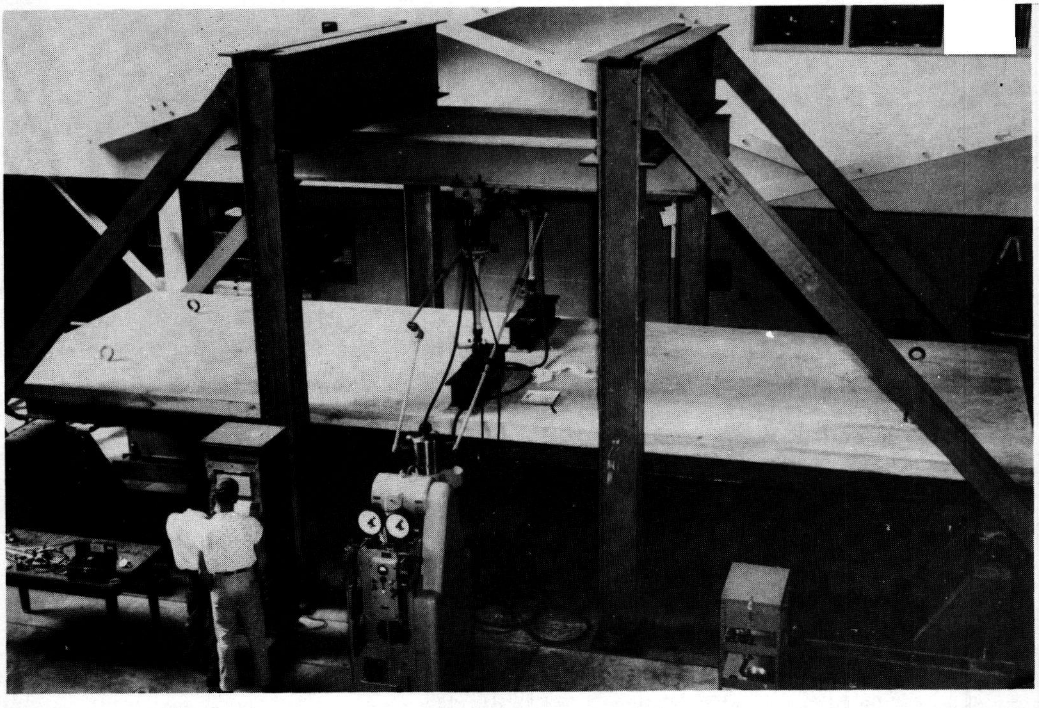


Figure 4.

TABLE 2
SECTION PROPERTIES AND LOADING

Steel Beams: Pair of 18 WF50					
Handbook properties					
Area	A_s	=	2 x 14.71	=	29.42 in.^{2}
Moment of Inertia	I_s	=	2 x 800.6	=	1601.2 in.^{3}
Section modulus	S_s	=	2 x 89.0	=	178.0 in.^{4}
Deck Slab:					
Width	b	=	131 in.		
Thickness	t	=	6 in.		
Area	A_c	=	786 in.^{2}		
Composite Section:					
Reduced area	A	=	108.02 in.^{2}		
Reduced moment of inertia	I	=	4912.0 in.^{4}		
Neutral axis, from top	x	=	6.26 in.		
Distance to top concrete fiber	c_c	=	- 6.26 in.		
Distance to top steel flange	c_u	=	- 0.26 in.		
Distance to bottom steel flange	c_L	=	17.74 in.		
Statical moment of steel area about N.A.	m	=	257.2 in.^{3}		
Loading:					
Dead load, distributed	w	=	920 lb/ft		
Design live load concentrated at mid-span	P	=	33,800 lb		

Test 9, the initial phase of the final static destruction test, was performed in two steps. At a slab age of 47 days the specimen was loaded to a maximum of 145,600 lb with a corresponding deflection of 4.15 in. Ten days later the specimen was brought up to the ultimate load. Figure 6b shows mid-span deflection of the south beam. At deflections of 4.15 and 6.90 in., respectively, the specimen was completely unloaded in order to reset the loading jacks. This is shown in Figure 6b by the unloading curves resulting in permanent sets of 3.00 and 5.50 in., respectively. Also indicated are the following points: Working load first yielding observed from SR-4 readings, first yielding determined from flaking of mill scale, first cracks in concrete slab. The maximum observed load was 172,800 lb, or 5.1 times the design live load, resulting in a mid-span deflection of the south beam of 6.93 in. For comparison the load limits under cyclic loading are shown in Figure 6a.

Figure 7 shows the specimen at ultimate load. The high curvature of the steel beams at mid-span forced the concrete slab to separate from the beam. This was especially pronounced on the south beam, resulting in actual clearance between beam and slab. However, at the locations of the studs the slab was held down effectively. Final failure was brought about by crushing of the concrete slab. The slip measurements are shown in Figure 8. Up to the theoretical yield load, P = 80,100 lb, no slip exceeded 0.0025 in.

Again, it should be stressed that the north beam, with the ¾-in. studs, showing practically no slip, had considerable more shearing area than the south beam with ½-in. studs. Therefore, no direct conclusions as to the influence of stud diameter on the slip should be made from comparison of the slip curves in Figure 8, but an interpretation should be made in the light of the analytical results given in following sections. Important slip on the south beam was setting in at a load of about 120,000 lb, corresponding to

