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TEST RESULTS AND DESIGN RECOMMENDATIONS
COMPOSITE BEAMS FOR BUILDINGS

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by

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ABSTRACT

The use of composite steel and concrete beams in building construction has been restricted somewhat because of a lack of specifications which would permit the most economical use of this type of member. An investigation of the minimum requirements for composite members subjected to static loads has been carried out. The results of this investigation and consideration of other work on composite members has resulted in the establishment of minimum requirements for shear connectors.

The design recommendations presented are based upon the conditions prevailing at ultimate load rather than elastic considerations as was the case with previously existing specifications. The ultimate strength of different types of commonly used mechanical shear connectors is first established. The minimum strength of shear connectors required to attain the ultimate strength of a member was derived from tests of composite members. The ultimate connector strength is then used to establish the number of connectors required for any composite member.

The design of composite members on the basis of ultimate strength is also recommended. This design procedure is simplier than elastic design procedure and offers certain additional advantages in terms of maximum economy and a uniform factor of safety with respect to the ultimate strength of the member.

1. INTRODUCTION

This report summarizes the results of a research program begun at Lehigh University in 1959 to study composite steel and concrete members. The purpose of this research program was to provide a more rational basis for the design of composite building members where only static loading occurs. Both elastic and ultimate strength design concepts were considered in analyzing the test results. However, the tests were planned so that the ultimate strength of members could be carefully investigated.

In this paper attempts have been made to describe the maximum strengths of shear connectors by either theoretical or empirical equations. Based on these maximum strengths, recommendations for the ultimate strength design of composite beams are presented. Recommended values of shear connector forces for use in allowable stress designs are also presented. The recommendations presented herein are largely based upon the results of testing twelve simple beams and one continuous beam. Nine pushout type tests and numerous supplementary tests were also performed. The results of these tests have been reported previously[3,4] and only a summary of results is presented here. A review of other research work in the field of composite design has also been presented to support the conclusions reached.

The provisions of the "Tentative Recommendations for the Design and Construction of Composite Beams and Girders for Buildings"[14] by Joint ASCE-ACI Committee on Composite Construction are used as a basis for discussing the results of this research program. Revisions and modifications of these "Recommendations" are suggested based upon actual test results. In addition to suggested modifications of the elastic design provisions, a new section providing the ultimate strength design and permitting plastic design of certain members is proposed.

The most important results from the point of view of more rationality in design and greater economy of composite construction are those pertaining to strength and arrangement of shear connectors. The addition of an ultimate strength procedure for design provides a useful tool for the design of composite members and provides also a means for attaining greater economy with this type of construction.

The specific problems investigated in connection with the testing program are as follows:

(1) Interaction created by bond and friction.

(2) Strength of shear connectors.

(3) Influence of slip on ultimate strength.

(4) Deformation and spacing of shear connectors.

(5) Application of plastic design concepts to continuous beams.

The conclusions reached from the study of each of these individual problems are given in the discussion of each topic. The design recommendations based upon these conclusions are presented in the form of proposed design specifications for ultimate strength design at the end of this report.

Details of all test members considered and descriptions of the tests performed are given in Tables 1 through 7. Figure 1 indicates the loading conditions which were used in the various beam tests.

2. INTERACTION BY BOND AND FRICTION

During tests of composite beams with mechanical shear connectors, it has been observed that a definite bond failure occurs during testing. This happens at different stages of loading even when comparing members identical in cross section and span, but it often occurs in the vicinity of design load. Observations also show that shear connectors carry almost no load before bond failure, and that immediately after bond failure, the load carried by shear connectors is suddenly increased. It has been observed that bridges in service have exhibited some composite action even though they were constructed without shear connectors.

Since no test results evaluating the amount of interaction which could be attained from bond and friction could be found, two members B1 and B2 were fabricated and tested[3] to evaluate the effectiveness of bond and friction in transferring shear. These members were similar in cross section to other members tested (See member B3 in Table 1) except that they were fabricated without mechanical shear connectors. Member B1 was loaded by hanging loads from the steel member and member B2 was tested by loading on top of the slab in the usual manner. This made possible an evaluation of bond alone as a shear connection in the case of member B1. Bond and friction due to applied loads as a shear connection were evaluated in the test of member B2. Figure 2 gives the results of tests B1 and B2 in the form of load versus deflection curves comparing the performance of the members to the predicted ultimate strength of the composite member and the ultimate strength of the steel beam alone. No attempt to artificially destroy or increase bond by treating the top surface of the steel beam was made. However, means to prevent accidental breaking of bond due to handling were employed in the case of both members. Unfortunately, bond failure from shrinkage of the concrete in the slab took place in both members prior to testing. The load deflection curves as well as other data from the tests indicate that there is no interaction due to bond alone (Member B1) and very little interaction due to friction (Member B2). Member B1 exhibited complete separation of slab and steel beam at the maximum load shown for this test. The ultimate strength of member B2 was 107% of the plastic moment of the steel beam alone. If one assumes a coefficient of friction of 0.5 resulting in the maximum possible compressive force in the concrete slab being half of the applied shear and the stress distribution given in Figure 10 for calculation of "modified ultimate strength", the ultimate strength of member B2 can be determined theoretically. The results of such a calculation agree with the test value within 2%.

It must be concluded from these tests that interaction due to bond can only be attained when shrinkage of the concrete slab is restrained by some type of shear connector and when the slab and beam are prevented from separating. Observed interaction in bridge decks without shear connectors must be attributed to the effect of a relatively heavy deck in providing relatively large friction forces as well as a high percentage of reinforcing steel in the deck slab which restrains shrinkage of the concrete. Since none of these conditions are normally present in building construction, the use of bond and friction in providing interaction must be ignored and all horizontal shear between concrete slab and steel beam must be carried by properly designed shear connectors. In evaluating test results, however, it must be remembered that the strength of mechanical connectors must be considered with the shear transfer by friction included.

3. ULTIMATE STRENGTH OF SHEAR CONNECTORS

The results of tests performed by several investigators have been considered along with the members tested under this program. The members which were tested as part of this investigation are listed under references (3) and (4) in Table 1. Only these members were planned with the intention of developing design rules for ultimate strength design of shear connectors.

It is essential that the ultimate strength of a section should not be reduced by secondary failure of shear connectors at loads below ultimate bending strength. Existing empirical design rules for the design of shear connectors have proven satisfactory for elastic design. These rules are also safe for ultimate strength design even though it is necessary that the strength of the shear connection must provide interaction for loads up to the ultimate load. However, an analysis of the ultimate strength of shear connectors is necessary so that the resulting designs can be made both economical and safe. The fact that the shear connectors must provide virtually all of the resistance to horizontal shearing forces at loads near ultimate load makes it necessary to develop design rules which will prevent failure of members due to shear connector failure. The strength of shear connectors must be known before the factor of safety of a composite member can be stated with any degree of certainity.

The load-slip characteristics of different types of shear connectors have been determined by means of pushout tests by many investigators. Generally the ultimate strength of the connector has been ignored, and the strength of the connector has been taken as the force provided at a certain amount of slip. The ultimate strength of connectors is now proposed as the basis of design. The strength of shear connectors in beams as well as pushout specimens has been used in developing this method.

In attempting to evaluate and compare the results of beam and pushout tests, basic differences in the performance of shear connectors in beams and slabs must be considered. Stresses, deformations, and cracking of pushout test specimens are determined by the magnitude of loads on the shear connectors alone. In beams, stresses, deformations, and cracking of the concrete slab are determined primarily by bending stresses rather than by the magnitude of loads on the shear connectors. Therefore, variables such as concrete strength and percentage of steel reinforcement in the slab which seem to produce considerable effect on connector strengths in pushout specimens may not be of equal importance in the case of beams. Also friction due to applied loads is only present in beam tests. In Tables 4 through 7, test values for ultimate connector strength obtained from both beam and pushout tests are listed beginning with the highest test result. This is done for three types of commonly used connectors--studs, spirals, and channels.

3.1 Apparent Connector Force in Beam Tests

The apparent connector force in beam tests is obtained by first calculating the compressive force in the concrete slab at ultimate load. The stress distribution at ultimate load is based upon the crushing strength of concrete and the yield strength of steel.

The stress distribution at ultimate moment for Case I having the neutral axis located in the concrete slab is shown in Figure 3. The total tension and compression forces are given by equations 1 and 2.

$$T = A_s f_y \qquad (1)$$

$$C = 0.85\ f'_c ba \qquad (2)$$

Since $C = T$ the depth of the compressive stress block is

$$a = \frac{A_s f_y}{0.85 f'_c b} \qquad (3)$$

This defines the distance e from the centroid of the steel area to the centroid of the concrete area as

$$e = \frac{d}{2} + t - \frac{a}{2} \tag{4}$$

The ultimate moment then is

$$M_u = Te = Ce \tag{5}$$

When the neutral axis falls within the steel beam, the following equations must be used to determine ultimate moment. This is referred to as Case II.

$$C = 0.85\ f'_c bt \tag{6}$$

$$T = C' + C \tag{7}$$

$$C' = \frac{A_s f_y - C}{2} \tag{8}$$

$$M_u = Ce + C'e' \tag{9}$$

The distances e and e' must be found by considering the stress distribution in the steel member along with the geometry of the section.

The assumptions upon which Figure 3 is based are (1) a fully plastic state of stress is present in both steel beam and concrete slab, (2) there is complete interaction between beam and slab at ultimate load and, (3) the concrete slab resists no tension. The fully plastic state of stress is taken as $0.85f'_c$ over a depth of slab necessary to resist the compressive force and f_y over the entire steel section. The theoretical ultimate moment for test members is given in Table 2, Column 5 as M_u. The moment M_u' given in Table 2, Column 6 is a theoretical ultimate moment for the member when the shear connection is inadequate. The theory for determination of M_u' is given in Art. 4.

The force per connector for members at ultimate load is determined from the equilibrium of forces shown in Figure 4 by dividing the compressive force C in the slab by the number of connectors between the sections at which ultimate moment and zero moment occur. Many of the tests reported were stopped before ultimate moment was reached (See Table 2, Column 3 and 4). In such cases it is first necessary to find the value of C which exists at the maximum test load by use of Figure 5. This curve was constructed for the cross section of members B3 through B13 and shows the resultant compressive force in the slab plotted against the applied moment. This curve was obtained by a theoretical analysis of strains and stresses in the steel beam and concrete slab after first yielding of the steel beam. Since Figure 5 is non-dimensional, it can be used for any similar composite member with little loss of accuracy for predicting the value of C at a load less than ultimate load. The connector forces for beam tests given in Tables 2, 4, 6 and 7 and which are plotted in Figures 6, 7, and 8 were obtained in this manner. It will be shown later that Figure 5 is only valid when the number of shear connectors provided is adequate to develop the compressive force C at ultimate load. When there are not enough connectors provided, the maximum value of C cannot exceed the strength of the connectors. Since the shear connector force is dependent upon the loading condition, Figure 5 may be valid for one loading condition but not for another loading condition on the same member. This must be kept in mind in evaluating test results when a member was tested several times with different loading conditions. No attempt to separate shear force transferred by bond or friction has been made in the case of beam tests. This force, the magnitude of which can be roughly estimated from the test of member B2, is therefore part of the connector force given for beam specimens and is referred to as "apparent connector force".

3.2 Stud Shear Connectors

Considering the tests of stud connectors first, it is found in studying Table 4a, representing tests of 1/2" connectors, that the force per connector at failure of the connector may range from 6.8 kips per connector in the case of pushout P7 to 17.8 kips per connector in the case of beam B6. These values may be compared with the tensile strength of the connector material which may range from 13.0

kips per connector to 13.9 kips per connector as given in Table 5. More than half of the test results given in Table 4a fall above the minimum material strength of 13.0 kips per connector and all of the beam test results except test B11-T13 fall above 13.0 kips per connector. Connectors which failed in beams have always failed in tension rather than shear indicating that the ultimate strength of the connector as used in a beam can be expected to reach the material ultimate strength. The ultimate strength of the connector material was exceeded in a sufficient number of pushout specimens to justify its use. Pushout tests of lower results must be attributed to the difference in performance of connectors in beams and pushout specimens and the lack of a standard pushout specimen from which consistent results can be obtained. Comparison of Tables 4b, 4c, 4d and 4e with Table 5 also support the conclusion that the ultimate tensile strength of the connector material can be used as the ultimate strength of a shear connector.

Other reasons for low connector strengths cited by previous investigators have been short connectors and low concrete strengths. These factors must not be overlooked in specifying the ultimate strength of connectors. For this reason, the form of the empirical formulas used for determining the strength of shear connectors in the past has been retained and the coefficients have been changed to give the ultimate strength of shear connectors. In Art. 10 for formulas for stud connectors proposed for use in specifications are

$$q_u = 930\ d_s^{\,2}\sqrt{f'_c} \qquad (10)$$

for h/d_s larger than 4.2 and

$$q_u = 220\ hd_s\sqrt{f'_c} \qquad (11)$$

for h/d_s less than 4.2

where q_u is the ultimate strength of the shear connector. In addition, the ultimate strength per connector should in no case be taken as greater than the ultimate tensile strength of the connector material. The above formulas were obtained by plotting the test results and fitting these curves to the data as shown in Figure 6 using a concrete strength of 3000 psi and non-dimensional coordinates. It is felt that these formulas are satisfactory for studs up to 1 inch in diameter. Larger studs have not been considered in this work.

The approximate working stress for stud connectors as specified for f'_c = 3000 psi in the 1961 AISC specifications and the ASCE-ACI recommendations is indicated by lines on Figure 6.

3.3 Spiral Shear Connectors

Tests which provide information on the ultimate strength of spiral shear connectors are listed in Table 6 along with results of tensile tests of the spiral material. In evaluating these results, the form of the formula for connector strength in current use has been retained and the coefficient changed to give the ultimate strength of the connector. The formula therefore becomes

$$q_u = 8000\ d_s\ \sqrt[4]{f'_c} \qquad (12)$$

in which q_u is the ultimate strength per turn of spiral. This formula was obtained by fitting a curve to the test results plotted in Figure 7. It is assumed in the case of spiral connectors that the weld develops the ultimate strength of the spiral. The ultimate strength of a turn of spiral is therefore taken as $2A_s f'_s$ as a maximum. This value also fits test results well except that it appears to be conservative for 1/2" and 5/8" spirals. However, in the case of stud connectors, some of the test results were also higher than the strength of the material.

The approximate working loads for spiral connectors as given in the 1961 AISC specifications and the ASCE-ACI recommendations are indicated by lines on Figure 7.

3.4 Channel Shear Connectors

The formula for the ultimate strength of channel connectors is given as

$$q_u = 550\,(h + 0.5t)\,w\,\sqrt{f'_c} \qquad (13)$$

in which q_u is again the ultimate strength per connector.

The parameters t and w in this formula are web thickness and length of the channel as in previous formulas for channel connector strength. In this equation h is taken as the average flange thickness whereas previously, h was taken as the maximum flange thickness. Either value may be used by selecting the proper coefficient in the formula. Average flange thickness is more convenient to use in design because it can be obtained from any handbook and is the same for all channels of a given size regardless of the manufacturer. The value of the coefficient of 550 was determined by fitting a curve to test results in terms of load per inch of connector versus the factor $(h + 0.5t)\sqrt{f'_c}$ as shown in Figure 8. A curve which gives satisfactory values of connector strength for all sizes of standard channels having depths of 3" through 5" was selected. The tests used in determining this formula are listed in Table 7. It will be seen that concrete strengths ranging from 2070 psi through 6480 psi are included in the tests and that several sizes of channels are also included. It is believed that this data is sufficiently representative of practice so that no restrictions need be placed on the use of this formula for standard channel sections, but the formula should not be used to determine ultimate strength of sections heavier than standard channel sections.

The approximate working loads for channel connectors as given in the 1961 AISC specifications and the ASCE-ACI recommendations are indicated by lines in Figure 8.

4. ULTIMATE MOMENT OF COMPOSITE MEMBERS WITH INADEQUATE SHEAR CONNECTORS

In considering the ultimate moment of composite members, it is not necessary to consider whether or not the interaction between steel beam and concrete slab is complete or incomplete as defined in the literature on elastic analysis. The interaction between slab and beam is considered complete if the theoretical ultimate strength is attained. However, to avoid confusion between elastic analysis and ultimate strength analysis, the terms "complete" and "incomplete interaction" are not used in discussing ultimate strength. In place of these terms, the terms "ultimate moment" and "modified ultimate moment" are used. Beams which because of weak shear connectors do not reach theoretical ultimate moment when tested to failure are considered from the point of view of their "modified ultimate moment". This is analogous to the theory of incomplete interaction in elastic design.

It was observed in the testing of some members of this program that the ultimate moment as predicted by the equilibrium of forces shown in Figure 3 and described in Art. 3.1

was attained in some tests and not in others. Table 2 lists the beam tests giving the predicted ultimate strength (M_u) and the maximum test moment including dead load moment (M). It was observed that in some members in which M_u was not reached the member failed by crushing of the concrete slab while in others of similar cross section and equal concrete strength, the shear connectors failed. Some of these members exhibited "apparent connector strengths" considerably above the tensile ultimate strength of the stud connectors, and also considerably above the maximum strength of the same connectors in pushout tests. Since the effect of friction had been investigated in the testing of beam B2, it is possible because of friction to conceive of stud connectors, for instance, developing slightly greater forces in beams than in pushout tests where there is no external force which tends to create friction forces between steel and concrete. However, it is not possible to explain apparent connector forces 25% to 30% greater than maximum pushout results as were observed in the case of beams B6-T2 and BIII-T6.

