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# Continuous Composite Beams for Bridges

Poutres mixtes continues pour des ponts

Durchlaufende Verbundträger für Brücken

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## 1. INTRODUCTION

Composite steel and concrete beams for bridges have been used extensively in the United States for a number of years. The popularity of composite bridge construction led to the adoption in 1944 of general specifications covering this type of construction. Extensive research leading to practical design rules for shear connectors and the introduction of stud shear connectors resulted in the widespread use of composite design in bridges by the end of the 1950's. Today composite construction is used in bridges primarily for medium span overpass structures. A report outlining the present worldwide state of the art is contained in Ref. 1.

Composite design for bridges in the United States has been limited primarily to simple spans or to the positive moment (slab in compression) regions of continuous spans. In the latter case, even though the steel beam and the concrete deck are continuous, connectors are usually omitted from the negative moment (slab in tension) regions over interior supports for basically two reasons: (a) the allowable live load stress range in the top (tension) flange of the steel beam considering fatigue requirements for connectors in this region is substantially reduced(2) and (b) there are presently no simple design rules which consider important factors such as connector strength, influence of slab cracking, optimum reinforcement, effective width and time effects (creep and shrinkage). As a result, most designs ignore any contribution of the slab and reinforcement in the negative moment regions.

Continuous composite designs with discontinuous shear connection are permitted by the 1969 AASHO bridge specifications providing that additional connectors are placed in the vicinity of the dead load inflection points to prevent overstressing of the shear connectors in the positive moment regions. The AASHO specifications therefore provide an interim means of improving designs of this type. There are no provisions which consider the influence of, or control of slab cracking.

A full scale fatigue test of a two-span continuous composite tee beam designed according to the 1969 AASHO specifications and employing discontinuous shear connection did in fact result in fatigue failures in the top layer of longitudinal reinforcement over the interior support at about half the design life based on the fatigue requirements of the connectors.(3) Bridges in service are experiencing deterioration of the slabs over the interior supports due in part to the presence of cracks resulting from shrinkage of the concrete as well as from live load stresses.

The balance of this paper will be devoted to a more detailed discussion of some of the points just raised. Emphasis will be given to the results of recent research at Fritz Engineering Laboratory concerning continuous composite beams for bridges.

## 2. PRESENT STATUS IN THE UNITED STATES

The basic design criteria and method of proportioning shear connectors for simple span composite bridge members were developed from recent studies at Fritz Engineering Laboratory.(4) These studies suggested that the same criteria are also applicable to the design of shear connectors for continuous composite bridge beams. This was confirmed in a subsequent investigation.(5,6) However, this investigation indicated that for beams in which connectors were omitted from the negative moment regions, overstressing and premature fatigue failures of connectors can occur in the positive moment regions well before the design life of the member has been reached.

On the basis of these findings, the 1969 AASHO Specifications now require that additional anchorage connectors be placed in the vicinity of the dead load inflection points. The purpose of these anchorage connectors is to develop the tension force in the continuous longitudinal slab reinforcement in the negative moment region. The number of connectors required by AASHO is based on both the static and fatigue requirements. To satisfy the fatigue requirements, the range of shear taken by the connectors is computed using the area of longitudinal reinforcement associated with the tee-beam over the interior support and the computed range of stress in the reinforcement due to live loads plus impact.

The value of 10,000 psi suggested by AASHO for the reinforcement in lieu of more accurate computations of stress range was based on the investigation reported in Ref. 6. This permitted a rapid evaluation of the additional shear connectors. In Ref. 6 a two-span continuous composite tee-beam was fatigue tested using a single concentrated pulsating load in each span. The maximum stress range measured in the reinforcement over the center support was about 20,000 psi, which was also equal to the maximum and allowable stress. The probability of attainment of a stress range of this magnitude in an actual bridge was believed to be low and corresponds to the occurrence of a design truck in each of two adjacent spans. It is more likely that the design stress range will correspond to the required number of passages of a single design truck. On this basis the reduced stress range was adopted by AASHO. The number of connectors required is also based on the assumption that the range of shear is uniformly distributed within the group of anchorage connectors.