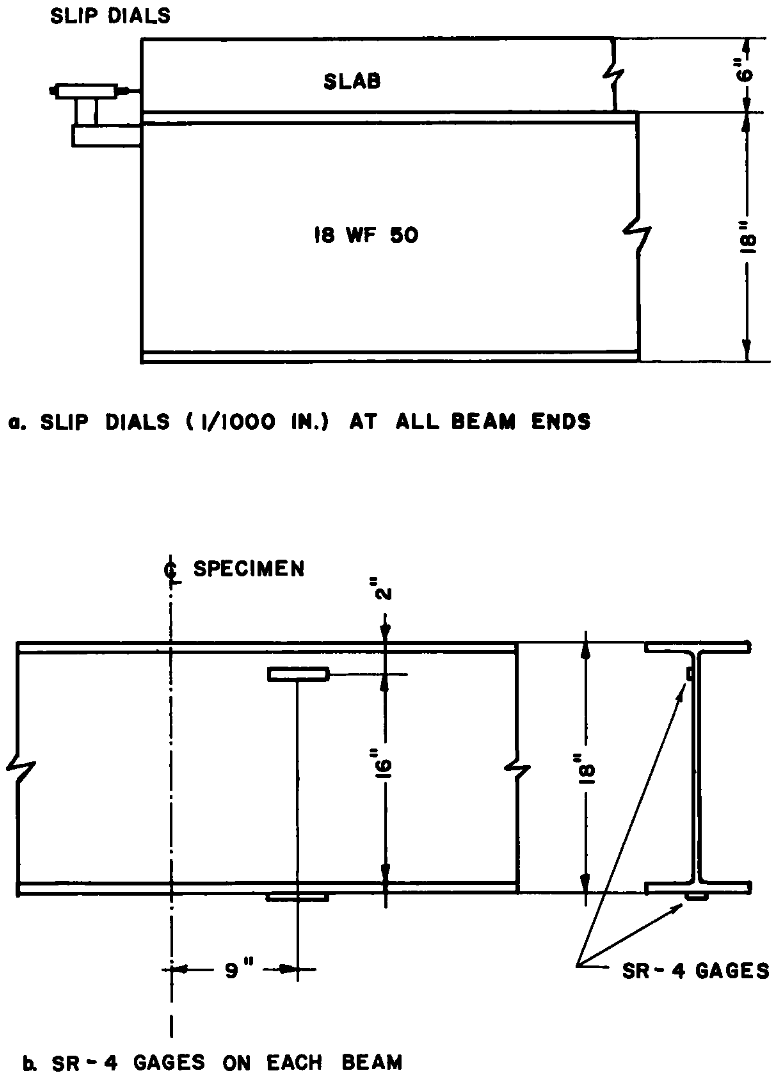


Figure 5. Instrumentation.

about 3.5 times the maximum design live load. At ultimate load the end slips took the following values:

Beam	Studs	Slip, in. West End	Slip, in. East End	Average
South	½ in.	0.0458	0.0634	0.0546
North	¾ in.	0.0108	0.0040	0.0074

Finally, Figure 9 shows the SR-4 strain gage readings. It can be seen that yielding in the bottom flanges occurred at loads of about 90,000 and 120,000 lb for the south and north beam, respectively. Yielding reached the upper gages, 16 in. above bottom flange, at approximately 135,000 lb. An interpretation of the test results becomes only meaningful in conjunction with an analysis. Such a procedure is absolutely necessary, especially if any valid extrapolation beyond the specific conditions of the test is contemplated. Therefore, a short analysis and interpretation of the results is presented in the following.

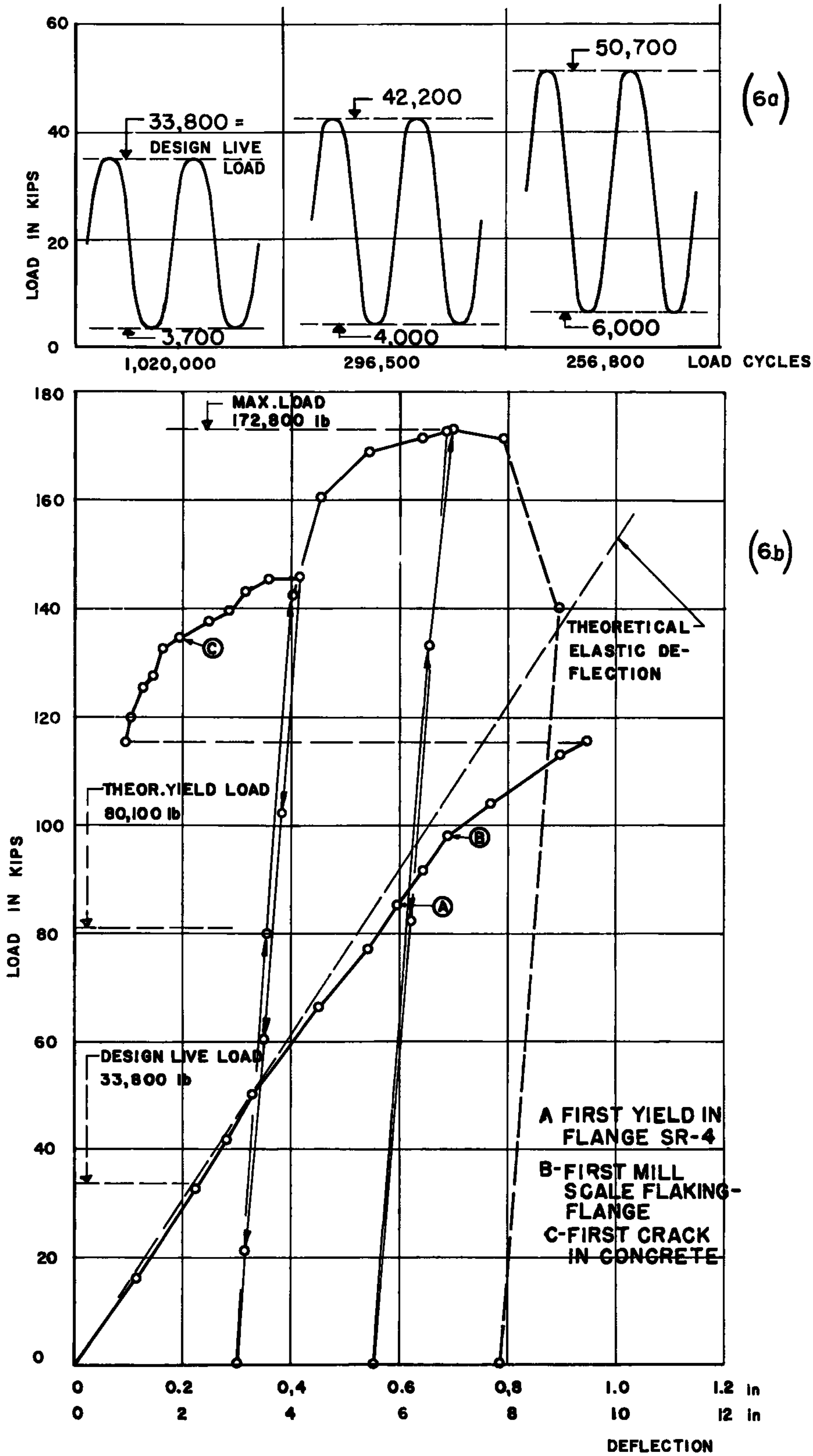


Figure 6. (a) Load limits under cyclic loading; (b) Measured center deflection.

ANALYSIS AND INTERPRETATION OF THE RESULTS

The analysis is based on the following generally accepted assumptions:

In the elastic range:
1. Complete interaction between steel beam and concrete deck; that is, no slip.
2. Modular ratio between steel and concrete, n = 10.
3. All the horizontal shear between slab and beam is transmitted by the studs only.
4. The entire width of the slab is fully effective.

In the plastic range:
5. At ultimate load the steel beam is fully yielded.
6. The compressive stress block in the concrete slab is rectangular in shape.
7. Concrete takes no tension.