In view of the above problem it is necessary to consider a theory for stress distribution in the cross section other than the one indicated in Figure 3. The stress distribution given in Figure 3 is entirely independent of the shear connector strength. It has been tacitly assumed that the number of connectors necessary to attain ultimate strength has been provided. The stress distribution which actually occurs in the case of members with weak shear connectors must, however, be based upon shear connector strength.

The terms "adequate" and "inadequate" connector strength are hereafter used. The shear connector strength is termed "adequate" if the number of connectors resisting the compressive force C in the slab, times the ultimate tensile strength of the connector material is equal to or greater than the compressive force C in the case of stud connectors. For channel and spiral connectors empirical curves derived from test results are used to obtain an ultimate strength of connectors for analysis. If the ultimate strength of all connectors in the shear span, hereafter referred to as $\sum q_u$, is less than C, the shear connector strength is termed "inadequate".

Using these definitions a non-dimensional plot of M/M_u versus q_u/C was made using the following values for ultimate connector strength:

q_u for 1/2" diameter stud = 13.9 kips per connector
q_u for 3/4" diameter stud = 31.2 kips per connector
q_u for 4 inches of 3 [4.1 = 47.3 kips per connector
q_u for 4 inches of 4 [5.4 = 62.9 kips per connector

To simplify compariosn of results, these values were used for all members, even though the application of proposed formulas might result in slightly different values for some members. All test results for the final test to failure for beams listed in Table 1 are plotted on this graph given as Figure 9. Tests which were stopped prior to failure were not plotted unless ultimate load was attained because of the difficulty in evaluating the point at which the test was stopped. Points for members B21S and No. 2 were not plotted because they fall to the right of the limits of the graph (See Table 3). Two straight lines which fit the data points empirically have been drawn. These lines intersect at the point $M/M_u = 1.0$ and $\Sigma q_u/C = 1.0$. For values of $\Sigma q_u/C$ greater than 1.0, the values of M/M_u for all points are equal to or greater than 1.0. For values of $\Sigma q_u/C$ less than 1.0, a line fitting the data points can be expressed by the equation

$$M/M_u = (\Sigma q_u/3C) + 0.66 \qquad (14)$$

Therefore, it appears that the maximum connector force is actually very close to the values assumed above and that the stress distribution in the member is different than that assumed in Figure 3 when members which have inadequate shear connector strength.

The stress distribution proposed for calculation of the ultimate moment of members having less than adequate shear connector strength is given in Figure 10.

The use of this stress distribution to predict the ultimate load gives satisfactory results. The predicted load-deflection curve for beam B6 is shown along with the actual load deflection curve in Figure 14.

It is assumed that the maximum compressive force in the slab is limited to the value

$$C = \Sigma q_u \qquad (15)$$

and that the steel section is stressed to f_y in both tension and compression so that

$$C + C' = T \qquad (16)$$

where C is the compressive force in the concrete slab, C' is the compressive force in the top flange of the steel beam, and T is the tensile force in the steel beam.

The depth at the compressive stress block in the concrete slab is defined by

$$a = \frac{\Sigma q_u}{0.85 f'_c b} \qquad (17)$$

The compressive force C' in the steel may be found from

$$C' = \frac{A_s f_y - \Sigma q_u}{2} \qquad (18)$$

and the modified ultimate moment, M'_u, is computed by equation (19)

$$M'_u = Ce' + C'e''$$

Both e' and e" must be determined from the stress distribution and geometry of the section. The ultimate moment calculated from this theory is termed "modified ultimate moment", M'_u.

The values of M_u' for all tests to which the theory applies are given in Table 2 and values of M/M_u and M/M_u' are given in Table 3. The test results viewed from the point of view of M/M_u' instead of M/M_u show better agreement with theory. Many of the tests for which M/M_u' is less than 1.0 were concluded by failure of the shear connectors. The value of M_u' may be higher than the test results because (1) connector strength assumed was too high for that member, and (2) connector failure may have been premature because of large deformations and cracks in the slab produced in previous tests by loads near ultimate load.

It is now obvious why a maximum value of connector force to be used in design was established. The minimum number of connectors to be used between the sections of maximum and zero moment are determined as $C/\Sigma q_u$. The test results indicate that this rule is not only necessary but sufficient to insure that the ultimate moment of the member will not be reduced by shear connector failure.

5. INFLUENCE OF SLIP OF SHEAR CONNECTORS

Previously, investigators have been greatly concerned about the effect of slip on the completeness of interaction between slab and beam. It was shown in the previous article that members having adequate shear connector strength can be expected to develop the full value of ultimate moment as determined assuming complete interaction.

Figure 11 shows the amount of end slip measured in tests of three members having adequate but different shear connector strengths. This clearly shows that the amount of slip is related to the number of shear connectors. However, the theoretical ultimate moment is attained in the tests of all three members even though the amount of slip exhibited by the member having the fewest number of connectors is more than four times the amount of slip exhibited by the member having the most shear connectors. Since all three of these tests were on identical members and with identical loading conditions, it can be concluded that slip does not affect the ultimate moment of a composite member provided that the shear connector strength is adequate. The magnitude of the slip is the amount of deformation required of each group of connectors to develop the compressive force, C, in the concrete slab. The magnitude of slip for each member could have been estimated from a load versus slip curve of a pushout specimen having the same type of connector.

Next the effect of slip on the load-deflection curves of the same three members is considered. These curves are given in Figure 12. No difference in the load-deflection curves can be discerned from these test results. It may therefore be concluded that for members having greater than adequate connector strength the influence of slip upon the load-deflection curve of a member is negligible.

The load versus slip characteristics of identical beams under identical loading conditions are given for tests B9-T2, B8-T2, and B6-T2 having ratios of $\Sigma q_u/C$ of 1.21, 0.988, and 0.473 in Figure 13. The load-deflection curves for the same tests are given in Figure 14. It will be noticed that the difference in end slip between test B9-T2 and B8-T2 is only small as compared with the differences between BI-T3 and BIII-T3 in Figure 11. However, this small difference has resulted in a noticeable effect on the load-deflection curves for the two tests as given in Figure 14. Test B6-T2, having very weak shear connector strength, exhibited a large amount of slip and also a substantially different load-deflection curve as compared with tests B9-T2 and B8-T2. However, the difference in the load-deflection curves for members having less than adequate shear connector strength can be explained by the fact that the change in stress distribution at high loads necessitated by the weak shear connectors has resulted in a concrete slab which is not fully effective as a cover plate and therefore a reduced value of the effective moment of inertia of the composite member results.

This reduced value of the effective moment of inertia is caused by yielding of a portion of the steel cross section and a reduction in the apparent value of E_c/E_s.

It will be noticed in Figure 14 that the load-deflection curves coincide up to a moment value of $M/M_u = 0.59$. Referring to Figure 5, this results in a value of $C/C_u = 0.39$ which is 82.5% of the strength of the shear connectors. The slip in these members with a weak shear connection permits the stress distribution of Figure 10 to form at loads near ultimate load, and this results in reduced value of effective moment of inertia and a change in the load-deflection curve.

Figure 15 shows strain measurements made during tests B5-T4 and B6-T2 at loads near the capacity of the members. These measurements verify that the stress distribution at high loads is similar to that assumed in the calculation of M_u'.

6. DEFORMATION AND DISTRIBUTION OF CONNECTORS

The amount of deformation of shear connectors in a composite beam is a function of the following:

(a) type of connectors used

(b) loading condition

(c) concrete strength

(d) number of connectors

The type of connector used has no effect upon the ultimate strength of the member provided that sufficient connectors are used and the connectors are able to prevent separation of slab and beam. It has been assumed that connectors are able to deform sufficiently to redistribute load, and this is a necessary requirement. Slip is therefore to be expected and the magnitude of slip is not important if strength requirements are satisfied.

It has been shown in Figure 11 that the amount of slip for beams having different values of connector strength varies for identical beams under identical loading conditions. Since these beams are quite small, it becomes necessary to investigate further to show that the amount of deformation is not a function of the span and cross section of a member.

Figure 16 shows the relationship between M/M_u and maximum end slip for three members having span lengths of 10, 15, and 30 feet. Only one of these three members, the bridge member, was actually tested to failure.* The value of $\sum q_u/C$ for all three members is approximately equal to 1.00. The amount of slip for member BIII-T3 is less than that of the other two members, but this is undoubtedly due to the fact that the concrete strength of this member is 5560 psi, whereas the concrete strength of the longer members is 3337 psi for member B8 and 3280 psi for the bridge member. The curves for B8-T3 and the bridge member are nearly identical even though the bridge member has twice the span and a considerably different cross section. This demonstrates that slip is not a function of span length or cross section.

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* The "bridge member" described in Table 1 and Ref. 6 was considered near enough in its details to typical building members, to warrant its inclusion in this study.

The amount of deformation of a group of shear connectors does not depend entirely upon the magnitude of the compressive force C at the plastic hinge. At each point along the shear span, the number of connectors between that point and the section having zero moment must be equal or greater than the number required to develop the force C in the slab. Therefore, the loading condition is an important factor in the design of the shear connectors. Hence, the amount of deformation observed for a beam under uniform loading will be larger than the deformation observed for concentrated loading on a similar beam having the same maximum moment.

The order of magnitude of the maximum slip which a group of connectors of a given type can undergo without failure can be obtained from the values of slip given in the last column of Table 2 and the slip values given in Tables 4, 6, and 7. The amount of slip for members having adequate connector strength at M_u as defined herein is considerably less than the values of slip given in these tables. Therefore, when enough shear connectors are provided, there is no danger of failure of connectors due to large slips which result from several applications of design load.

A criterion which can be conveniently and logically used to determine if the amount of slip at working load is excessive is that the slip should not exceed the amount of slip which occurs in a non-composite beam designed to carry the same load. Member B11 loaded with five concentrated loads, for instance, could be replaced by a non-composite member 14WF43 having the same slab dimensions. Calculations show that the approximate end slip for this member assuming no interaction would be 0.028 inches at working load assuming one concentrated load at midspan. The end slip for member B11 was 0.014 inches after ten cycles of 108% of working load with the more severe uniform loading condition. Working load for B11 is here defined as $M_u/2$. Thus even when a composite member is not provided with sufficient connectors, the amount of slip is still considerably less than the amount which occurs in a non-composite member designed for the same loads.

The ability of connectors to undergo relatively large slips prior to failure renders valid the assumption of equal loads on connectors at ultimate load. Since there is a considerable difference in the magnitude of slip at M_u (for beams with adequate connectors) and at failure of connectors, it is not necessary to space shear connectors in accordance with the shear diagram. Connectors may be spaced uniformly over the length of the member regardless of the shape of the shear diagram.

In determining the number of shear connectors to be used in a member supporting a uniform load the critical point for design of connectors will not be midspan of the member.

This is illustrated by Figure 17 which shows available connector strength, $\sum q_u$, and required compressive force, C, along the half span of a member loaded with a uniform load. This curve is based upon providing sufficient connectors uniformly spaced to resist the maximum value of C at midspan. It will be observed that at several points in the member, the value of $\sum q_u$ is less than the value of C. This condition requires a redistribution of stress at points throughout the length of the member and results in the existence of points in the member where the maximum moment obtainable is M_u' rather than M_u.

The number of shear connectors provided in members B10, B11, and B12 was inadequate such that the ultimate strength is M_u' rather than M_u at midspan. Because of the shape of the C/C_u versus length curve, there are points along the member where $\sum q_u$ is not equal to the required value of C. Therefore, even the value of M_u' calculated for these members cannot be reached without connector failure. Figure 18 shows a curve for $\sum q_u$ and C versus length for members such as B7-T2. Notice that in this member the magnitude of C is less than $\sum q_u$ if the number of connectors provided at the critical section is sufficient. This comparison serves to illustrate that the design of shear connectors for members with uniform loading is more critical than design for other loading conditions, and that designing for a section at midspan is not sufficient for uniform loading. However, the amount of error is less than the approximations made in the design assumptions, and it is not necessary to make special provisions in specifications to cover this situation.

The effect of spacing connectors uniformly instead of in accordance with the shear diagram is illustrated in comparing Figures 19 and 20. Figure 19 shows the moment versus end slip for members B10, B11, and B12. It will be noticed that member B12, having shear connectors spaced by the shear diagram, attains higher values of moments with less slip than members B10 and B11 having the same number of connectors spaced uniformly. However, at a slip of approximately 0.075 inches, the load-slip curves coincide for all members.

The load-deflection curves for the same three members are given in Figure 20. These curves exhibit no significant differences. It can be concluded from this that shear connectors may be uniformly spaced so long as they exhibit a load-slip curve similar to those of welded studs and channel connectors.

It has been demonstrated that the amount of slip which occurs in a composite beam can be varied depending upon the number of connectors provided. The number of connectors to be specified in design should be arrived at on the basis of adequate strength and economics. It is obvious from Figure 3 that the strength of shear connectors provided should be equal to or greater than 0.85 $f'_c ba$ for case I or 0.85 $f'_c bt$ for case II. This represents the most economical solution possible using M_u as a basis for design. The number of connectors can be reduced only by designing on the basis of the modified ultimate moment, M'_u. This would not be satisfactory because it does not utilize the cross section of the member efficiently. There is also no reason for using extra connectors in addition to the least that is adequate because these extra connectors do not increase the load-carrying capacity of a member. Extra connectors do not reduce deflections at working load.

7. APPLICATION TO CONTINUOUS BEAMS

The design of composite construction might be made even more economical by applying the concepts of plastic analysis along with ultimate strength design. To investigate whether this application of plastic design is feasible, one continuous member was tested. Figure 21 gives dimensions and loading conditions for the test of member B13, which was continuous over two spans.

The ultimate strength of this member was determined using both plastic design and ultimate strength theory. The ultimate moment of the positive moment region was taken as Mu of the composite section, whereas the ultimate moment of the negative moment region was taken as M_p of the steel member and longitudinal slab reinforcement.

The member was tested first by loading only one span at a time and stopping the loading below ultimate. Finally the member was tested to failure with two concentrated loads on each span. The maximum load, P_p, determined by plastic analysis was calculated. The load-deflection curves for loading of both spans simultaneously are shown in Figure 22 with the loads given in terms of percentage of P_p. The load P_p was exceeded on the test even though the values of $\sum q_u/C$ were only 0.888 for the ends and 0.978 for the interior portion. The deflection in the East Span was larger than in the West Span probably because the East end of the member was more free to expand during the test. This test indicates that the design of shear connectors is not critical for members designed by plastic analysis.

Complete design rules for plastic design of composite structures cannot be developed from only one test. The performance of this member suggests some tentative recommendations to permit plastic design for similar members. It was observed during the test that wide cracks formed in the negative moment region even at loads below working load. A means of controlling this cracking should be employed in the design, either in the form of an expansion joint or sufficient slab reinforcement to reduce crack width to an acceptable value.

Plastic design should be restricted to those members in which the negative plastic hinge forms first. This excludes all built-up members with cover plates in the negative moment region in which positive plastic hinges might form prior to the formation of negative plastic hinges. If the positive hinge forms first in a composite member, there is danger of insufficient rotation capacity because of crushing of the concrete slab. Rotation capacity, cover plates and slab reinforcement requirements should be studied further before unrestricted use of plastic design in composite construction is recommended.

8. EVALUATION OF ULTIMATE STRENGTH DESIGN PROVISIONS

Although ultimate strength design is essential to plastic design, there is good reason to have ultimate strength design available even though plastic design were not permitted. The principal advantages of ultimate strength design as compared to elastic design are (1) it is simple and logical, (2) it provides a uniform factor of safety, and (3) there is some economy possible in reducing the factor of safety for certain members.

The factor of safety against ultimate strength of a composite member which results from current elastic design procedure ranges from a minimum of 2.14 to a maximum of 2.60. The factor of safety, assuming f_y = 33 ksi becomes (33/20) (Z_c/S_c) where Z_c/S_c varies from approximately 1.30 to 1.60. S_c is the section modulus of the composite member referred to the bottom flange, and Z_c is defined as the ultimate moment divided by f_y.

A factor of safety against ultimate strength of 2.0 is recommended as being adequate for composite members. This is based upon the consideration that (1) the load-deflection curve should be linear up to working load, (2) the stress in the steel beam should be less than f_y, and (3) deflections should be stable at working load. Figure 23 shows a typical load-deflection curve for a beam having adequate shear connector strength. The relative position of working load and first yielding of the steel beam are noted. It will be noted that working load and first yielding of the bottom flange of the steel beam nearly coincide. This is due to the fact that in computing M_u for this member, the actual values of f_y for the web and flanges were used and reinforcing steel in the slab was also considered. Therefore, the working load indicated in Figure 23 is higher than the design working load which would be obtained using nominal values. It can be seen that the load-deflection curve is linear up to working load and that the deflection at working load is satisfactory as compared with the commonly accepted maximum for building members of L/360.