## 3. PARAMETERS INFLUENCING BEAM BEHAVIOR IN THE NEGATIVE MOMENT REGION

Based on the research results reported in Refs. 5 and 6 a further analytical and experimental research program was initiated at Fritz Engineering Laboratory. The objective of this research program was to develop a comprehensive design procedure for continuous composite beams. The program had four general objectives which were treated as four phases of study. They were: (I) to evaluate the extent to which connectors can be omitted in the negative moment region; (II) to determine the necessary requirements for longitudinal reinforcement in the negative moment region; (III) evaluation of effective width in the negative moment region; and (IV) to examine the feasibility of utilizing prestressing in the negative moment region to improve slab behavior as well as overall composite beam behavior. Some of the significant results of the first three phases will be discussed in this paper.

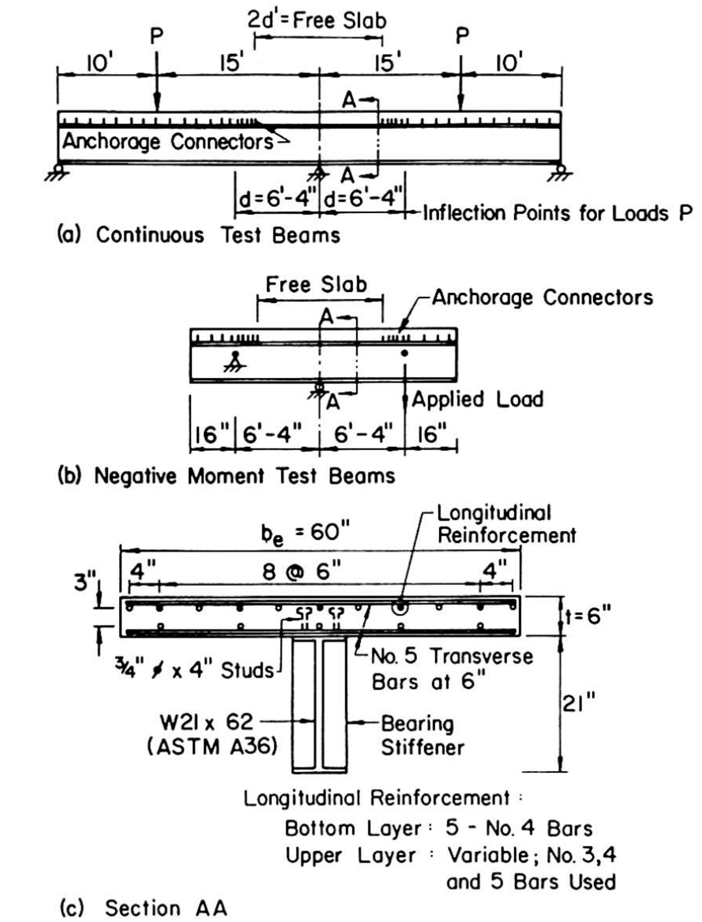



FIGURE 1

In the detailed Investigation of Phases I and II, three parameters were isolated for study and evaluation: (1) Ratio of the length 2d' over which connectors are omitted (called the free slab length) to the length 2d defined by the live load inflection points (Fig. 1a) (2) Reinforcement ratio p, or the ratio of the total area of the longitudinal reinforcement in the negative moment region to the area of the slab cross section ($b_e xt$) expressed in percentage where $b_e$ is the effective width (Fig. 1c) and, (3) Perimeter ratio r, or the ratio of the total perimeter of the longitudinal reinforcement in the negative moment region to the area of the slab cross section.

## 4. THEORETICAL STUDIES

Two methods of analyses were developed for investigating the continuous composite beams. The first analysis considered the composite beam as a series of one-dimensional discrete elements.(7) Nodal points were assumed at each shear connector location and at intermediate points when connectors were omitted from portions of the negative moment region.