The elastic stress and deflection calculations are based on the method of transformed section. In the present case the transformation is made with the steel modulus, E_S, as a basis, hence reducing the concrete section by the modular ratio. Table 2 summarizes the section properties and the loading. The dead load and live load moment are given by:

$$M_{DL} = \frac{wL^2}{8} = \frac{920 \times 30^2}{8} = 103{,}400 \text{ ft-lb} \tag{1}$$

$$M_{LL} = \frac{PL}{4} = \frac{33{,}800 \times 30}{4} = 253{,}500 \text{ ft-lb} \tag{2}$$

Keeping in mind that the dead load is entirely carried by the steel beams (no temporary supports), the dead load stresses are:

$$f_{DL} = \frac{M_{DL}}{S_S} = \frac{103{,}400 \times 12}{178} = \pm\ 6{,}980 \text{ psi} \tag{3}$$

The negative sign corresponds to compression in the top flange of the steel beam, the positive sign to tension in the bottom flange.

The live load produces stresses in the composite sections as follows:

$$f_{LL} = \frac{M_{LL}c}{I} \tag{4a}$$

Top flange of steel beams, with $c = c_u = -0.26$ in.

$$f_{LL} = \frac{253{,}500 \times 12(-0.26)}{4{,}912} = -161 \text{ psi} \tag{4b}$$

Bottom flange of steel beam, with $c = c_L = 17.74$ in.

$$f_{LL} = \frac{253{,}500 \times 12 \times 17.74}{4{,}912} = 10{,}980 \text{ psi} \tag{4c}$$

The concrete stress is obtained from Eq. 4a by dividing with the modulus ratio, n = 10.

Maximum concrete stress, with $c = c_c = -6.26$ in.

$$f_{LL} = \frac{253{,}500 \times 12\ (-6.26)}{4{,}912} \times \frac{1}{10} = -388 \text{ psi} \tag{4d}$$

The horizontal shear force, S, per unit length between one steel beam and the concrete slab is determined from:

$$S = \tfrac{1}{2} \times \frac{V\,m}{I} \tag{5a}$$

with V being the total vertical shearing force at the section under investigation and in the statical moment of the steel area about the neutral axis. The factor $\frac{1}{2}$ attributes half of the total horizontal shear to each of the two beams. Concerning the value of V,

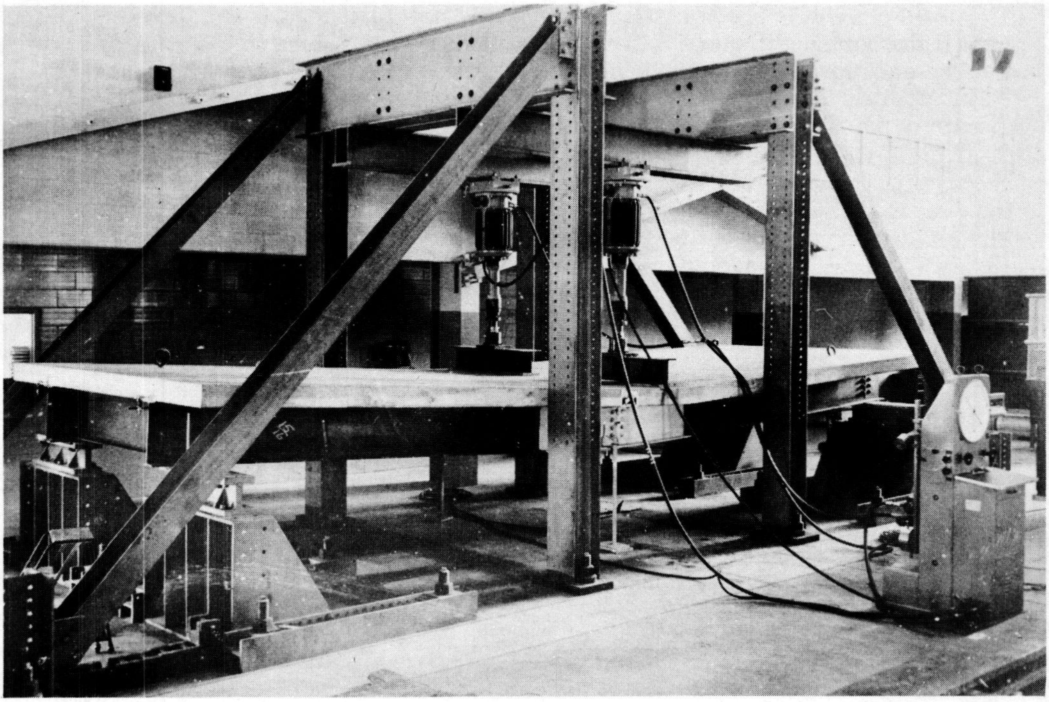


Figure 7.

dead load did not produce any shear. The live load shear was uniform over one-half the span length, with a change equal to P at the loading point.

$$V_{LL} = \frac{P}{2} = 16{,}900 \text{ lb} \tag{6}$$

Hence,

$$S = \frac{1}{2} \times \frac{16{,}900 \times 257.2}{4{,}912} = 442 \text{ lb/in.} \tag{5b}$$

It may be added that this shear is acting on the top flange of the steel beam in an outward direction from mid-span. Following the assumption that all shear is transmitted by the studs exclusively, the force Q per connector can readily be calculated. If the spacing is s and the number of shear connectors in one row is n,

$$Q = \frac{S\,s}{n} \tag{7}$$

Applying Eq. 7 to the present case gives:

$\frac{3}{4}$-in. studs (north beam): $s = 11.5$ in.
$n = 2$

$$Q = \frac{442 \times 11.5}{2} = 2{,}540 \text{ lb} \tag{8}$$

½-in. studs (south beam): $s = 14$ in.

$n = 3$

$$Q = \frac{442 \times 14}{3} = 2{,}060 \text{ lb} \tag{9}$$

It should be emphasized that under actual conditions the connector forces are considerably smaller, as most of the shear is transmitted by bond or mechanical friction. The same applies also to riveted connections, where under working conditions considerable shear is transmitted by friction between the plates, created by the clamping action of the rivets. However, the computation is by no means meaningless; it presents an index of comparison and can also be used for design, as shown later.