The use of a load factor of 2.0 is also recommended for the design of the steel beam for supporting unshored dead loads. The use of these two load factors along with a specified maximum effective width of concrete slab of sixteen times the slab thickness is sufficient to limit the stress in the bottom flange of the steel beam to values less than f_y for any loading condition. The stress in the bottom fiber of the steel section may be obtained as follows:

$$f_s = M_D/S + M_L/S_c \qquad (20)$$

where the maximum

$$M_D = Zf_y/2 \qquad (21)$$

and

$$M_L = Z_c f_y/2 - M_D \qquad (22)$$

giving

$$f_s = (Z/S + Z_c/S_c - Z/S_c)f_y/2 \qquad (23)$$

In the case of the bridge member of Figure 23, the bottom fiber steel stress based on design section properties would be 30.4 ksi computed from the following section properties: $Z = 100.8$ in.^{3}, $S = 89.0$ in.^{3}, $S_c = 139.4$ in.^{3}, and $Z_c = 199.2$ in.^{3}. This is approximately the maximum steel stress to be expected in practice, although extremely unfavorable combinations of section properties could result in higher steel stresses. It does not seem necessary to restrict the value of Z_c to be used in design in an effort to reduce steel stresses. Members B3 through B12 were loaded with ten repetitions of 108% of working load, based upon actual yield stresses, before proceeding with testing. This loading did not cause any progressive increase in deflection after the second cycle, indicating that deflections at this loading are stable.

9. SUMMARY AND CONCLUSIONS

Several composite steel and concrete beams and a number of pushout specimens were tested to determine the behavior of composite beams and of non-rigid mechanical shear connectors in the elastic and inelastic ranges. The results were included with results from other investigations in a study to recommend applications in design of composite beams for buildings.

All findings are for composite beams in which the concrete slab is poured flush with the top surface of the steel beam. Observations and conclusions from this study are as follows:

(1) In beams without shear connectors, shrinkage of concrete may destroy natural bond before the member is loaded. In this case there is no resulting interaction due to natural bond and very little interaction due to friction between the steel and concrete surfaces.

(2) In beams with shear connectors, the restraint furnished by the shear connectors may preserve the natural bond. When such beams are tested, the shear connectors carry almost no load before bond failure, the interaction being provided by bond. However, a definite bond failure usually occurs during testing, often in the vicinity of design load. Following bond failure, the shear connectors provide interaction.

(3) Semi-empirical formulas were developed to express the ultimate strengths of stud, spiral, and channel shear connectors as determined from tests. These formulas are used as the basis for design recommendations.

(4) The ultimate moment computed on the basis of full plastification of steel and concrete may be reached when an adequate number of shear connectors is used. If the sum of the ultimate strengths Σq_u of all connectors in the shear span is greater than or equal to the compressive force tending to slide the concrete slab along the steel beam, the shear connectors are adequate.

(5) If the amount of shear connectors is less than the amount termed adequate, a modified ultimate moment may be calculated which will predict the ultimate moment of these members. The compressive force in the concrete slab is limited to the sum of the ultimate loads of the active shear connectors. The steel beam is assumed to have sufficient yielded areas in both tension and compression to produce equilibrium at the critical cross section.

(6) For members having adequate or greater than adequate shear connector strength to develop the full possible ultimate moment of the member, the influence of slip upon the load-deflection curve of a member is negligible.

(7) For members having less than adequate shear connector strength, slip does influence the load-deflection curve of the member. This may be explained by the fact that the effective moment of inertia of the cross section is reduced due to inadequate shear strength.

(8) The ductility of non-rigid shear connectors will permit sufficient redistribution of load to allow uniform spacing of connectors along the part of the member requiring shear connectors, regardless of the shape of the shear diagram.

(9) A single test of a continuous two-span composite beam attained the ultimate load predicted on the basis of composite moment in the positive moment regions and the plastic moment furnished by the steel in the negative moment regions. The test indicated a need for controlling cracking of the concrete in negative moment regions.

Recommendations for the design of composite beams are presented in the following article.

10. DESIGN RECOMMENDATIONS

The results of this investigation indicate that modifications of existing elastic design provisions can be made which will result in more economical composite construction.

The design of shear connectors for elastic design can be based upon the ultimate strength of connectors as specified in the "Ultimate Strength Provisions" which follow. The ultimate strength of a connector divided by a suitable factor of safety is the allowable shear connector force at working load. The minimum recommended factor of safety to be used for static loading is 2.0. Test results indicate that shear connectors may be spaced uniformly throughout the shear span regardless of the shape of the shear diagram for the member provided that an adequate number of shear connectors is furnished. An adequate number is the minimum number required to insure that the ultimate moment of the member is attainable, and this is insured if one designs by the following "Ultimate Strength Provisions".

The design recommendations which follow are written as a suggested addition to "Tentative Recommendations for the Design and Construction of Composite Beams and Girders for Buildings".[14] These new recommendations provide for ultimate strength design of all simple-span composite members for static loads, and for the plastic design of certain continuous beams as specified.

ULTIMATE STRENGTH PROVISIONS

501 - Definition and Scope

1. This section presents recommendations for design of composite steel and concrete members on the basis of the fully plastic stress distribution which exists at ultimate load.

2. Ultimate strength provisions may be used in conjunction with plastic analysis for the design of continuous members in which the negative plastic hinge forms before the positive plastic hinge. Built-up members in which positive plastic hinges form first may be designed if it can be shown that the member has sufficient rotation capacity to permit satisfactory redistribution of moments.

3. The ultimate moment of the composite section in the region of negative bending moment shall be computed on the basis of the steel area of the cross section including reinforcing steel, if shear connectors are used in this region.

502 - General Requirements

1. Previous sections of these recommendations apply except where amended herein.

2. Analysis of indeterminate structures such as continuous beams shall be based on simple plastic theory.

3. Proper slab reinforcement or expansion provisions shall be provided at negative moment areas of continuous beams.

4. Ultimate moment of members is not altered by the method of construction (shored or unshored) employed.

503 - Assumptions

Ultimate strength design of composite members is based upon the following assumptions:

(a) Plane sections normal to the axis remain plane after bending.

(b) Tensile strength of concrete is neglected.

(c) Complete interaction exists for all loads up to ultimate.

(d) At ultimate load the stress diagram for concrete in compression may be taken as rectangular with a maximum stress of $0.85\ f'_c$. The stress diagram for steel may be taken as rectangular with a maximum stress of f_y for steel in either tension of compression.

504 - Load Factors

1. Members must be proportioned so that an ample factor of safety exists for the case of the steel member alone supporting unshored dead load, and for the composite section supporting total load.

2. The following load factors are to be used in the design of steel beam for unshored dead load and the composite member for total load.

For structures in which effects of wind and earthquake can be neglected:

$$M_u = 2.0 \ (M_D + M_L)$$

For structures in which wind and earthquake loading must be considered, the following applies to the composite section only:

$$M_u = 2.0 \ (M_D + M_L) + 1.4E$$

where M_u is the theoretical ultimate moment, M_D is the moment due to dead load, M_L is the moment due to dead load and live load applied to the composite section, and E is moment due to wind and earthquake loading.

505 - Design of Shear Connectors

1. Shear connectors shall be designed on the basis of their ultimate strength assuming that all connectors will carry equal load at ultimate load.

2. The number of connectors required between sections of maximum and zero moment at ultimate load shall be equal to the total compressive force C, in the concrete slab at the section of maximum moment divided by the ultimate strength of one connector.

3. The required number of connectors shall be spaced uniformly between the sections of maximum and zero moments.

4. The ultimate strengths of shear connectors shall be taken as the values given in section 506.

506 - Ultimate strength of shear connectors

1. Stud shear connectors

$$q_u = 930\ d_s^2 \sqrt{f'_c}$$

for h/d_s larger than 4.2

$$q_u = 220\ hd_s \sqrt{f'_c}$$

for h/d_s smaller than 4.2, in which q_u is the ultimate load per one stud in pounds, h denotes the height of stud in inches, d_s is the diameter of stud in inches, and f'_c represents the 28-day compressive strength of concrete in pounds per square inch. In no case should the ultimate load per stud exceed $A_s f'_s$ where A_s is the area of the stud in square inches, and f'_s is the ultimate strength of the stud material in tension.

2. Spiral shear connectors

$$q_u = 8000\ d_s \sqrt[4]{f'_c}$$

in which q_u is the ultimate load per one pitch of spiral in pounds, and d_s denotes the diameter of bar in inches. In no case should the ultimate load per turn of spiral exceed $2A_s f'_s$ where A_s is the area of the spiral in square inches, and f'_s is the ultimate strength of the spiral material.

3. Channel shear connectors

$$q_u = 550 \ (h + 0.5t) w \sqrt{f'_c}$$

in which q_u is the ultimate load per one channel connector in pounds, w represents the length of channel in inches, t is the thickness of web in inches, and h denotes the average flange thickness in inches.

4. For connectors other than the above, the ultimate load should be developed from test data.

5. A minimum load factor of 2.0 should be used with the above formulas to determine the allowable load per connector for use in elastic design.

ACKNOWLEDGEMENTS

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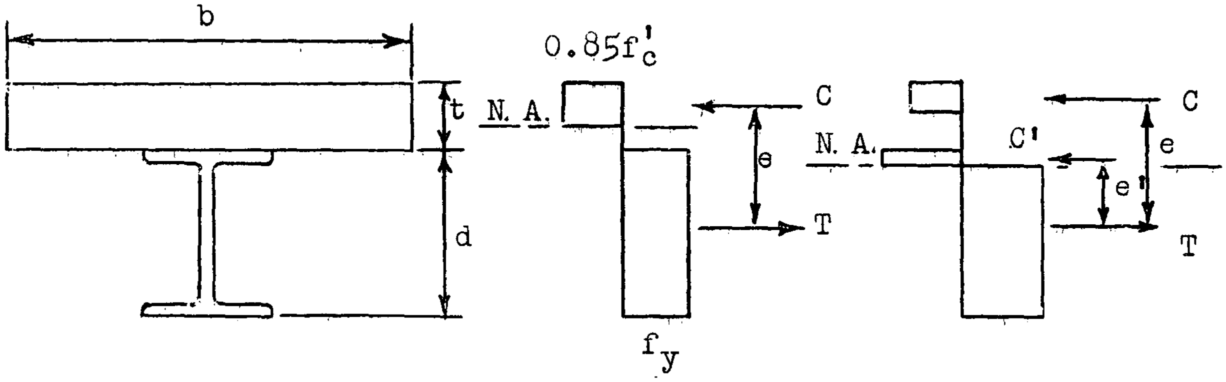
The work was completed at Fritz Engineering Laboratory, of which Dr. L. S. Beedle is the director. Appreciation is expressed to Mrs. L. Morrow for typing this report.

NOMENCLATURE



a = depth of compressive stress block in concrete slab

A_s = area of steel in tension

A_s' = area of steel in compression

b = effective width of concrete slab

C = compressive force in concrete slab

C' = compressive force in steel beam

C_u = compressive force in concrete slab at ultimate moment

d = depth of steel section

d_s = diameter of shear connector

e = distance between compressive force in slab and tension force at ultimate moment

e' = distance between compressive force in steel beam and tension force at ultimate moment

f_s = steel stress in bottom fiber of steel beam at working load

f_y = yield stress of steel beam

f'_s = ultimate strength of steel

f'_c = 28-day concrete strength

h = average flange thickness of channel connector

H = height of shear connector

I = moment of inertia of steel beam

I_c = moment of inertia of composite beam

L_s = distance between sections at which ultimate moment and zero moment occur

M_D = moment due to dead load before concrete attains 75% of f'_c

M_L = moment produced by live load and superimposed dead load

M_p = plastic moment of steel section

M_u = ultimate moment of composite section

M'_u = modified ultimate moment of composite section with insufficient shear connectors

q = resistance value of one shear connector at working load

q_u = ultimate strength of one shear connector

Σq_u = ultimate strength of shear connectors in distance L_s

s = spacing of shear connectors

S = section modulus of steel beam

S_c = section modulus of composite section

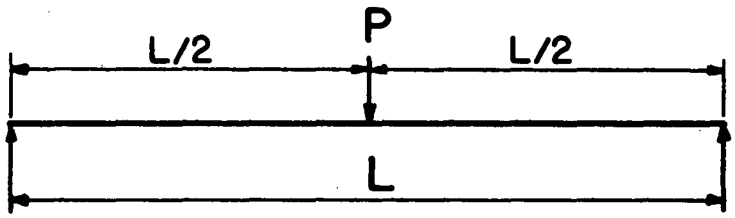
t = thickness of concrete slab; thickness of web of steel beam

T = total tensile force

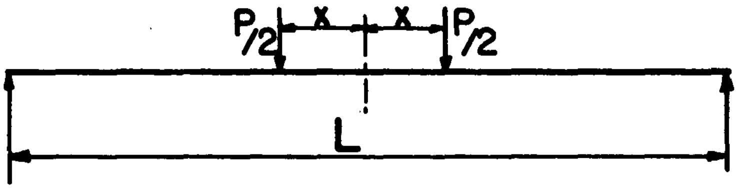
w = width of channel shear connector

Z = plastic modulus of steel section

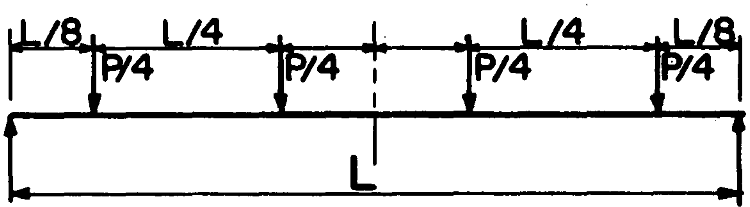
Z_c = plastic modulus of composite section defined as ultimate moment divided by yield strength of steel



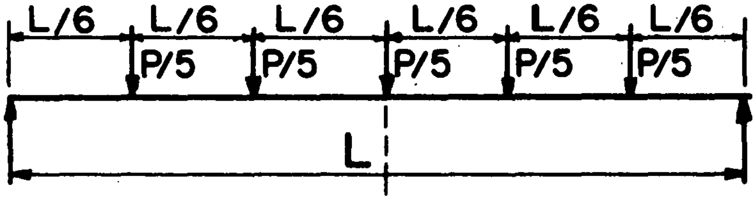
Test T1



Test	Length x in Inches
T2	9
T3	12
T4	18
T5	21
T6	23
T7	28
T8	30
T9	33
T10	36
T11	38