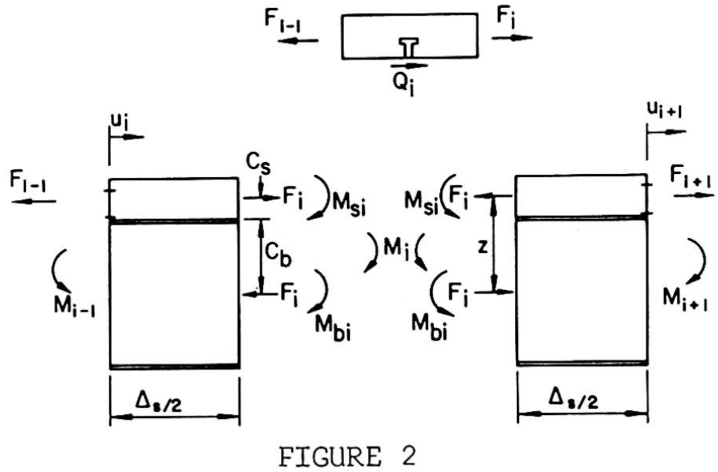



FIGURE 2

The analysis considered both elastic and inelastic behavior of all components; i.e. the steel section, concrete slab, reinforcement, and shear connectors. The analysis provided the distribution of stress resultants at any arbitrary cross section in both the positive and negative moment regions. From the stress resultants on an element such as element i in Fig. 2, equilibrium of the element can be established. Since the elongation of the slab relative to the steel beam is $\int_i(\epsilon_s-\epsilon_b)ds$ and is equal to the slip that occurs in element i, this yields

$$\int_i(\epsilon_s-\epsilon_b)ds = u_{i+1} - u_i \tag{1}$$

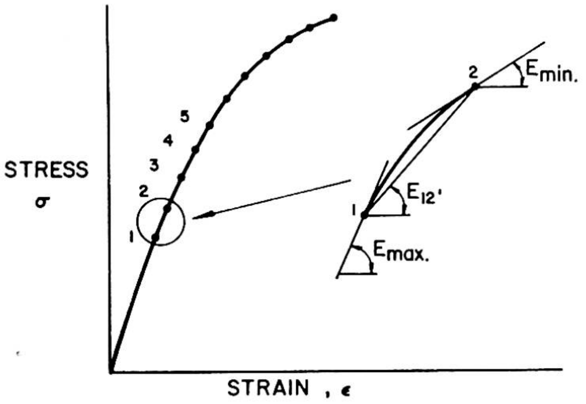
For a linear load-slip relationship, a system of equations in terms of the unknown element forces is therefore obtained as follows:

$$\frac{1}{K} F_{i-1} - \left(\frac{1}{K_i} + \frac{1}{K_{i+1}} + \alpha_i \, \Delta s_i\right) F_i + \frac{1}{K_{i+1}} F_{i+1} = - M_i \, \beta_i \, \Delta s_i \qquad (2)$$

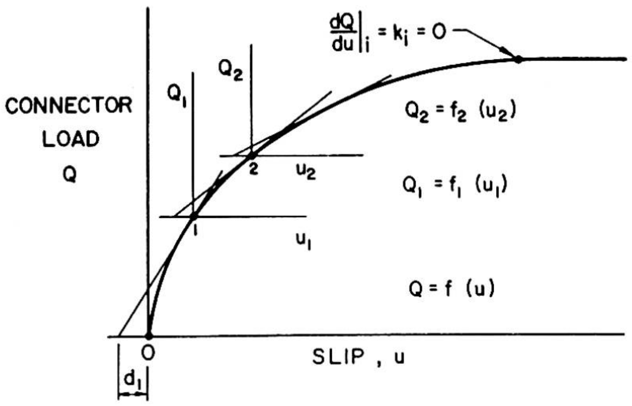
where $\alpha_i = \dfrac{1}{E_s A_s} + \dfrac{1}{E_b A_b} + \dfrac{1}{E_s I_s + E_b I_b}$, $\beta_i = \dfrac{Z}{E_s I_s + E_b I_b}$

Equation 2 is identical to the simultaneous equations that result from Newmark's differential equation(8) for incomplete interaction when expressed in finite difference form.

An incremental load procedure was used to obtain solutions in the inelastic region of various components. Different stiffnesses were assigned to each element by determining the tangent modulus for the concrete and steel at each increment of load as illustrated in Fig. 3a. Incremental strains and stresses were determined for each load increment and were used to approximate the tangent modulus for successive load increments.