The average shearing stress in a stud follows:

$$v_{st} = \frac{Q}{A_{st}} \tag{10}$$

where A_{st} is the cross-sectional area of one stud. Again, v_{st} should be taken as a nominally computed stress. For the test specimens the corresponding values are:

¾-in. studs: $A_{st} = 0.441$ sq in.

$$v_{st} = \frac{2{,}540}{0.441} = 5{,}750 \text{ psi}$$

½-in. stud: $A_{st} = 0.196$ sq in.

$$v_{st} = \frac{2{,}060}{0.196} = 10{,}500 \text{ psi}$$

TABLE 3

SUMMARY OF THEORETICAL STRESS COMPUTATION

NO	LOADING	CONCRETE STRESSES		STEEL STRESSES		CONNECTORS			
						3/4 INCH STUDS		1/2 INCH STUDS	
		TOP (in psi)	BOTTOM (in psi)	TOP FLANGE (in psi)	BOTTOM FLANGE (in psi)	FORCE PER STUD (in lbs)	AVERAGE SHEARING STRESS (in psi)	FORCE PER STUD (in lbs)	AVERAGE SHEARING STRESS (in psi)
(1)	Dead Load W = 920 lbs/ft	—	—	−6,980	−6,980	—	—	—	—
(2)	Design Live Load P = 33,800 lbs	−388	−16	−161	+10,980	2,540	5,750	2,060	10,500
(3)	125% Live Load P = 42,200 lbs	−485	−20	−201	+13,710	3,180	7,190	2,580	13,100
(4)	150% Live Load P = 50,700 Lbs	−582	−24	−242	+16,480	3,810	8,620	3,090	15,700
(5)	DL+LL=(1)+(2)	−388	−16	−7,141	+17,960	2,540	5,750	2,060	10,500
(6)	DL+125%LL=(1)+(3)	−485	−24	−7,181	+20,690	3,180	7,190	2,580	13,100
(7)	DL+150%LL=(1)+(4)	−582	−24	−7,222	+23,460	3,810	8,620	3,090	15,700

It is now apparent that the ½-in. studs on the south beam were under much more severe loading than the ¾-in. studs on the north beam, regardless of the exact values of the connector forces, Q.

For the higher live loads of 125 and 150 percent, the same equations apply, as the specimen is still within the elastic range. The corresponding results are summarized in Table 3.

The mid-span deflection, y, due to live load is readily computed from

$$y = \frac{PL^3}{48\,E_S I} \tag{11}$$

with $E_S = 30 \times 10^6$ psi being the modulus of elasticity of steel. Therefore,

$$y = \frac{33,800 \times 30^3 \times 12^3}{48 \times 30 \times 10^6 \times 4,912} = 0.222 \text{ in.} \tag{12}$$

At ultimate (maximum) load, consideration must be given to the inelastic behavior of steel and concrete. The following analytical considerations are based on an assumed stress distribution over a cross-section as shown in Figure 10. With the entire steel section yielded, the maximum resultant tensile force, T, is determined in magnitude and location at the centroid of the steel section.

$$T = f_y A_S = 38,700 \times 29.42 = 1,140,000 \text{ lb} \tag{13}$$

where f_y = 38,700 psi is the average static yield stress of the beam and A_S the cross-sectional area of the two WF beams. The resultant compressive force, C, in the con-

TABLE 4

COMPARISON BETWEEN BEHAVIOR OF ¾ INCH STUDS AND ½ INCH STUDS

NO	APPLIED SHEAR V (in lbs)	MEASURED END SLIP (IN INCHES X 10^4)				AVERAGE SLIP (IN INCHES X 10^4)		COMPUTED VALUES			
		NORTH BEAM 3/4 INCH STUDS		SOUTH BEAM 1/2 INCH STUDS				3/4 INCH STUDS		1/2 INCH STUDS	
		EAST END	WEST END	EAST END	WEST END	3/4 INCH STUDS	1/2 INCH STUDS	FORCE Q PER CONNECTOR (in lbs)	AVERAGE SHEARING STRESS V_{st} (in psi)	FORCE Q PER CONNECTOR (in lbs)	AVERAGE SHEARING STRESS V_{st} (in psi)
(1)	0	0	0	0	0	0	0	0	0	0	0
(2)	7990	0	0	0	0	0	0	1,200	2,710	971	4,960
(3)	16250	0	0	2	6	0	4	2,440	5,540	1,980	10,100
(4)	20800	0	0	4	8	0	6	3,140	7,080	2,530	12,930
(5)	24900	0	0	4	10	0	7	3,740	8,470	3,030	15,480
(6)	33300	0	0	4	12	0	8	4,980	11,280	4,030	20,600
(7)	38600	0	0	8	22	0	15	5,810	13,180	4,700	24,000
(8)	42600	0	4	12	26	2	19	6,710	14,500	5,200	26,550
(9)	52000	0	8	20	38	4	29	7,830	17,700	6,340	32,400
(10)	60000	0	16	30	50	8	40	9,040	20,400	7,310	37,400
(11)	66400	0	26	46	70	13	58	10,000	22,600	8,100	41,300
(12)	68800	6	32	82	102	19	92	10,320	23,400	8,370	42,700
(13)	72600	12	40	150	172	30	161	10,920	24,750	8,850	45,100
(14)	72800	18	56	244	234	37	239	10,980	24,800	8,880	45,300

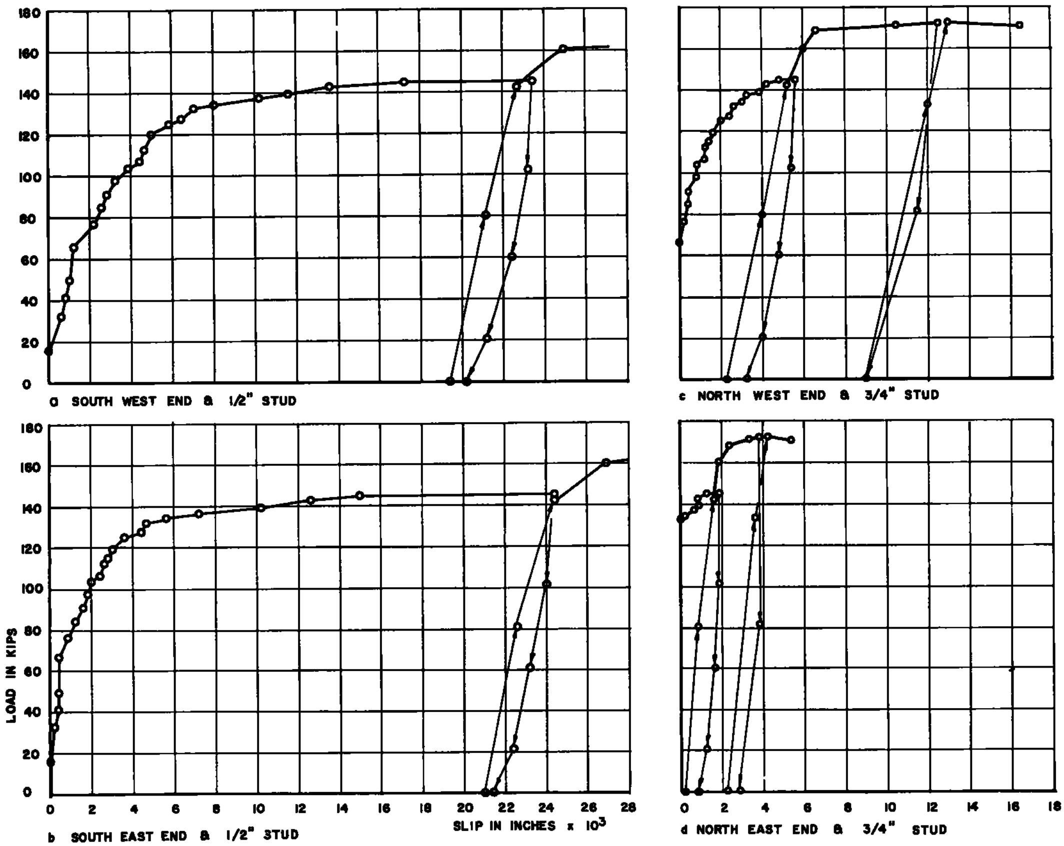


Figure 8. Measured end slips.

crete is equal but opposite to T. It is also determined by the slab width, the cylinder strength of the concrete, and the depth of the rectangular stress block; hence,

$$C = b f_c x$$

with b = slab width, f_c = cylinder strength of concrete, and x = depth of rectangular stress block (see Fig. 10).