Test T12



Test T13

Figure 1 Summary of Test Beam Loading Conditions

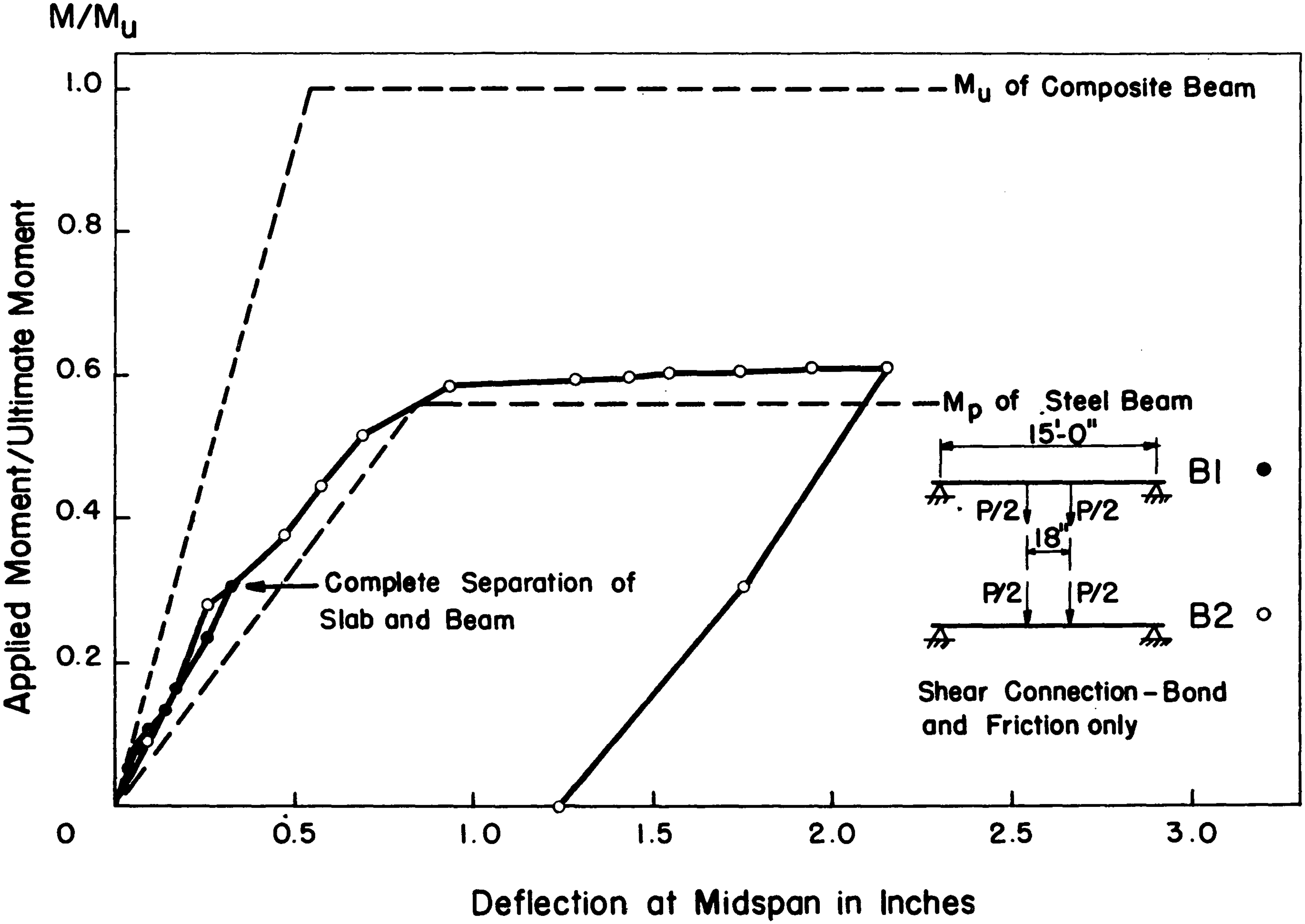
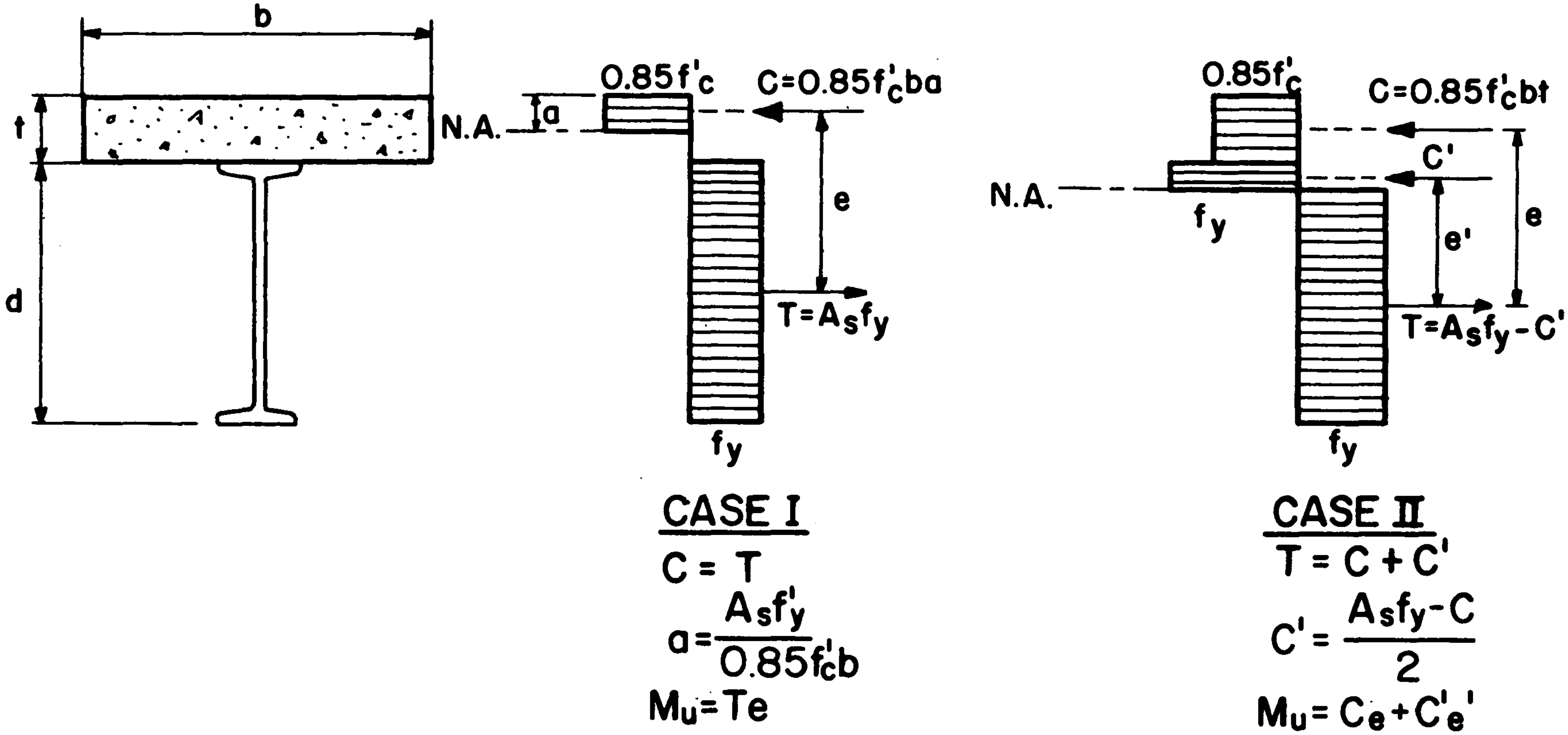


Figure 2 Load Deflection Curves for Beams Without Shear Connectors



Stress Distribution at M_u

Figure 3 Method of Calculation of Ultimate Moment (M_u)