(a) Stress-Strain Relationship of Steel or Concrete




(b) Load-Slip Relationship For Shear Connectors

FIGURE 3

The non-linear load-slip behavior of the connectors was also accounted for by the incremental load method. For the load-slip relationship given in Fig.3b, an incremental connector stiffness $K_i$ can be determined for each point on the curve. The displacement $u_i$ can be expressed as

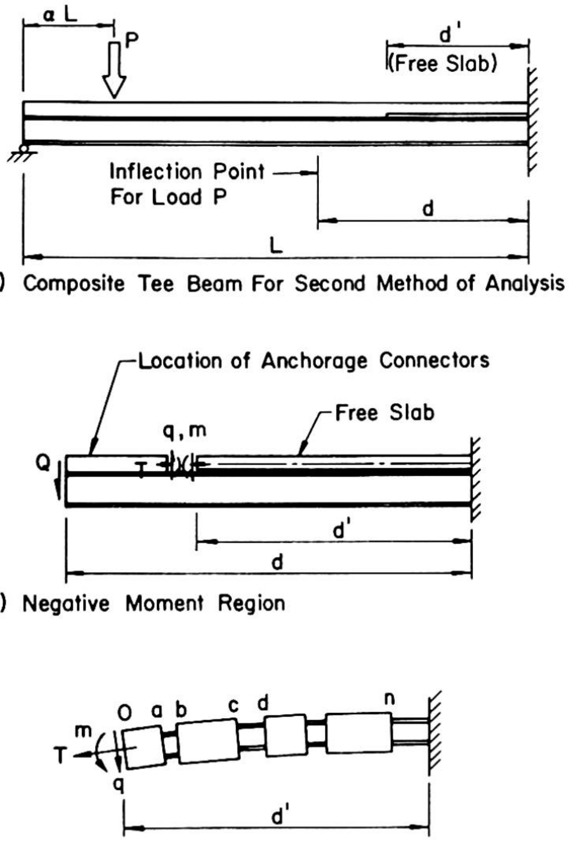
$$u_i = \frac{Q_i}{K_i} - d_i \qquad (3)$$

This value can be readily incorporated into Eq. 1 and results in a correction$(d_{i+1} - d_i)$ that must be added to the right side of Eq. 2.

To account for the cracked slab that existed in the negative moment region, an equivalent uncracked slab with decreased stiffness was assumed. The magnitude of this stiffness was evaluated by the second analysis.

The second analysis considered the stress resultants in the negative moment region of a continuous beam having a length of free slab.(9) It provided a means of estimating the cracked slab stiffness and was more suited to developing a simplified design procedure for continuous composite tee-beams.

The composite tee beam used in this analysis is shown in Fig. 4a. This beam represents a symmetrical two-span continuous beam with symmetrically placed live loads P equi-distant from the exterior supports. The negative moment region of this beam which contains the free slab is shown in Fig. 4b. The beam in this figure is subjected to a live load shear force Q at the inflection point where Q is a function of P. The corresponding stress resultants at the end of the free slab are shear q,




(a) Composite Tee Beam For Second Method of Analysis

(b) Negative Moment Region

(c) Free Slab

FIGURE 4

moment m, and axial force T as shown in Fig. 4.

A first-order flexibility (force) analysis is used in this analysis and expressions derived relating Q (or P) to q, m and T within three domains defined by location of inflection point which are described by the variable $\alpha$ (Fig. 4) which can vary between 0 and 1. The free slab stress resultants cause random transverse cracking of the free slab. For analysis purposes, the free slab is idealized by a series of uncracked slab elements and reinforcing bar segments of random lengths as shown in Fig. 4c. The flexibility of the free slab therefore is a function of the flexibilities of the slab elements o-a, b-c, etc. and reinforcing segments a-b, c-d, etc.