With the pertinent numerical values the depth is

$$x = \frac{T}{b f_c} = \frac{1,140,000}{131x3,930} = 2.22 \text{ in.} \tag{14}$$

Having x, the distance d between T and C is $d = 9 + 6 - \frac{2.22}{2} = 13.89$ in.

The ultimate moment is

$$M_{ult} = d T = 13.89x1,140,000 = 15,800,000 \text{ in/lb} \tag{15}$$

and

$$P_{ult} = \frac{M_{ult}}{L} = \frac{4x15,800,000}{30x12} = 175,800 \text{ lb} \tag{16}$$

For the computation of the horizontal shear force, S, per unit length of one steel beam Eq. 5a still applies at the ends of the specimen where the stresses are within the elastic range. However, near mid-span this equation would give much too high results.

An average S can be computed by considering equilibrium of one-half the concrete slab as a free body, as shown in Figure 10.

Then

$$S = \frac{1}{2}\,\frac{C}{L/2} = \frac{1}{2} \times \frac{1,140,000}{16\mathrm{x}12} = 2,970 \text{ lb/in.} \qquad (17)$$

Again, the factor ½ distributes the horizontal shear evenly to the two beams. The actually measured ultimate load, P = 172,800 lb, coincided closely with the predicted value given by Eq. 16. Assuming that the distance d was at its computed value, the actual resultant compressive force in the concrete was

$$C = \frac{172,800}{175,800} \times 1,140,000 = 1,120,000 \text{ lb}$$

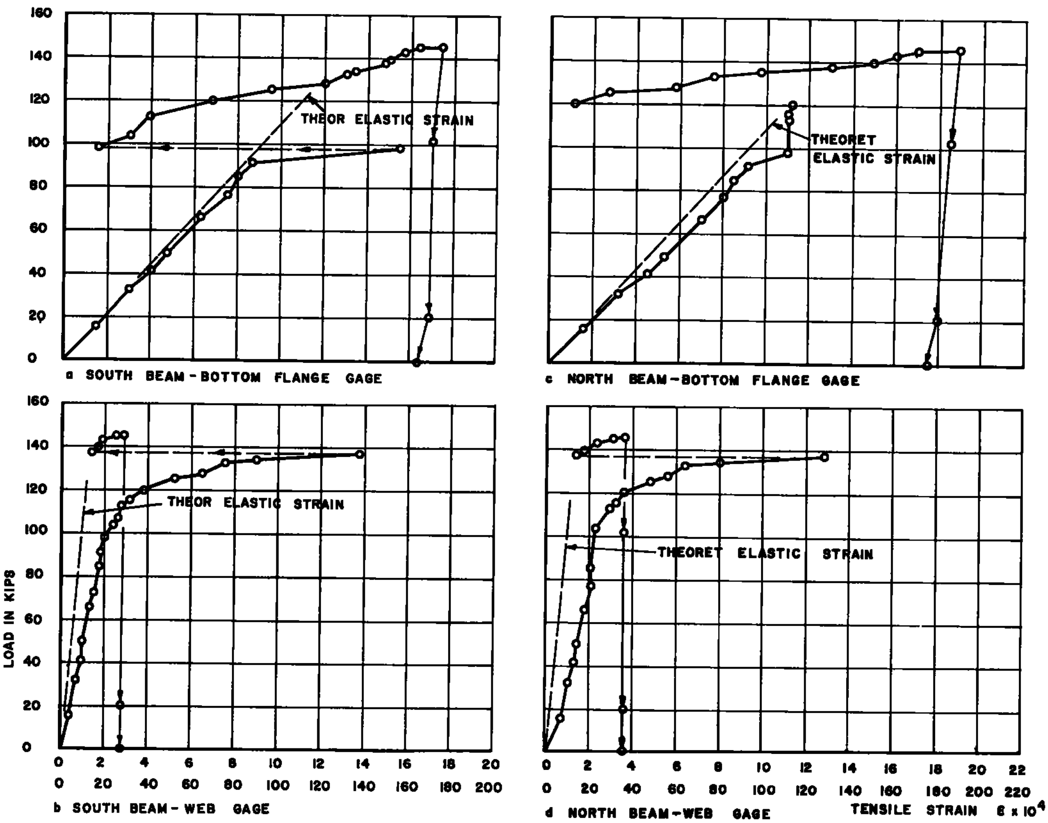


Figure 9. Measured strains.

and the corresponding average shear force in the test

$$S = \frac{1}{2} \times \frac{1,120,000}{16\mathrm{x}20} = 2,915 \text{ lb/in.}$$

Using this latter value the connector forces and the average shearing stress in each connector at ultimate load can be computed from Eqs. 7 and 10.

¾-in. studs: $$Q = \frac{2,915\mathrm{x}11.5}{2} = 16,770 \text{ lb}$$

$$v_{st} = \frac{16,770}{0.441} = 38,000 \text{ psi}$$

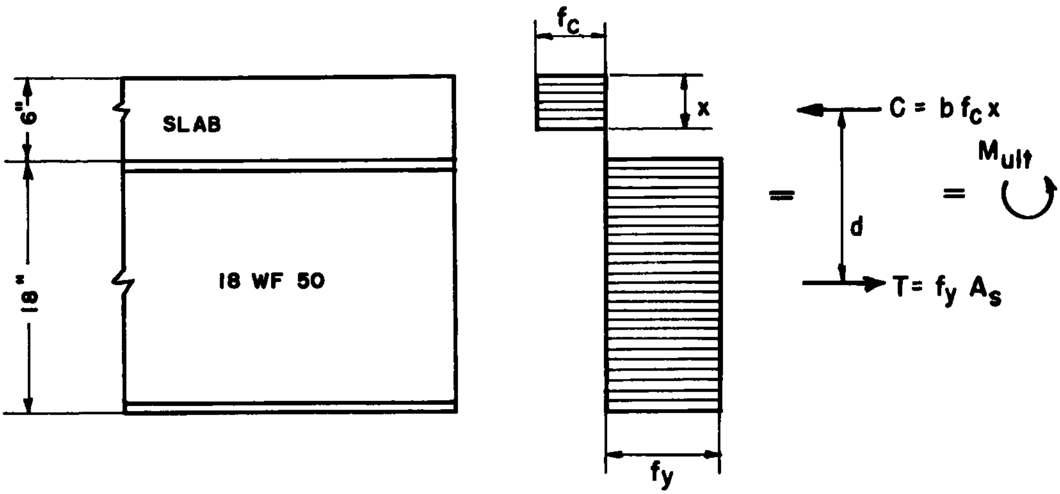
$$\tfrac{1}{2}\text{-in. studs:} \quad Q = \frac{2{,}915 \times 14}{3} = 13{,}600 \text{ lb}$$