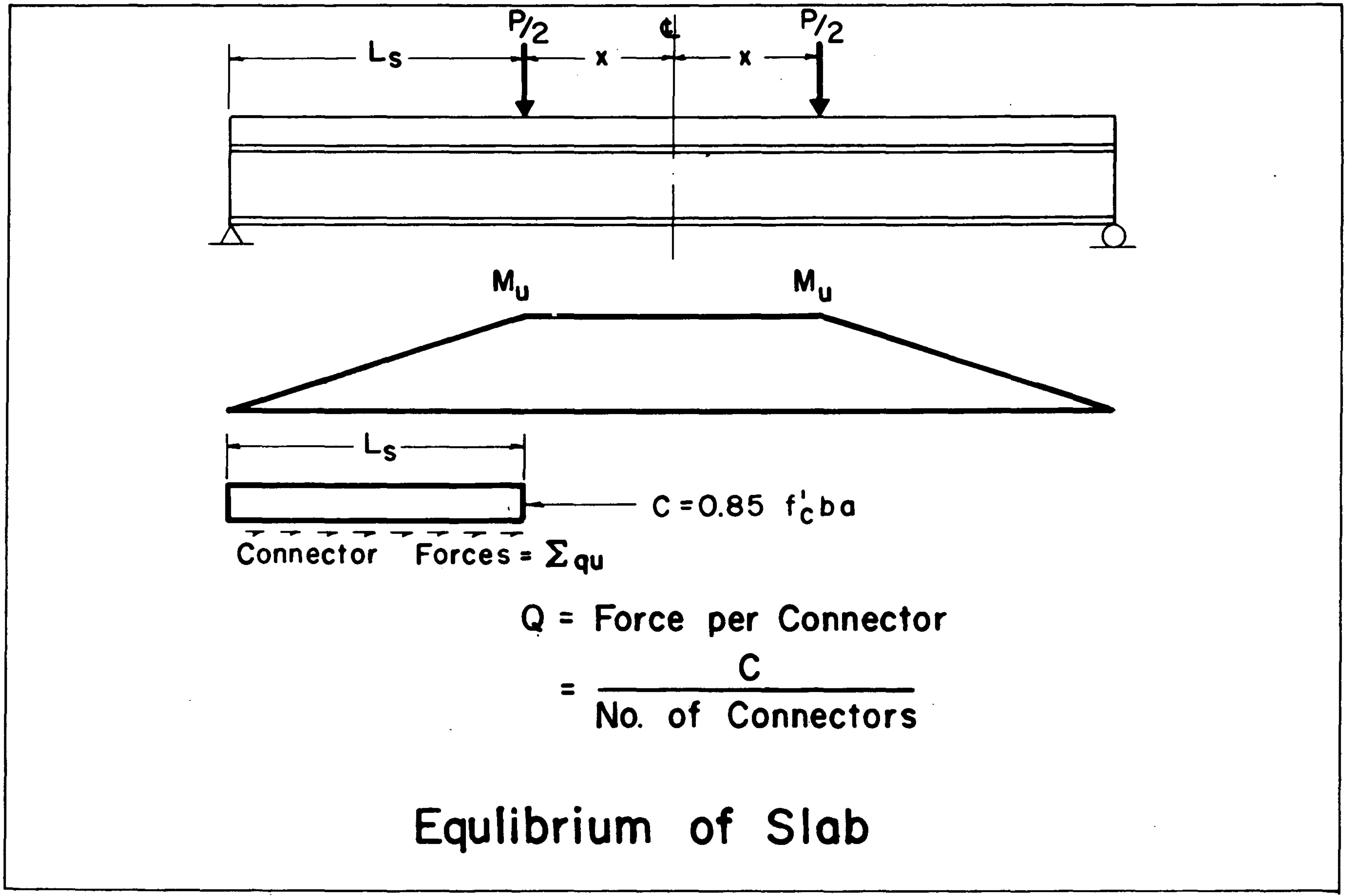


Figure 4 Calculation of Load per Shear Connector

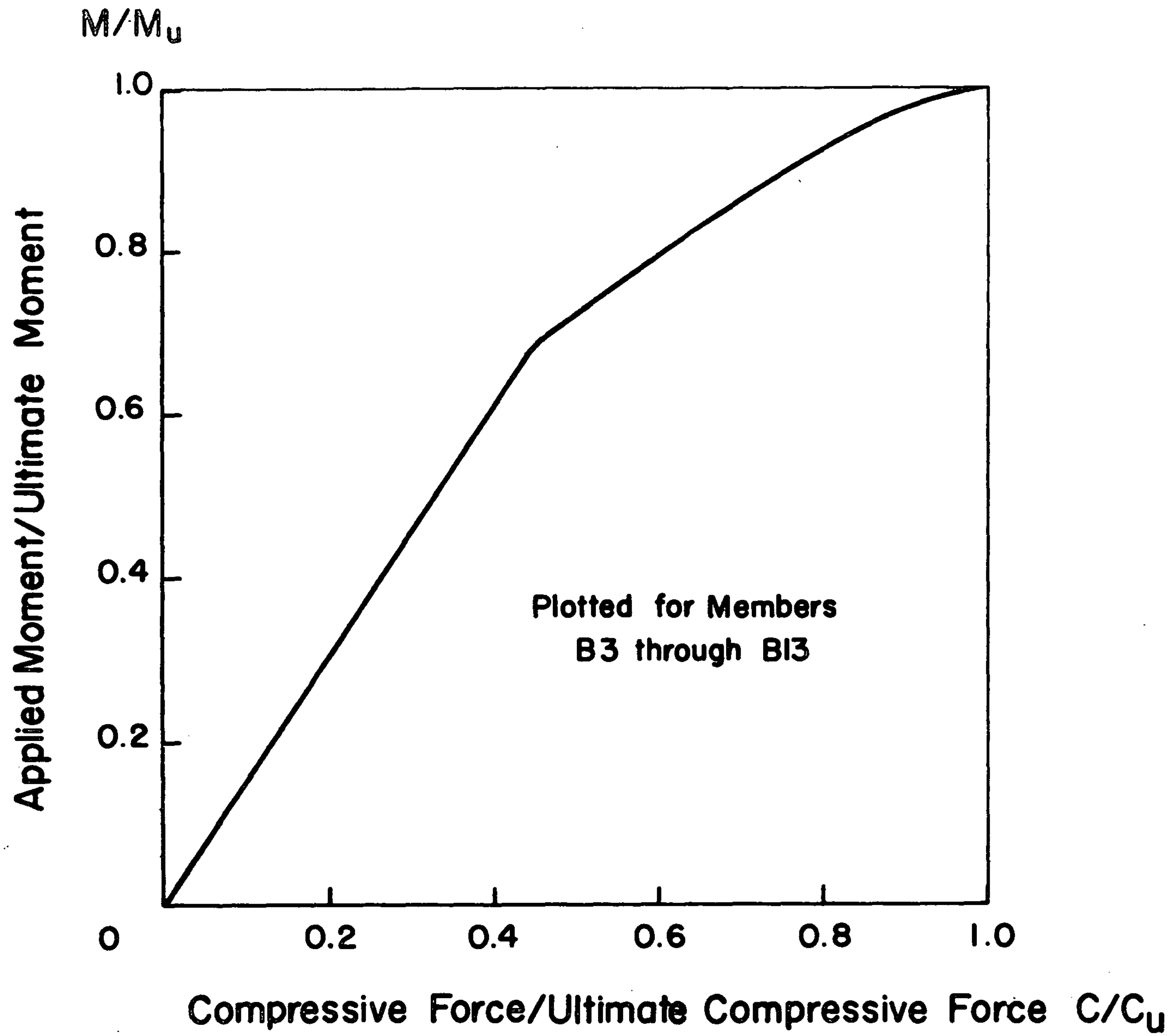


Figure 5 Relationship Between Compressive Force and Applied Moment

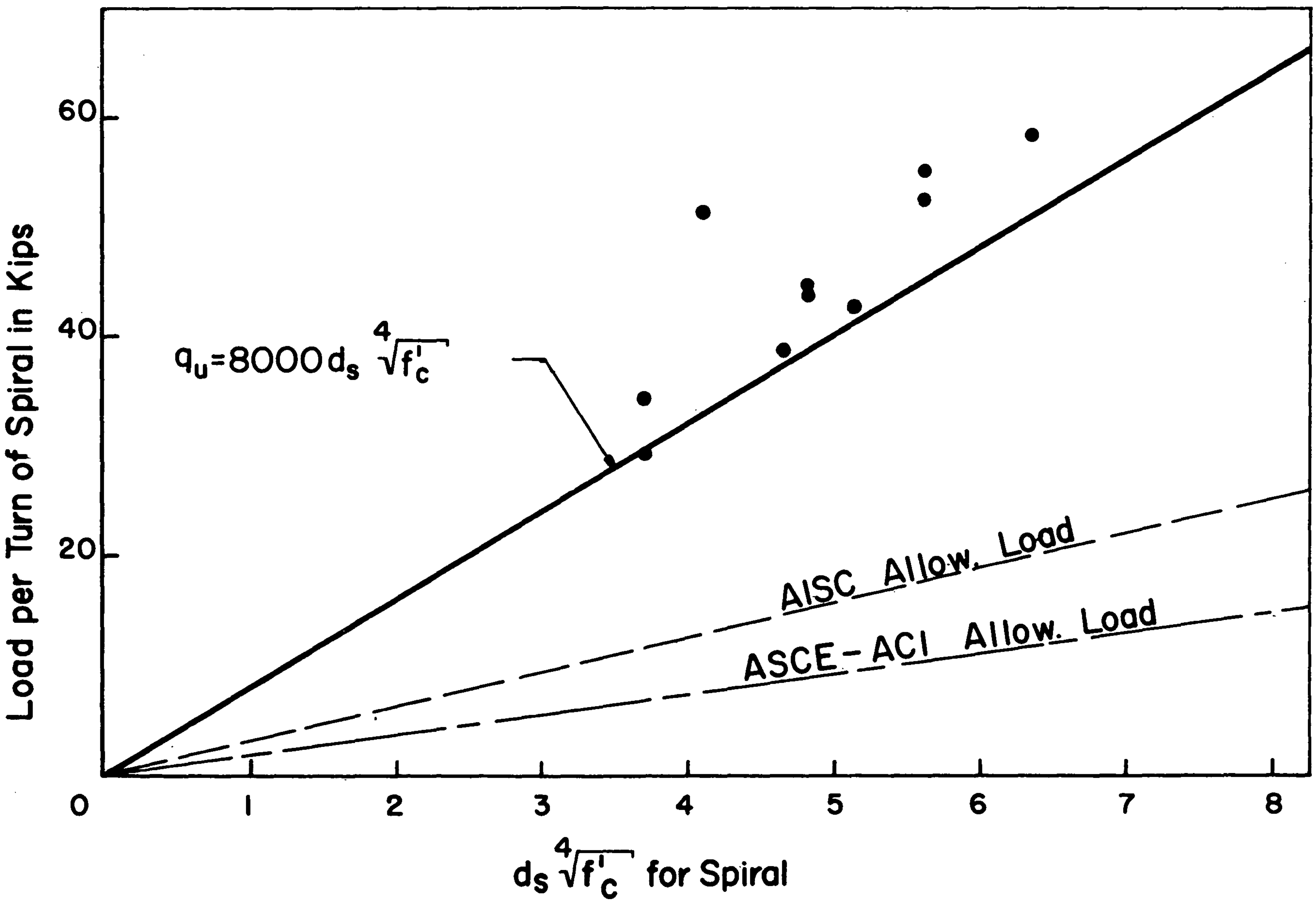


Figure 7 Ultimate Strength of Spiral Shear Connectors

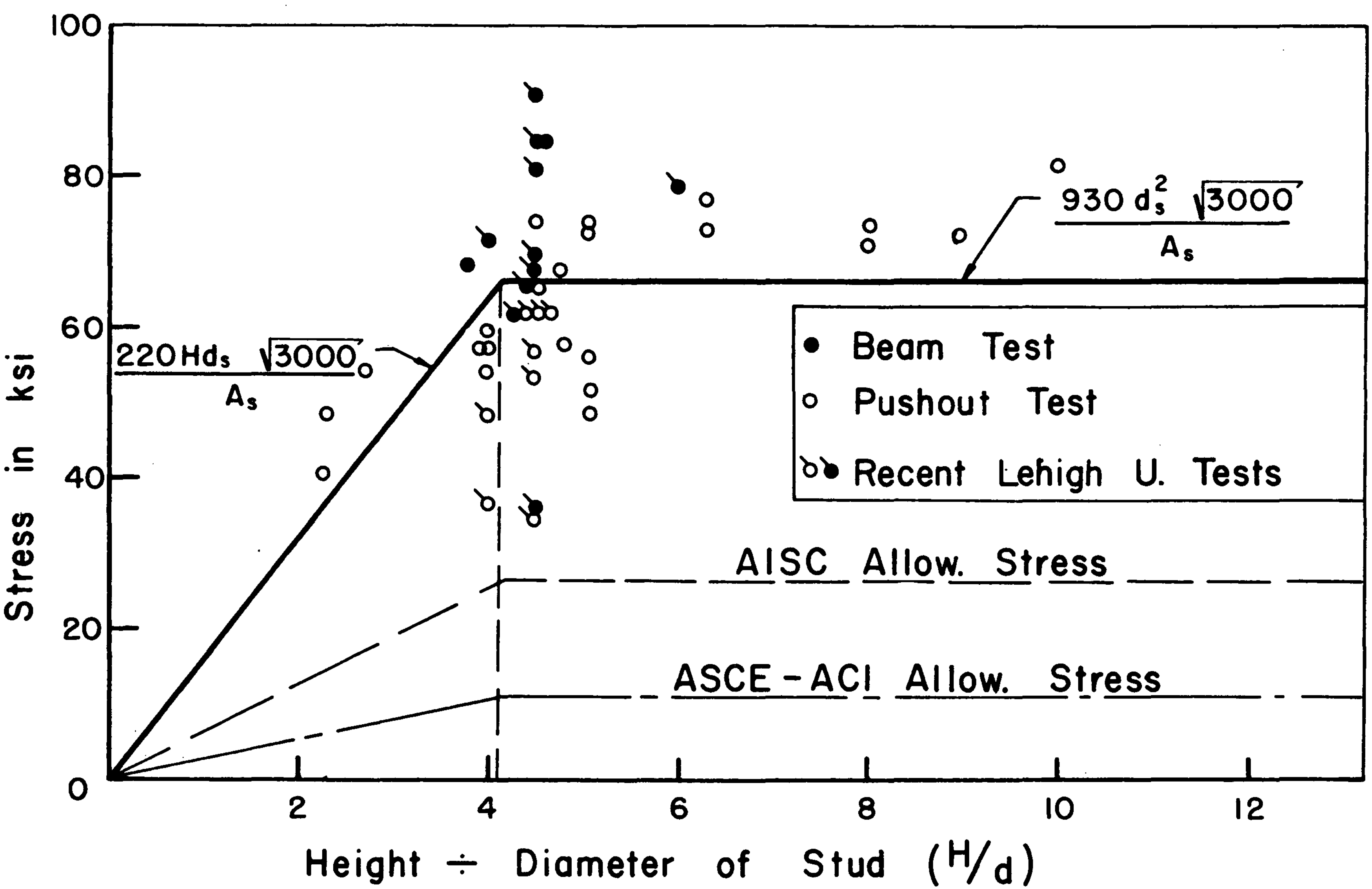


Figure 6 Ultimate Strength of Stud Shear Connectors

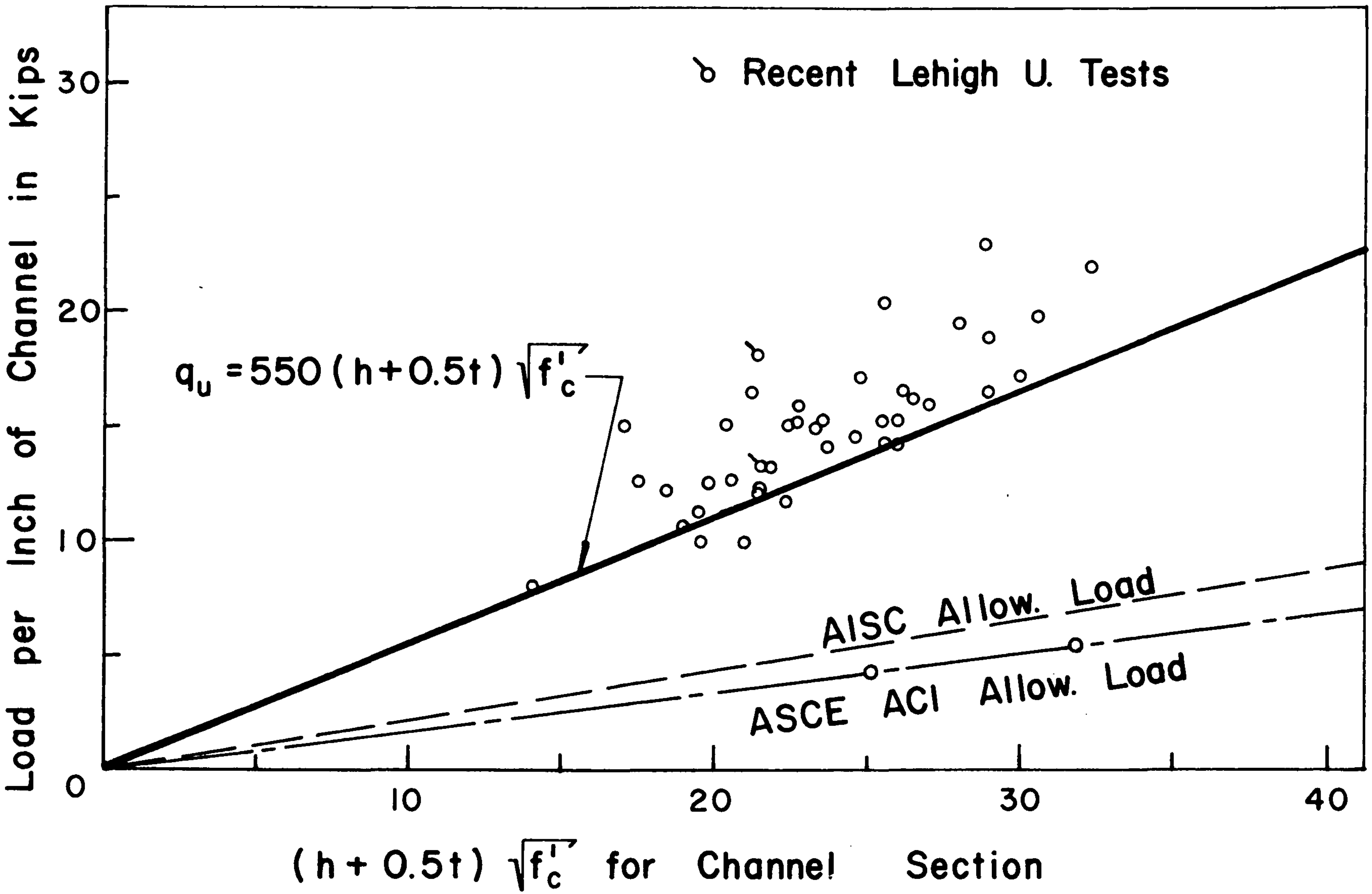


Figure 8 Ultimate Strength of Channel Shear Connectors

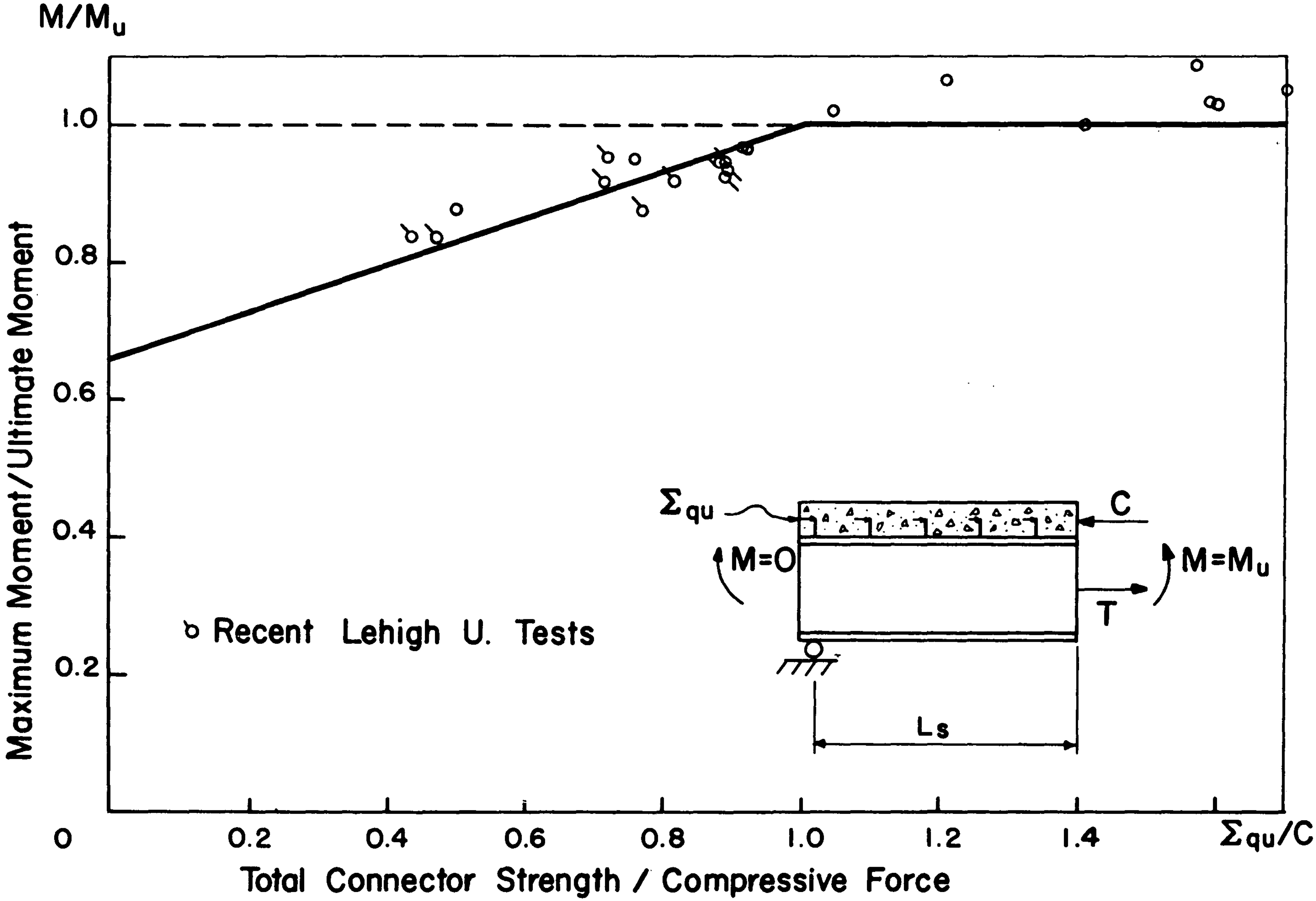
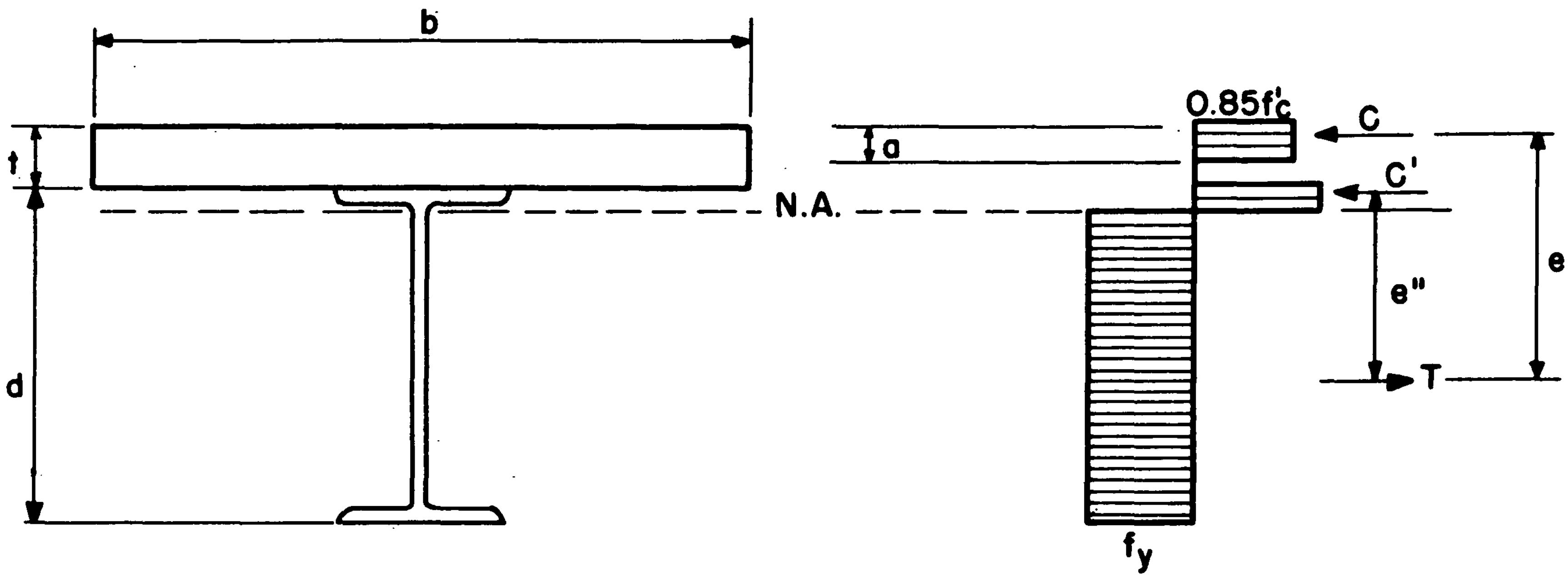


Figure 9 Relationship between Shear Connector Strength and Moment Capacity



$$C = \Sigma q_u$$

$$a = \frac{\Sigma q_u}{0.85 f'_c b}$$

$$C' + T = A_s f_y$$

$$C + C' = T$$

$$M'_u = Ce' + Ce''$$

Stress Distribution at M'_u

Figure 10 Calculation of Modified Ultimate Moment (M'_u)