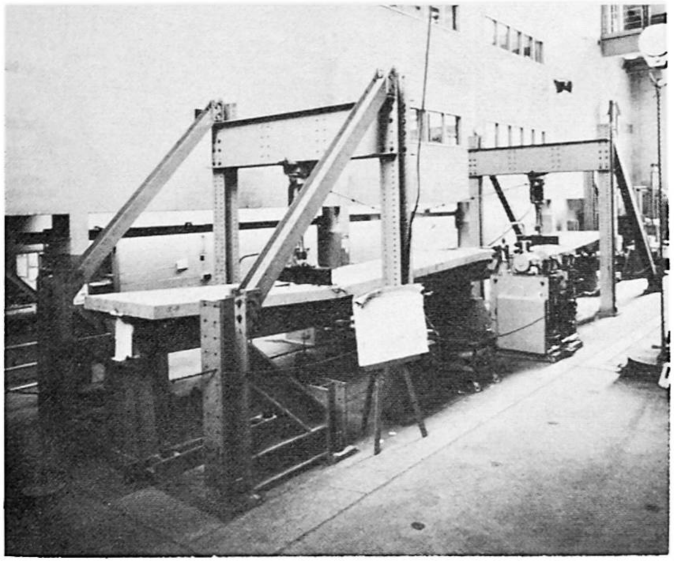
The axial flexibility of the cracked free slab can be expressed in terms of a coefficient of participation, designated $C_1$. This coefficient is the ratio of the total length of uncracked slab elements (o-a+b-c+...etc.) to the length d' of the free slab, Fig. 4c. This coefficient is unity for an uncracked slab and zero for a fully cracked slab. The latter case corresponds to the assumption normally used in a simple tie-bar analysis. The evaluation of the coefficient $C_1$ which is a function of magnitude and position of the load P as well as other factors such as the existence of shrinkage cracks, the length of free slab, the reinforcement and perimeter ratios, and the concrete strength requires experimental evaluation. It is expected that further studies will enable a simple closed form expression for $C_1$ to be formulated.

## 5. EXPERIMENTAL STUDIES

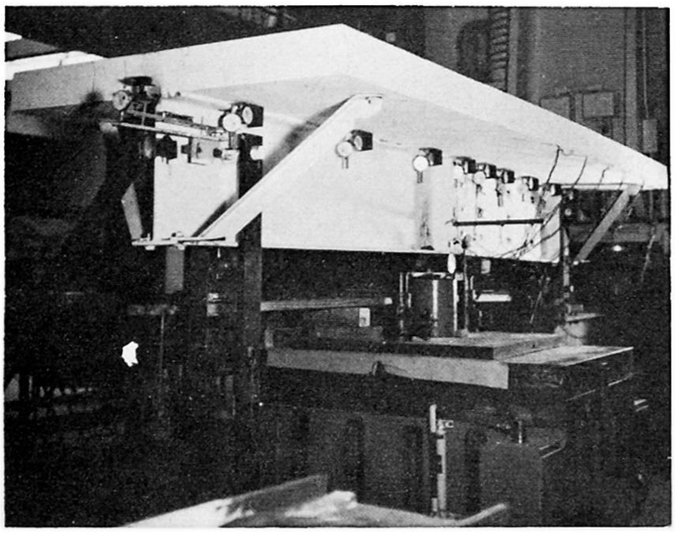
The experimental phase of this investigation extended the work reported in Refs. 5 and 6. The purpose was to (1) evaluate the assumptions used in the analyses previously discussed, (2) determine the magnitudes of the several stress resultants in the negative moment region, (3) evaluate the coefficient $C_1$, and (4) compare actual and predicted behavior.

Two two-span tee beams were tested. Details of these beams are shown in Fig. 1a. These beams were similar to the beams reported in Refs. 5 and 6 except for variations in the three parameters previously mentioned.

In addition, six shorter test beams consisting only of the negative moment regions of the two-span beams were tested. Details of these beams are shown in Fig. 1b. Cross-section details for all test beams are shown in Fig.1c. Two of the shorter beams were made identical to the negative moment regions of the continuous beams. The remaining four beams differed in the length of the free slab, the number of anchorage connectors and the reinforcement ratio for the longitudinal bars.