$$v_{st} = \frac{13{,}600}{0.196} = 69{,}400 \text{ psi}$$

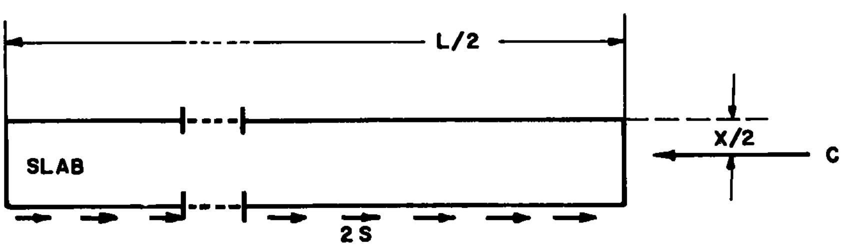
Once more it is stressed that for instance the average shearing stress v_{st} = 69,400 psi represents a nominal value, the actual maximum shearing stress being considerably smaller because in the analysis the frictional resistance between steel beams and slab was neglected. Nevertheless this hypothetical value is not useless. With proper interpretation it allows the derivation of safe design values.

Comparison of the theoretical results for deflections and bending stresses of the steel beams with the corresponding experimental results are made in Figures 6 and 9, which in general show fair agreement. Keeping in mind the local influence of the loads on the strain distribution in their immediate neighborhood, the SR-4 readings are in satisfactory agreement. Even under cyclic loading these readings were in fair correspondence with the theoretically computed values. Figure 11 compares an oscillogram taken at 125 percent live load (P_{max} = 42,200 lb, P_{min} = 4,000 lb) with theoretically computed values.

No attempt was made to measure the strains in and around the studs. Besides the fact that such measurements would probably offer extreme difficulties, it is believed that they would not be of much value in arriving at design recommendations. There are



a. DISTRIBUTION OF BENDING STRESSES



b AVERAGE HORIZONTAL SHEAR FORCE S

Figure 10. Assumed conditions at ultimate load.

very likely relatively high local stresses in the studs and in the surrounding concrete. However, it is known that such local stresses as they occur in many other cases (for example, around rivet holes, in welds, under concentrated reactions in concrete) can safely be sustained without damage to the member as such. Therefore, it is felt that nominal stress computations for the average shearing stress in the studs, neglecting probable bond and friction, are sufficient to derive safe working values.

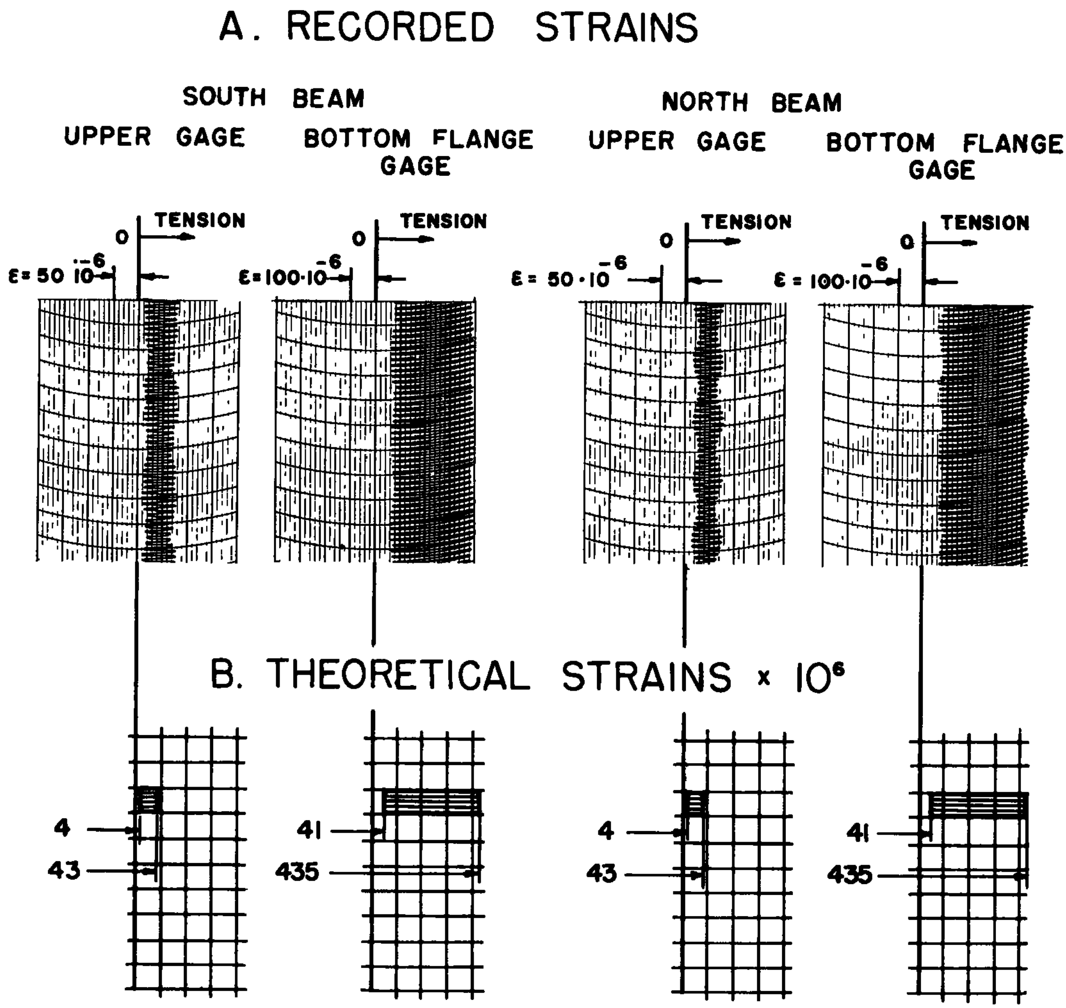


Figure 11. Comparison between recorded and calculated strains under cyclic loading; P_{min} = 4,000 lb, P_{max} = 42,200 lb.