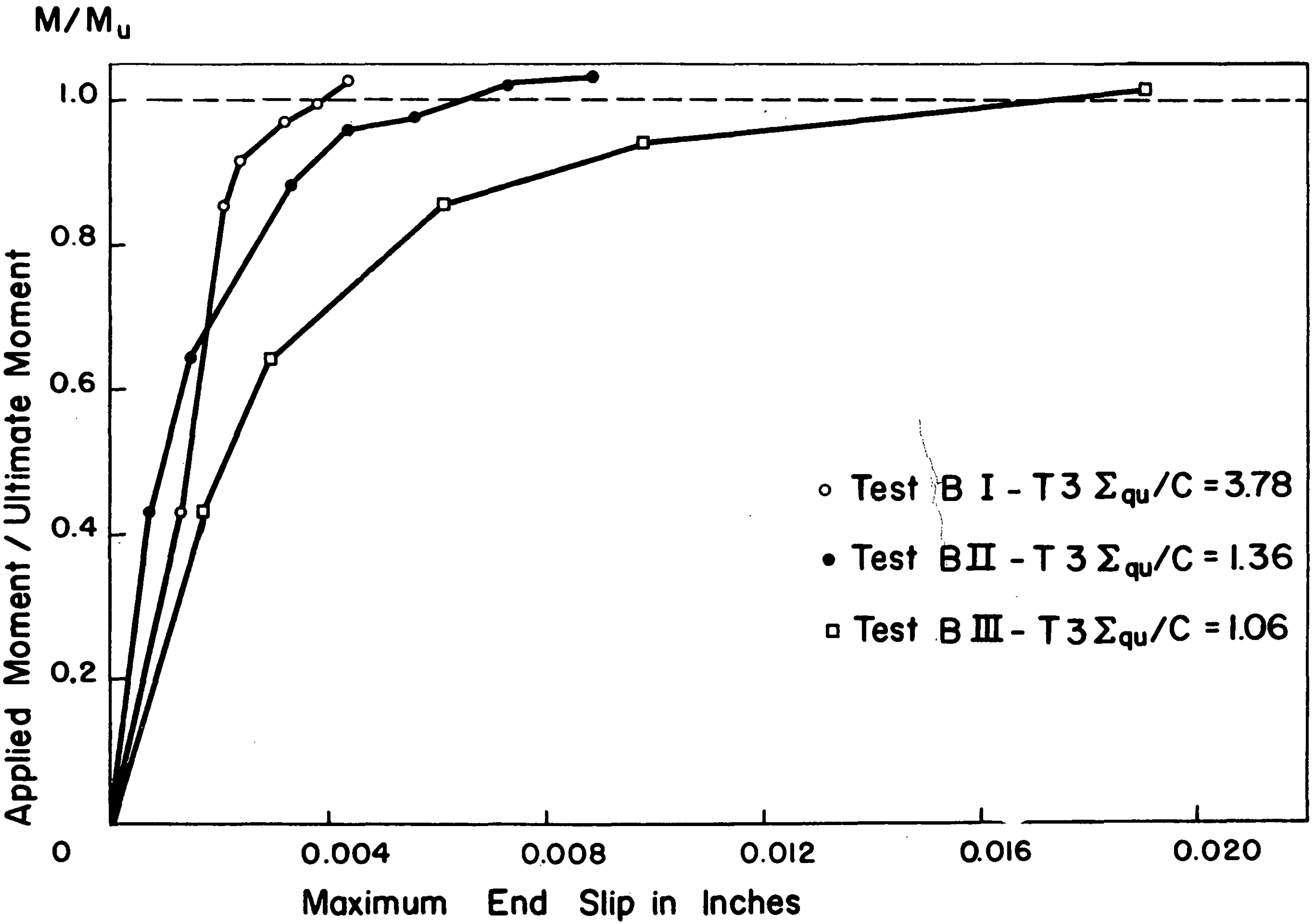


Figure 11 End Slip for Members Having Adequate Shear Connectors

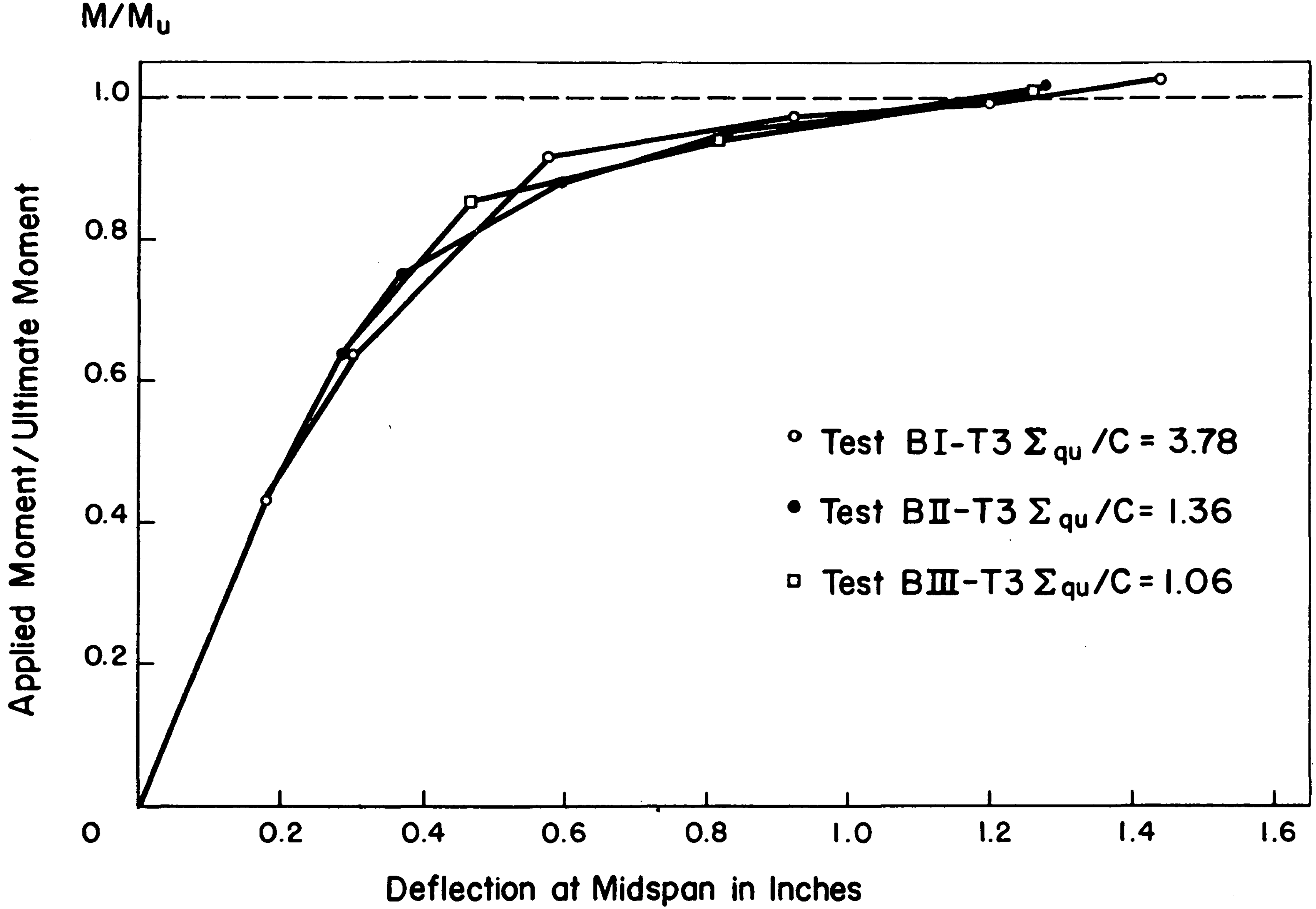


Figure 12 Load-Deflection Curves for Members Having Adequate Shear Connectors

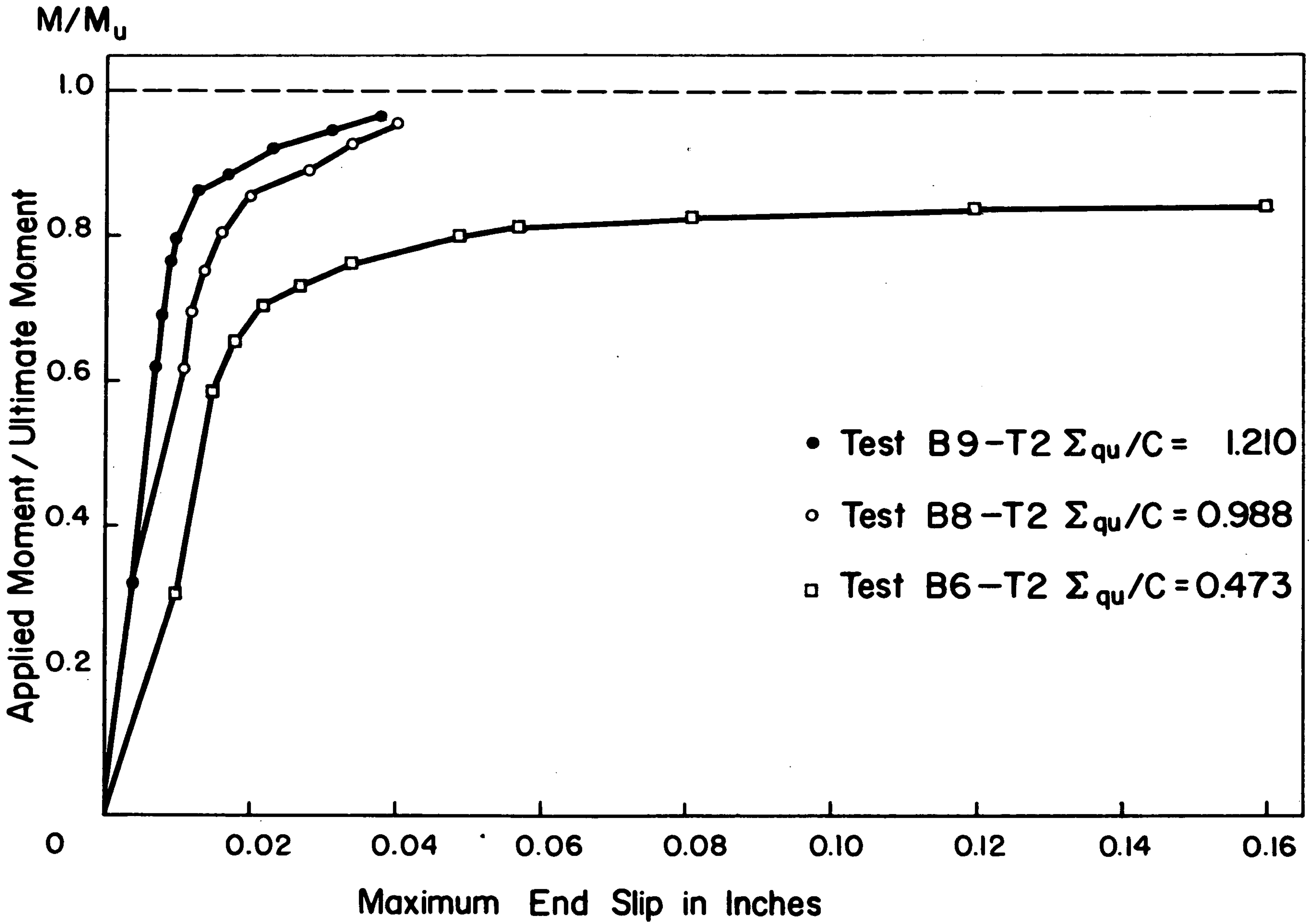


Figure 13 End Slip for Members Having Inadequate Shear Connectors

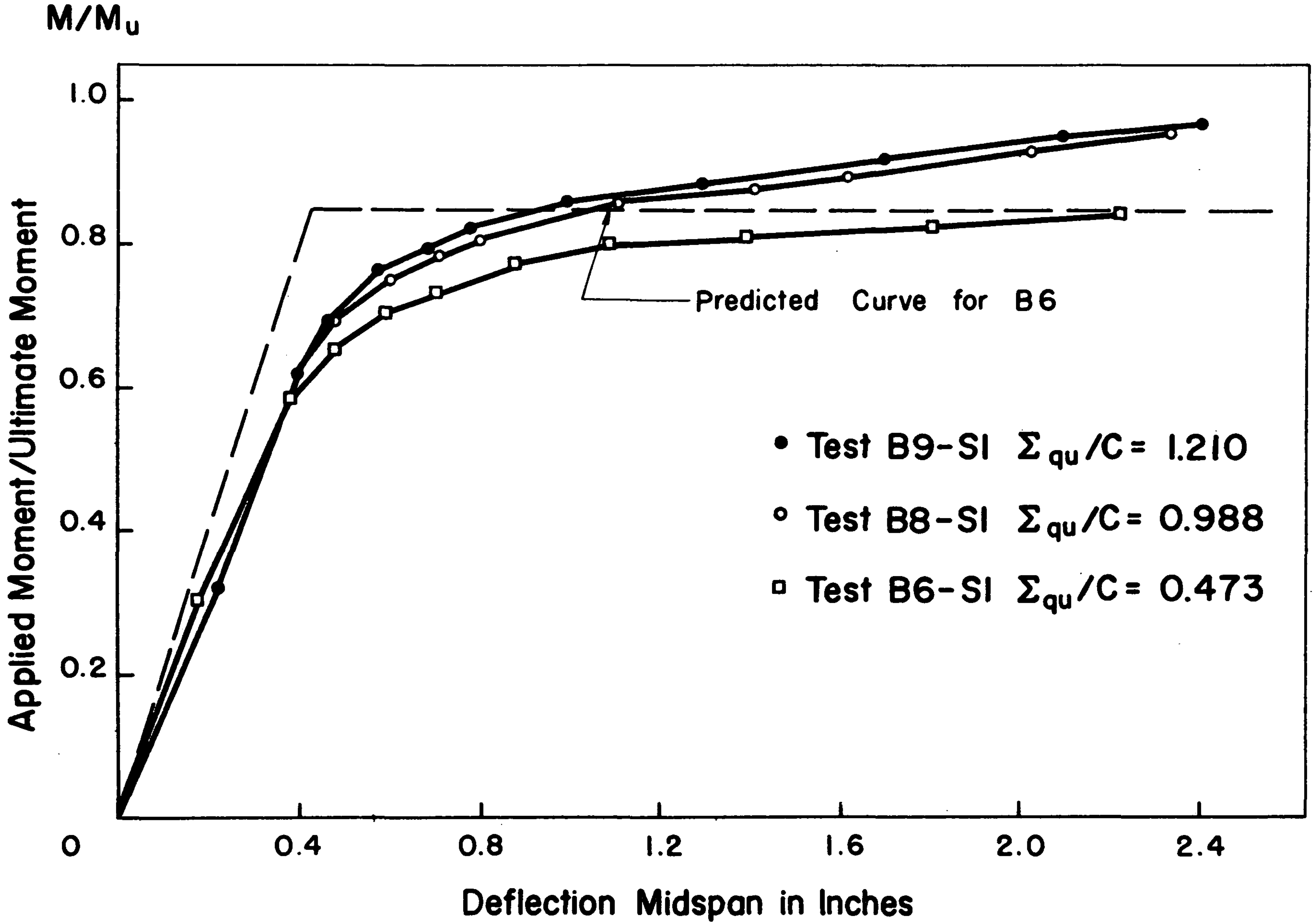


Figure 14 Load-Deflection Curves for Members Having Inadequate Shear Connectors