(a) Continuous Beam



(b) Negative Moment Test

FIGURE 5

The two-span beams were first tested to 2,000,000 cycles of load application that subjected the shear connection to the average allowable values permitted by AASHO for the loading condition. Connectors were also placed so that the nominal tensile stresses in the beam flange would not be critical for fatigue of the base metal adjacent to the connectors. The beams were also tested statically to their maximum load carrying capacity. The shorter beams were also loaded statically to their maximum load. The loads were applied to produce negative bending moment (slab in tension) as shown in Fig. 1b. The beams under test are shown in Fig. 5.

The complete results of this investigation are presented in Refs. 3, 7, 9, 10, 13 and 14. The more significant results will be discussed with emphasis on the parameters affecting the behavior of the negative moment region.

## 6. INFLUENCE OF REINFORCEMENT AND PERIMETER RATIOS

The cracking behavior of the free slab for the six negative moment test beams is shown schematically in Fig. 6 for the working load level which was defined previously. Also shown are the values of d'/d, p and r used in the investigation and the average measured crack widths. The crack patterns in the negative moment regions of comparable continuous beams are similar. It is apparent that the average observed crack width decreases with an increase in both the reinforcement and perimeter ratios. The greatest reduction, however, was associated with increases in reinforcement ratio. Reinforced concrete research has also established the effectiveness of a larger number of smaller bars in minimizing crack widths.(11)

A rule of thumb often used by designers is that for concrete in tension, the yield strength of the reinforcement should not be less than the cracking strength of the concrete. Applying this rule to the test beams would require a reinforcement ratio of about 1.0 percent. This is based on $f'_c$ = 4,000 psi for the concrete and $f_y$ = 40,000 psi for the reinforcement. The test result showing a sharp reduction in crack width for p = 1.02 support the validity of such a rule. On the basis of this and previous investigations, a minimum reinforcement ratio not less than 1.0 to 1.5 percent is recommended in the negative moment region for

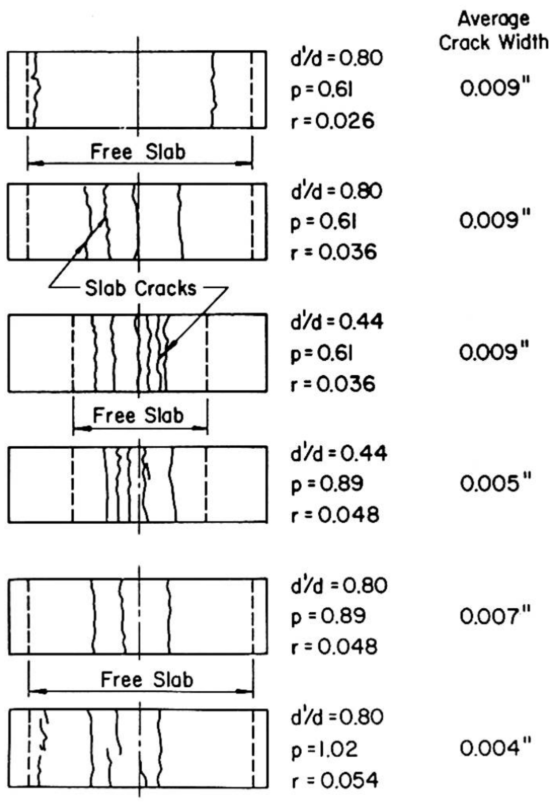



FIGURE 6

effective crack control. A larger proportion (say two-thirds) of this longitudinal reinforcement should be near the top of the slab. It is of interest to note that in the United States the Federal Highway Administration recommends somewhat higher reinforcement ratios for slabs in the negative moment regions of typical continuous concrete T-beam and box girder bridges.(12) The recommended ratios are about 2.5 percent for the T-beam and about 4 percent for the box girder bridges. The respective ratios for top slab reinforcement alone are about 2 percent and 3.5 percent.