A final comparison between the behavior of the ¾-in. studs and the ½-in. studs may be useful. Rather than to plot a slip vs load diagram, as in Figure 7, the average shearing stress in the stud (from Eqs. 5a, 7, and 10) corresponding to the measured slip is computed in Table 4 and presented in Figure 12. It can be seen that ½-in. studs started to slip at an average shearing stress of about 5,000 psi, compared to a stress of about 13,000 psi for the ¾-in. studs. However, the smaller studs seemed to "hang on" better, surpassing the larger studs at a slip of about 0.0008 in.

The geometric shape of the connectors could be the possible explanation of this rather surprising behavior. The hook of the small studs comes into action at the instant bending of the stud starts. It has the tendency to press the slab more firmly against the top flange and hence increase the mechanical friction. The larger studs with the upset head are longer and may have less tendency to increase this contact pressure. Push-out tests may be extremely helpful to clarify this point.

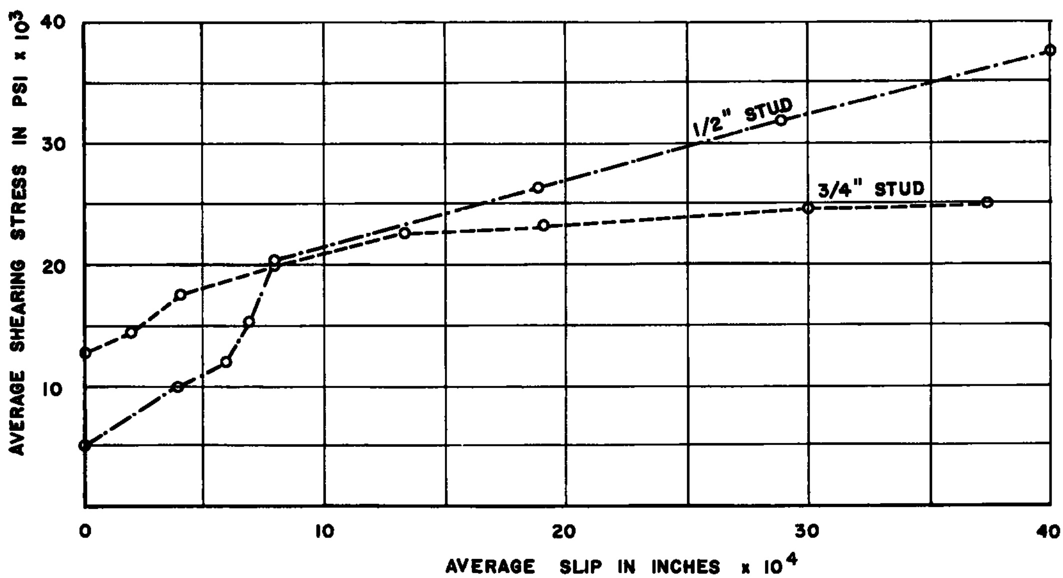


Figure 12. Comparison between behavior of 3/4-in. stud and 1/2-in. stud.

DESIGN RECOMMENDATIONS

The described test is one of the very few full-scale fatigue tests of composite beams, although a few fatigue tests on specimens of about 12-ft length have been reported (3). It is believed that the results obtained warrant the recommendation of design values for ½-in. stud shear connectors of the type tested. As the north beam with ¾-in. shear connectors offered more shear area per unit length than the south beam with the ½-in. studs (0.077 sq in. vs 0.042 sq in. per inch of length) the recommendations are only made for ½-in. studs. However, comparison of the behavior of the ¾-in. studs with design recommendations proposed by AASHO (4) is made.

The recommendations are stated first, followed by the necessary explanations.

Design Recommendation for ½-Inch "L" Shear Connectors

1. Bridge Design
   - (a) The geometric shape of the connector is shown in Figure 13.
   - (b) The hooks should preferably be oriented against the direction of the horizontal shear (toward middle for simple beams).
   - (c) The maximum pitch should not be more than 24 in.
   - (d) The useful static capacity of the shear connector in pounds is given by

$$Q_{uc} = 7,300 \sqrt{\frac{f_c'}{3,600}} = 120 \sqrt{f_c'}$$

in which f_c' is the 28-day cylinder strength of concrete, in psi.

The resistance value at working load is obtained by dividing Q_{uc} by the appropriate safety factor (4).

   - (e) In any case, the allowable maximum connector force in pounds (Q_{all}) produced by live load (or dead load plus live load in case of shoring of steel beams) should be limited to

$$Q_{all} = 2,500 \text{ lb}$$

2. Building Design (Primarily Static Loading):
   (a) Useful capacity of the shear connector in pounds:

$$Q_{uc} = 120\sqrt{f_c'}$$

   (b) Allowable maximum connector force in pounds:

$$Q_{all} = \tfrac{1}{2}\,Q_{uc} = 60\sqrt{f_c'}$$

The following considerations lead to these recommendations: Recommendation (a) proposes a slightly longer shear connector than the ½-in. connectors used in the test specimen. With this increased height it is possible to accommodate the lower transverse reinforcing steel directly in the bend of the connector and still maintain a minimum concrete cover of 1 in. It is believed that fastening the reinforcement as indicated will be beneficial for composite action.

For recommendation (b), reference is made to Figure 7. In the test specimen all hooks were oriented toward the west end of the beam. The result was a smaller slip on the southeast end than the southwest end. Therefore, a proper orientation of the hooks as proposed should improve the interaction.

Recommendation (c) is taken from the AASHO Specifications (4). The useful capacity of the connectors is determined from the load slip curves, Figure 8. It should be remembered that the ultimate moment as computed was nearly reached despite considerable slip. However, it is probably desirable to limit this slip. From Table 4 it can be seen that the average slip for ½-in. connectors was 0.0040 in. at an average connector force Q = 7,310 lb. Beyond this force slip started to develop rapidly. It is felt that a total slip of about 0.0040 in. is quite tolerable. The corresponding residual slip is less than 0.0030 in. From this condition the value shown under (d) was derived. The influence of the concrete strength on the capacity Q_{uc} was assumed to vary as the square root of the cylinder strength (2, 4). At the date of testing the concrete had an age of 47 days with a cylinder strength of 3,600 psi.

The tentative AASHO Specifications (4) for the design of stud shear connectors (Art. 3.9.5—Shear) cover only straight studs with upset heads of 4-in. height. A check on the ¾-in. straight studs on the north beam shows the following behavior. At a load of 145,600 lb the average slip reached 0.0037 in. (see Table 4 and Figure 8). This corresponds to a useful capacity of approximately

$$Q_{uc} = 11,000\sqrt{\frac{3,000}{3,600}} = 10,100 \text{ lb}$$

based on a concrete strength $f_c' = 3,000$ psi. According to the AASHO Specification (4) the corresponding capacity of a ¾-in. diameter stud is

$$Q_{uc} = 332\, d^2\sqrt{f_c'} = 332 \text{x} 0.75^2\sqrt{3,000} = 10,200 \text{ lb.}$$

The correspondence with the test value is rather remarkable.