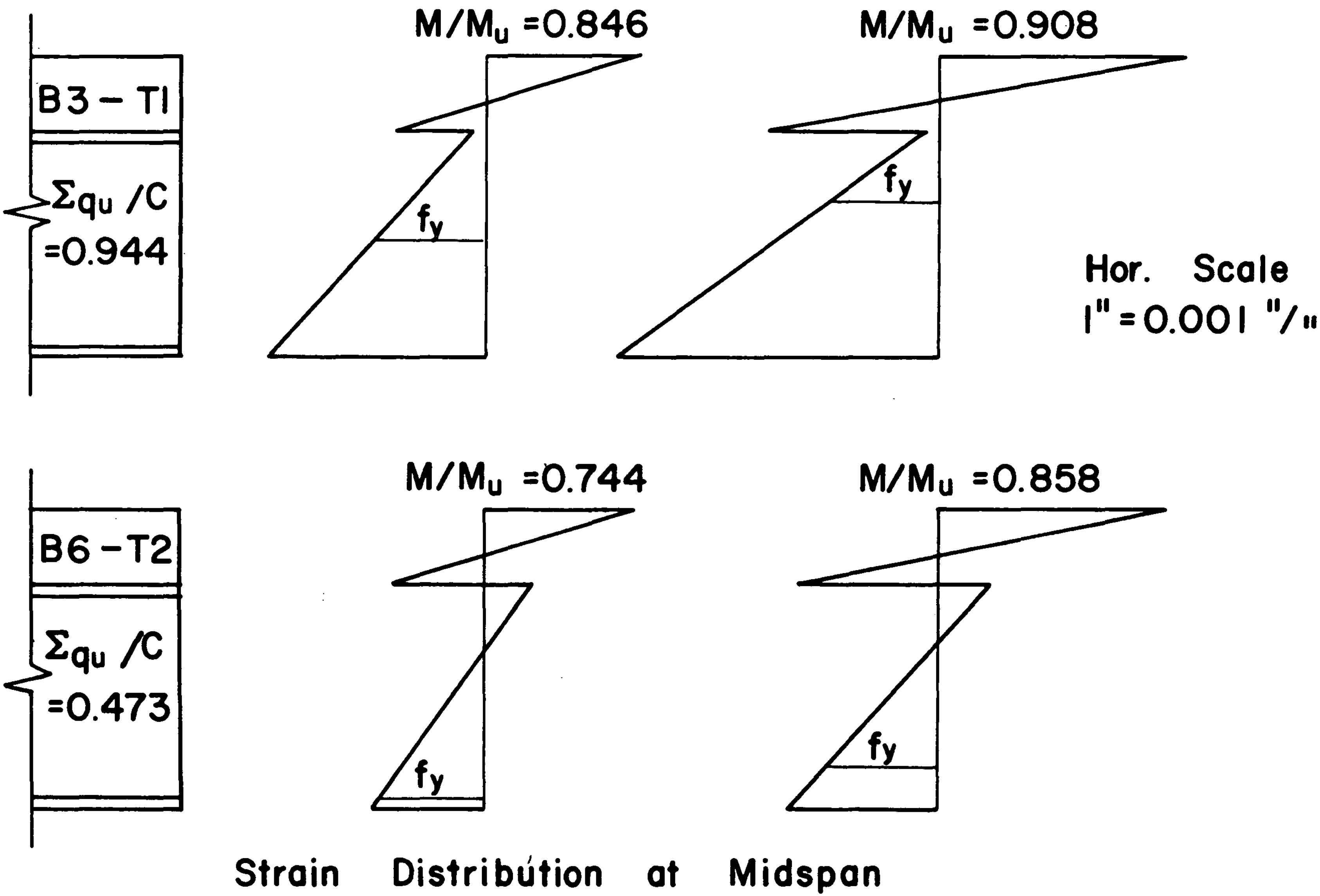


Figure 15 Measured Strains on Members at Midspan

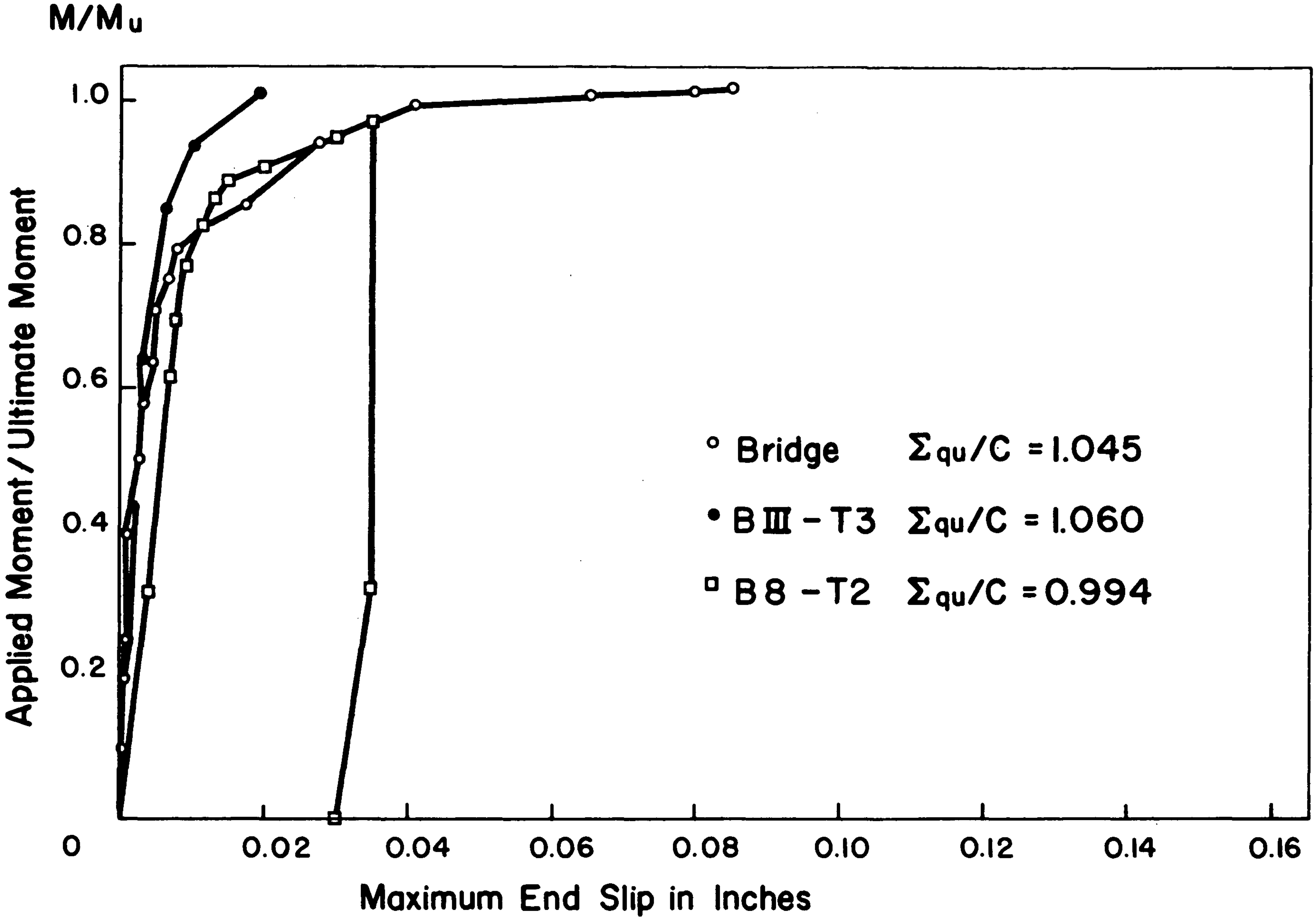


Figure 16 End Slip for Members Having $\Sigma q_u/C$ Approximately = 1.00

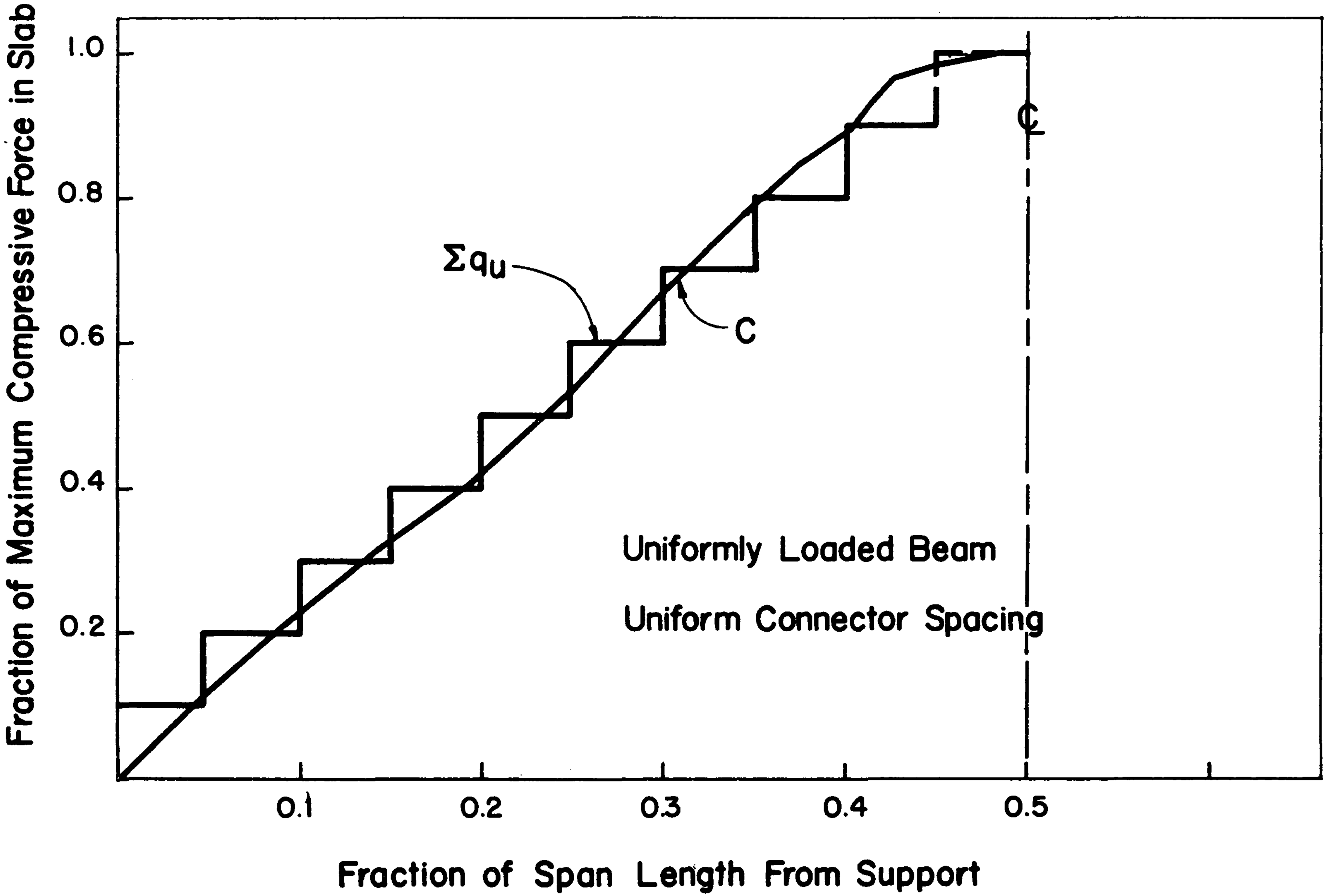


Figure 17 Theoretical and Available Compressive Force Curve for Uniform Loading

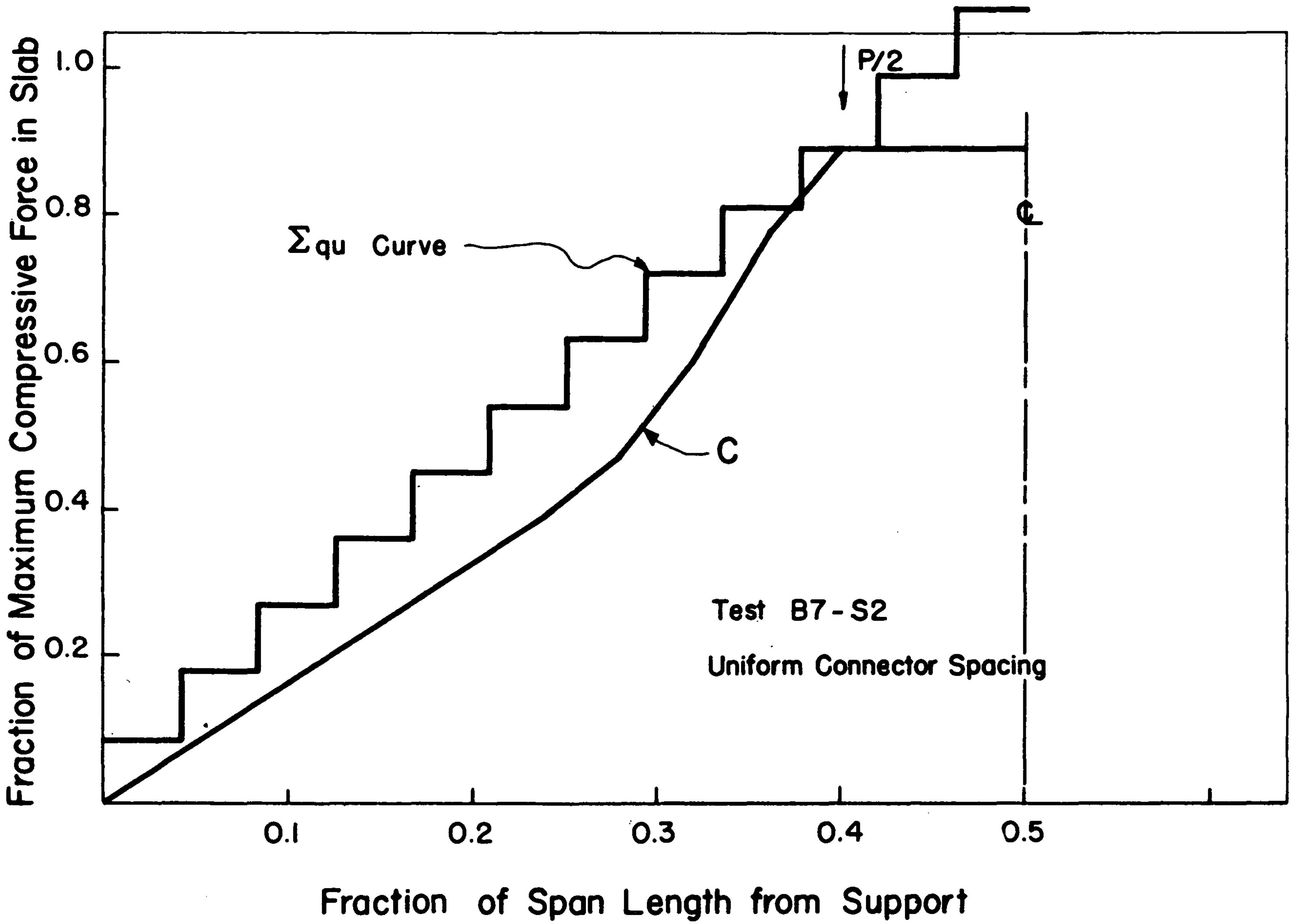


Figure 18 Theoretical and Available Compressive Force Curve for Concentrated Loading