## 7. INFLUENCE OF THE FREE SLAB

In the investigation reported in Ref. 6, the ratio d'/d varied from 0.28 to 1.0. For this investigation, the ratio d'/d was chosen as 0.44 or 0.80 for all test beams. The smaller value corresponds to placing connectors approximately at the point where the allowable stress in the base metal is governed by the AASHO fatigue requirements.(2) The larger value corresponds to placing the required anchorage connectors within the limits specified by AASHO. (All anchorage connectors are assumed to be placed on the negative moment side of the inflection point.) Examination of Fig. 6 shows that the effect of the reduced free slab length is to reduce the average crack widths. These beams also exhibit a larger number of narrower cracks compared with beams with the longer free slabs.

One of the purposes of the experimental phase of the investigation was to evaluate the coefficient of participation of the free slab $C_1$. The value of $C_1$ was influenced considerably by the level of axial force T carried by the free slab which is a function of the applied loads P. For loads close to zero, $C_1$ varied from about 0.80 to 0.90 for all test beams. As the loads increased, the values of $C_1$ decreased nearly linearly. For most of the beams, $C_1$ varied from 0.25 to 0.60 when the maximum load carrying capacity was reached. The minimum value of $C_1$ was 0.25 for all tests. At the assumed working load level, $C_1$ varied from 0.40 to 0.77 with an average value of 0.60 for those beams with reinforcement ratios of 0.89 and 1.02.

These studies indicate that the behavior of the free slab at working loads can be adequately represented by a tie bar whose length is about 60 percent uncracked concrete and 40 percent reinforcing steel. Such a simplified model can be used to calculate the average stress range in the longitudinal reinforcement. A similar calculation was performed for the continuous test beams. The allowable stress range for the reinforcement in these two beams for 2,000,000 cycles of load application is about 20 ksi.(9) The predicted stress range for the continuous beam with a longitudinal reinforcement ratio of 0.6% and a calculated value of $C_1$ equal to 0.59 was approximately 25 ksi. On this basis fatigue failure of reinforcing bars could be expected and was observed at about 1,000,000 cycles. On the other hand, the predicted stress range for the continuous composite beam with a longitudinal reinforcement ratio of 1.02% and a calculated value of $C_1$ equal to 0.46 was about 17 ksi. No fatigue failures of the reinforcement in this beam occurred prior to reaching 2,000,000 cycles of load application.

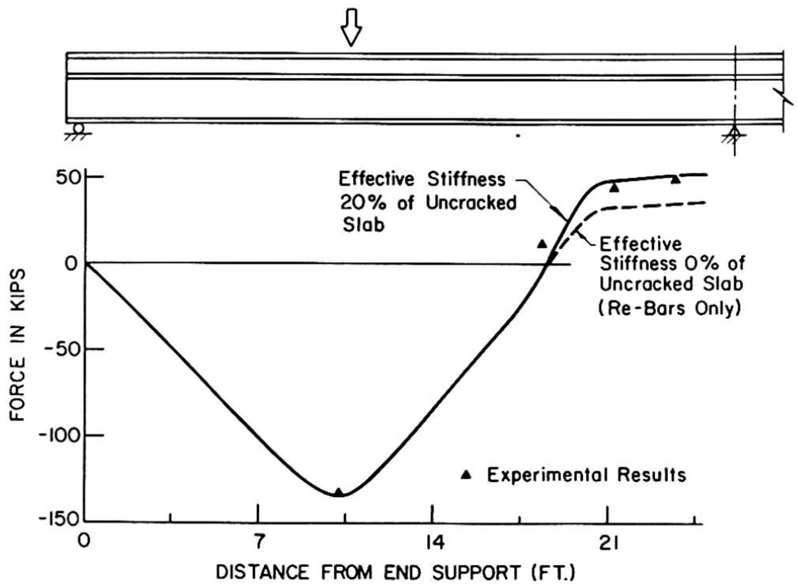



FIGURE 7

The above results were confirmed using the one-dimensional discrete element analysis previously described. A coefficient of participation $C_1 = 0.6$ is equivalent to a free slab with an effective axial stiffness equal to about 20% of the uncracked slab. The discrete element analysis predicted a variation in slab force throughout the length of the two-span composite beam with 1.02% longitudinal reinforcement which is shown in Fig. 7. In the negative moment region two conditions were evaluated. One considered the stiffness of the reinforcement alone (the dashed line) and the second used the effective stiffness. The predicted slab forces are compared with the experimental results and show good agreement when the effective stiffness was considered.