However, in checking the ½-in. L-connectors on the south beam no such correspondence was observed. Indeed the AASHO formula covers only straight studs with upset heads of 4-in. height (15 percent reduction for 3-in. height), whereas the L-connectors introduce a new geometric shape not previously tested. The test results lead to the following useful capacity for ½- in. L-connectors:

$$Q_{uc} = 120\sqrt{f_c'} = 120\sqrt{3,000} = 6,580 \text{ lb.}$$

A rigid application of the AASHO formula to ½-in. L-connectors would yield

$$Q_{uc} = 332\ d^2 \sqrt{f_c'} = 332 \text{x} 0.50^2 \sqrt{3,000} = 4,540 \text{ lb}$$

The considerable difference indicates that the latter formula does not cover the behavior of L-connectors. However, if applied it will result in an inequitable evaluation of the L-connector and, consequently, in a very conservative design.

Further tests are under way to substantiate the difference in behavior of the L-connectors and the straight stud connectors with upset heads.

Recommendation (e) follows directly from the behavior under cyclic loading. It is felt that the sustained cyclic loading assures a sufficient factor of safety against fatigue even under 125 percent of the design live load. The corresponding connector force for the ½-in. L-connectors was 2,580 lb (see Table 3). In an actual bridge the connectors at mid-span may be subjected to complete load reversals in short-span bridges. However, the test conditions were more extreme in the magnitude of the loads than any conditions occurring in practice. It is likely that recommendation (e), rather than (d), will generally govern a design. Further fatigue tests may lead to a relaxation of this recommendation.

Further research work on the fatigue behavior of different types of shear connectors is certainly desirable. Presently, comparative studies are restricted to static push-out tests (1, 2). It is felt that fatigue tests may substantially contribute to an investigation of the effectiveness and reliability of the different types of connectors.

Recommendation 2 (b) for the allowable connector force in building design (primarily static loading) contains a safety factor of 2. According to the "American Institute of Steel Construction Specifications," steel structures are designed with a safety factor of 1.65 against nominal yielding and 1.88 against ultimate load. As the evaluation of the stud behavior was on an ultimate basis, a safety factor of 2 is certainly adequate.

ACKNOWLEDGMENT

This paper is based on experimental results reported in Fritz Laboratory Report No. 200: 56: 225: 1 by George C Driscoll, Jr. The tests were sponsored by K S M Products, Inc., Stud Welding Division, Merchantville, N. J.

The test specimen was designed by E. L. Erickson, Chief, Bridge Branch, Bureau Public Roads, and his staff. The testing procedure was planned and developed by Mr. Driscoll in conjunction with K S M Products, Inc. The tests were performed at Fritz Engineering Laboratory, Lehigh University, of which Professor W. J. Eney is director.

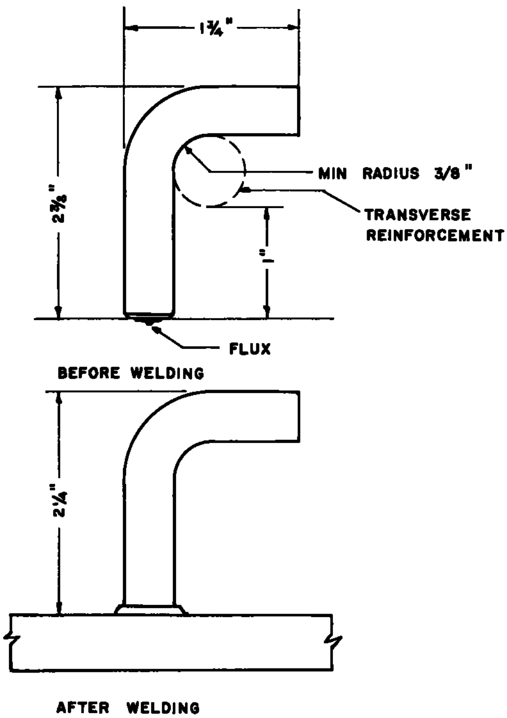


Figure 13. Detail dimensions of ½-in. diameter "L-stud connector."

REFERENCES

1. Viest, I. M., and Siess, C. P , "Composite Constructions for I-Beam Bridges." Proc. Highway Research Board, 32: 161 (1953).

2. Viest, I. M., "Investigation of Stud Shear Connectors for Composite Concrete and Steel I-Beams." Jour. Amer. Conc. Inst., 27: 8,875 (Apr. 1956).

3. Ros, M., "Träger in Verbundbauweise," Report No. 149, Swiss Federal Mat. Test Lab. (EMPA), Zurich (1944).

4. "Standard Specifications for Highway Bridges," T14(56), Art. 3.9.5., Amer. Assn. of State Highway Officials, Wash., D. C., (1956).

5. Thürlimann, B., and Eney, W.J., "Modern Installation for Testing of Large Assemblies under Static and Fatigue Loading," Fritz Laboratory Report No. 237.7, Lehigh University, Bethlehem, Pa.

Appendix

TABLE A

Tensile Coupon Strengths of Steel in 18WF50

SPECIMEN NOS	MATERIAL	LOCATION	STATIC YIELD STRESS (psi)	ULTIMATE STRESS (psi)
1	ASTM	Flange	36,000	62,680
2	A-7	Web	41,300	64,580
3	Structural	Flange	35,500	62,460
4		Web	42,000	65,340
		Average	38,700	63,765

TABLE B

Cylinder Strength of Concrete in Slab

(Mix Design 3000 psi)

SPECIMEN NOS	AGE AT TEST (days)		STRENGTH (psi)
5	7		2,680
6	7		2,460
7	7		2,150
	7	Average	2,430
8	30		3,480
9	30		3,570
10	30		2,790
	30	Average	3,280
11	64		3,887
12	64		4,240
13	64		4,240
14	64		3,357
	64	Average	3,931

TABLE C

Tensile Tests of Stud Material

SPECIMEN NOS.	MATERIAL	NOMINAL DIAMETER (in)	YIELD STRESS (psi)	ULTIMATE STRESS (psi)	LOCATION OF FRACTURE
15	ASTM	1/2	59,700	70,500	Middle of Rod
16	A15-54T	1/2	59,300	70,000	" " "
17	Structural	3/4	65,200	71,200	" " "
18		3/4	65,000	70,500	" " "

Table D

Double Shear Test on Welded Shear Connectors

SPECIMEN NOS	MATERIAL	NOMINAL DIAMETER (in)	ULTIMATE SHEAR LOAD (lb)	ULTIMATE SHEARING STRESS (psi)	REMARKS
19	ASTM	1/2	28,300	72,300	Stud Sheared
20	A15-54T	1/2	28,600	73,000	" "
21	Structural	1/2	25,600	65,300	" "
22		3/4	52,000	58,900	" "
23		3/4	48,500	55,000	" "
24		3/4	39,600	44,800	Excessive Bending before stud sheared

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY-COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.