The simplified model mentioned above was used to calculate the stress range in the free slab reinforcement for several typical three span bridges considering the passage of an AASHO HS20-44 truck or equivalent lane loading.(9) Main spans were varied from 50 to 240 ft. and $C_1$ was varied from 0.4 to 0.8. The reinforcement ratio P was taken equal to 1.0. The resulting stress ranges varied from about 7.8 ksi for the 240 ft. main span with $C_1 = 0.80$, and lane loading to about 6 ksi for the 50 ft. main span with $C_1 = 0.80$ and truck loading. The AASHO stress range provision of 10,000 psi based on test results reported in Ref. 6 for a 25 ft. span thus appears to be conservative for most practical bridge beams when used to proportion anchorage shear connectors.

## 8. INFLUENCE OF CONNECTOR CONCENTRATION

The influence of the degree to which connectors are concentrated or spread out in the negative moment region on the free slab behavior, and the individual connector behavior, is quite complex. Although additional analytical studies are needed, some tentative comments are possible based on the investigations reported in Refs. 7 and 9. These investigations indicate that as the free slab length decreases, the total shear taken by the anchorage connectors may increase or decrease depending in part on the reinforcement ratio. A decrease in free slab length tends to increase the axial force carried by the free slab because of the increased stiffness of the negative moment region. However, this results in a greater degree of cracking of the free slab (Fig. 6) which has the effect of reducing the value of $C_1$. A decrease in $C_1$ will in turn result in a decrease in the axial force in the free slab.

The predicted distribution of force acting on individual connectors confirmed that connectors adjacent to the free slab were more highly stressed than expected if the concrete slab were neglected altogether. Design procedures must account for this force if satisfactory connector behavior is to be provided.

Investigations also indicate that the assumption of uniform distribution of stress range to connectors within the anchorage connector group is fairly realistic. This appears to result from the redistribution of forces which occur within the connector group during cyclic loading.

## 9. DESIGN RECOMMENDATIONS

These studies have confirmed the desirability of increasing the amount of longitudinal reinforcing steel in the slab over the negative moment region to at least 1% of the cross-section area of the concrete. It is desirable for most of this reinforcement to be placed near the top surface of the slab. This will assist with controlling the cracking behavior of the negative moment region and provide a more favorable stress state in the longitudinal reinforcement.

The magnitude of slab force was greater than predicted from the steel reinforcement alone when shear connectors were omitted from the negative moment region. The connectors placed near the points of contraflexure and the reinforcement were overloaded. This condition was partially corrected with an increase in the amount of longitudinal reinforcement to 1% of the cross-section area of the concrete.

If it is desired to make a more accurate estimate of the stress $f_r$ in the longitudinal reinforcement, methods for making these estimates have been developed. These studies indicate that the longitudinal slab force can be reasonably approximated from 20% of the uncracked area of the concrete slab in the negative moment region when the percentage of reinforcement is 1%. Alternately, the free slab can be represented by a tie bar whose length is composed of 60% of the uncracked concrete slab and the remaining 40% the longitudinal reinforcement.

The study has confirmed the suitability of other provisions of the AASHO specifications. Applying the same effective width provisions to both the positive and negative moment regions appears reasonable.

## 10. ACKNOWLEDGMENTS

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## 12. SUMMARY

This paper describes analytical and experimental studies that were undertaken to evaluate the extent to which shear connectors can be

omitted in the negative moment (slab in tension) regions of continuous composite steel-concrete bridge beams. In addition, the necessary requirements for continuous longitudinal reinforcement in the negative moment region was examined.

The major parameters examined include the free slab length (length of negative moment region over which shear connectors are omitted), the area and perimeter of the longitudinal reinforcement, and the influence of connector concentration at the ends of the free slab.

These studies have indicated that satisfactory performance results if the continuous longitudinal reinforcing steel is at least 1 to 1.5% of the cross-section area of the free slab and sufficient anchorage connectors are placed near the points of contraflexure to develop the longitudinal reinforcement.

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