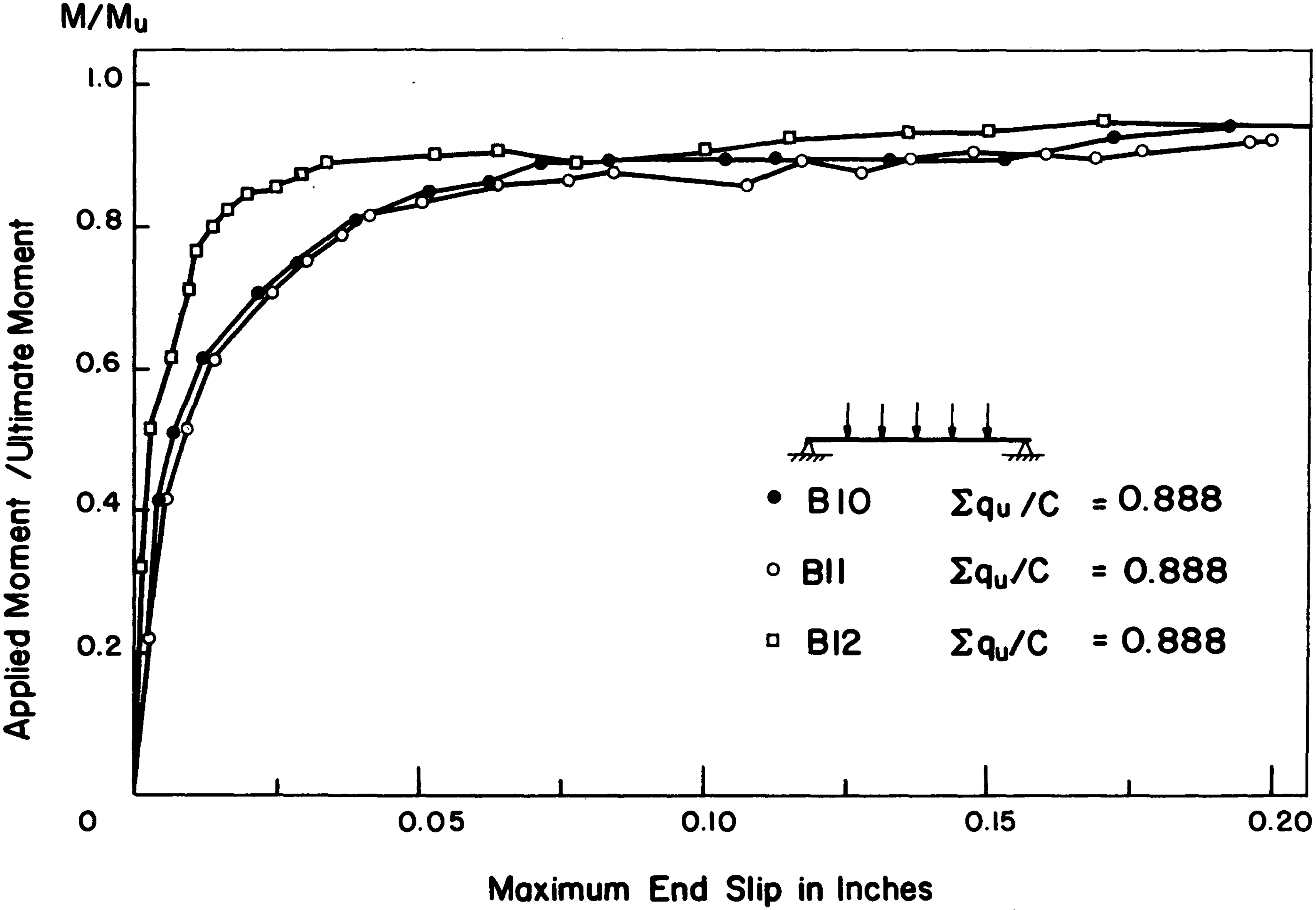


Figure 19 Maximum End Slip for Member with Loading T-13

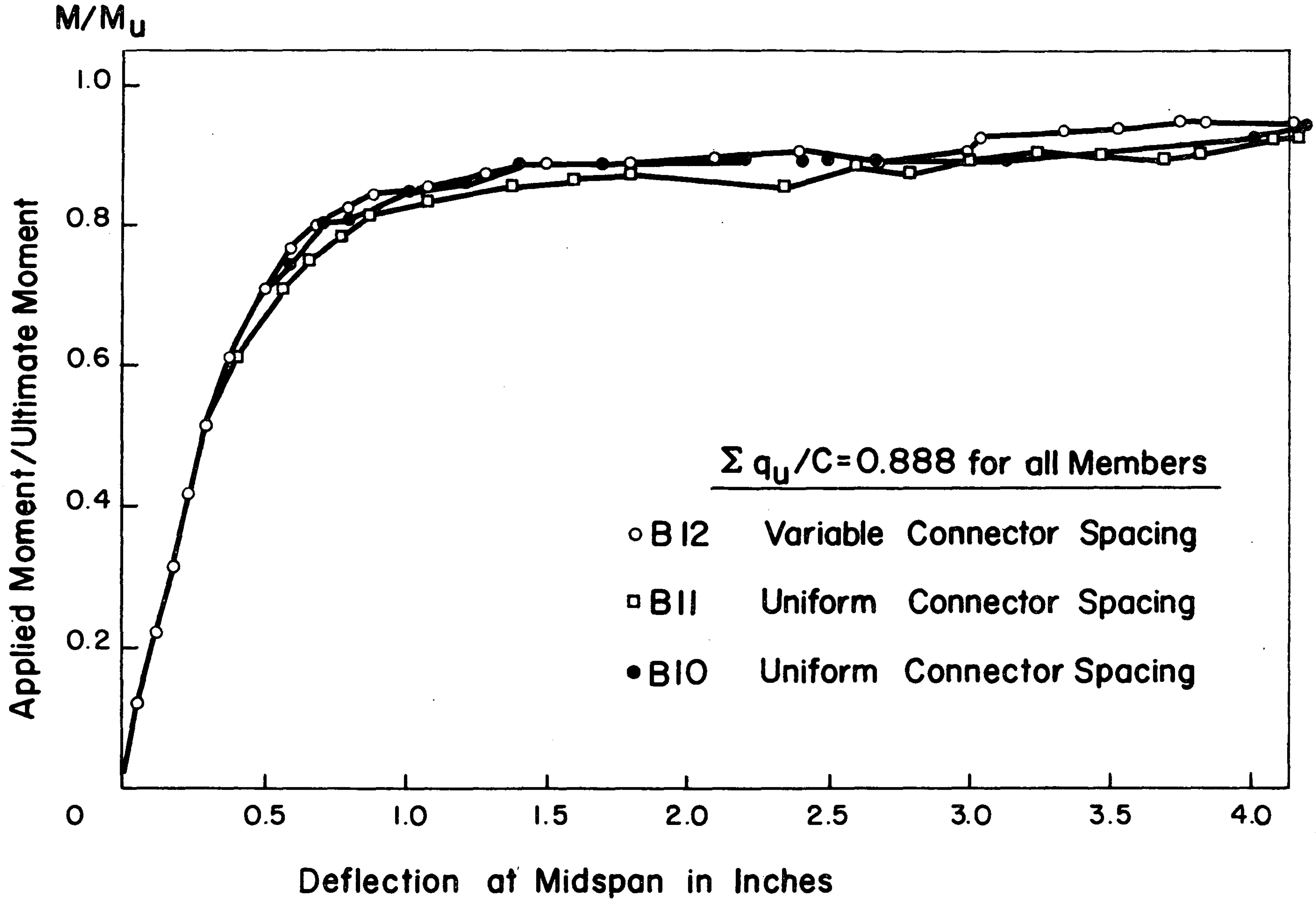


Figure 20 Load-Deflection Curves for Tests at B10, B11, and B12

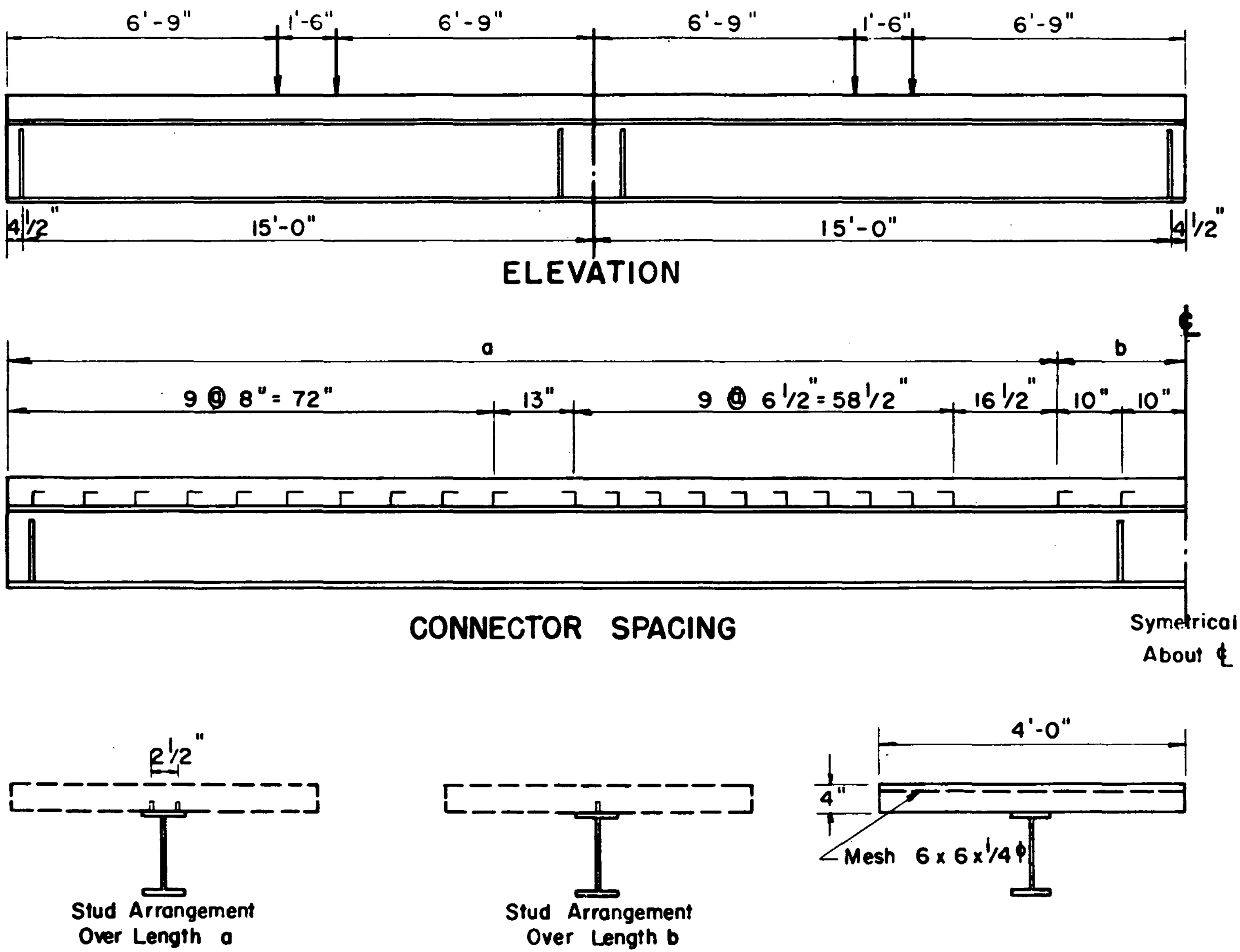
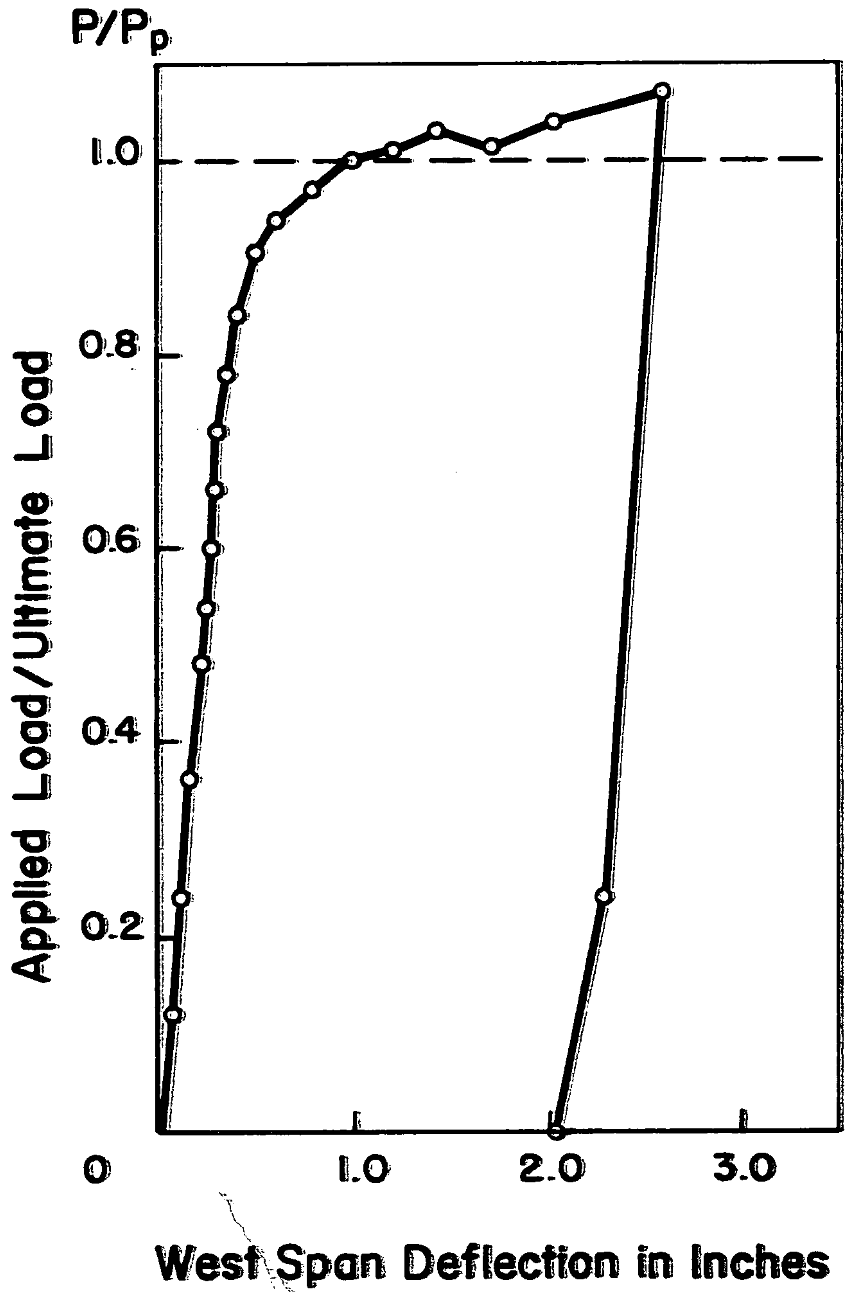


Figure 21 Dimensions and Loading for Beam B13



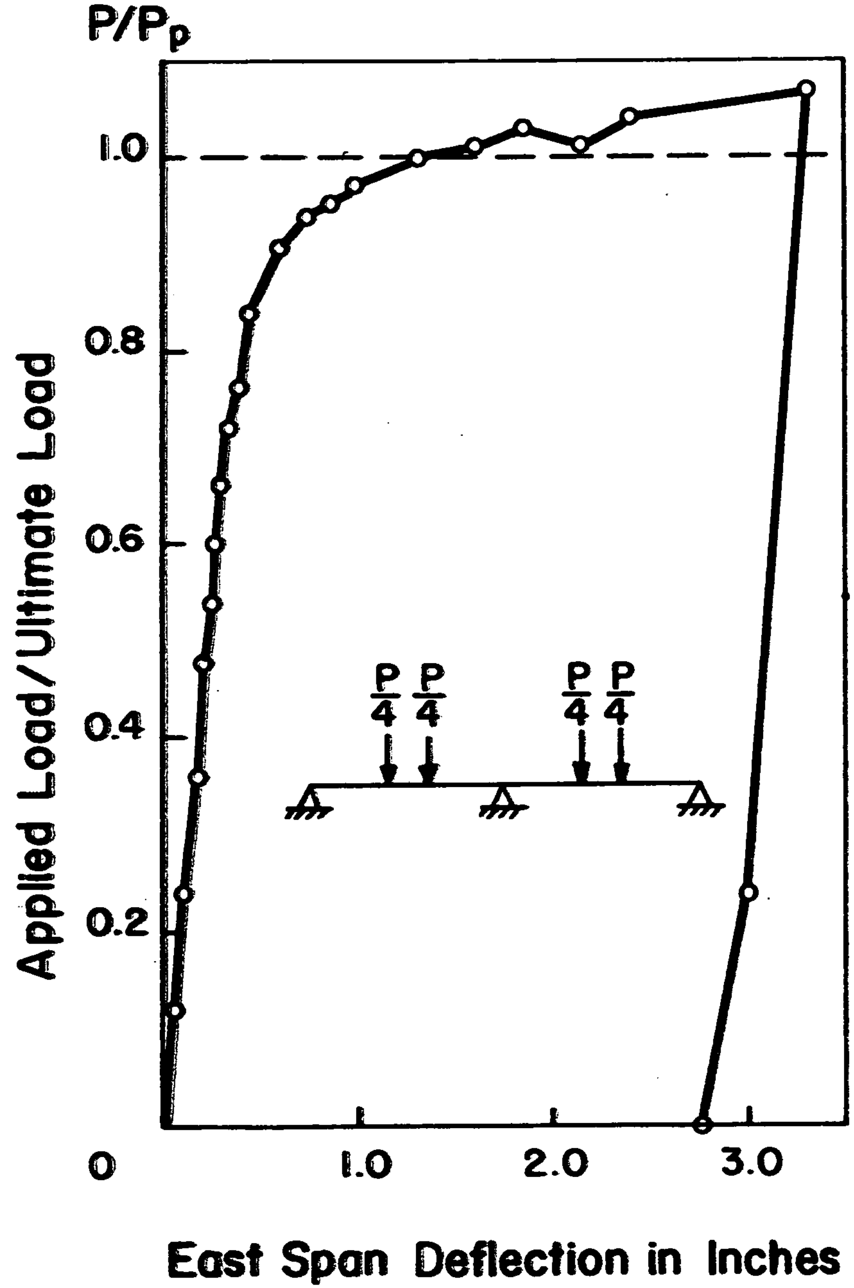


Figure 22 Load-Deflection Curves for Continuous Beam B13

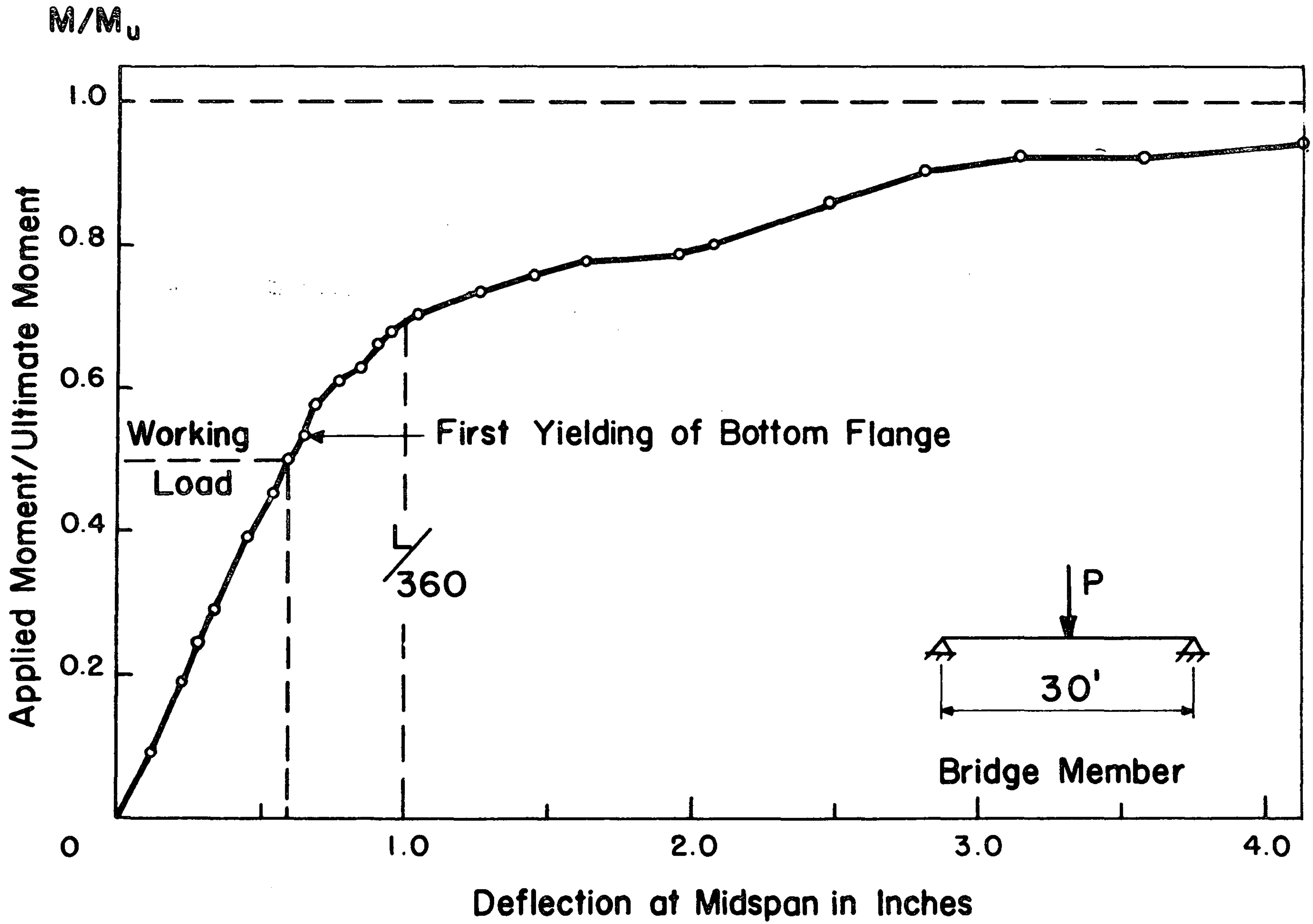


Figure 23 Definition of Working Load

Table 1 - Summary of Tests

Specimen	Reference	Steel Section	Size Conc. Slab	Test Span	Type of Connectors	Connector Spacing	Concrete Strength
BI	2	8WF17	3"x24"	10'-0"	1/2" Studs	3 @ 5 1/2"	5563
BII	2	"	"	"	"	2 @ 5 1/2"	"
BIII	2	"	"	"	"	2 @ 7"	"
B3	3	12WF27	4"x48"	15'-0"	"	2 @ 7 1/2"	3600
B4	3	"	"	"	"	2 @ 7 1/2"	"
B5	3	"	"	"	3 [4.1	4" @ 20"	"
B6	3	"	"	"	"	1 @ 7 1/2"	"
B7	4	"	"	"	"	2 @ 7 1/2"	3337
B8	4	"	"	"	"	2 @ 7 1/2"	"
B9	4	"	"	"	3/4" Studs	2 @ 15"	"
B10	4	"	"	"	"	2 @ 9"	3595
B11	4	"	"	"	"	2 @ 9"	"
B12	4	"	"	"	"	Variable	"
B21S	10	21WF68	6 1/4x72"	37'-6"	4 [5.4	6"@14 1/2"	6480
B21W	10	21WF68	6 1/4x72"	37'-6"	"	4" @ 36"	5580
B24S	10	24WF76	"	"	"	6" @ 14 1/2"	5620
B24W	10	24WF76	6 1/4x72"	"	"	6" @ 18"	5500
Bridge	6	18WF50		30'-0"	1/2" Studs	3 @ 14"	3280
1	13	18WF17	7 1/2"x30"	21'-0"	1/2"&5/8" Spiral	Variable	7380
2	13	18WF17	"	"	"	"	7040
3	13	"	"	"	"	"	7380
4	13	"	"	"	"	"	7040

* References 3 and 4 describe tests which were done as part of this research project.

Table 2 Summary of Test Results

Member	Test See Fig.1	Failure Type	Max. Test Mom. M (kip")	M_u (kip")	M_u' (kip")	Apparent Max Connector Force Kips per Connector	Max. End Slip at M
BI	TB	C	1178	1141	-	7.0 per 1/2"Stud	0.0044
BII	T3	A	1164	1141	-	10.6per 1/2"Stud	0.0089
	T4	C	1214	1141	-	12.1 " " "	0.0446
BIII	T3	A	1154	1141	-	13.4per 1/2"Stud	0.0218
	T4	A	1146	1141	-	15.4 " " "	0.0712
	T6	D	1085	1141	1051	16.6 " " "	0.0925
B3	T2	A	2708	2880	-	12.4per 1/2"Stud	0.040
	T4	A	2636	"	-	12.9 " " "	0.077
	T7	D	2514	"	2647	15.7 " " "	0.092
B4	T2	A	2571	2750	-	11.7per 1/2"Stud	0.015
	T4	A	2546	"	-	12.5 " " "	0.020
	T8	D	2614	"	2490	16.6 " " "	0.126
B5	T2	A	2695	2880	-	54.1per 4"of3[4.1	0.029
	T4	A	2758	"	-	70.5 " " " "	0.046
	T11	B	2418	"	2401	72.4 " " " "	0.207
B6	T2	D	2416	2880	2440	17.8per 1/2"Stud	0.120
B7	T2	A	2506	2730	-	11.2per 1/2"Stud	0.059
	T4	C	2554	"	2691	13.0 " " "	0.139
B8	T2	A	2618	2730	-	12.4per 1/2"Stud	0.035
	T4	A	2634	"	-	14.0 " " "	0.063
	T9	C	2491	"	2557	15.4 " " "	0.129
B9	T2	A	2586	2730	-	22.1per 3/4"Stud	0.040
	T5	A	2574	"	-	26.4 " " "	0.039
	T10	B	2514	"	2626	31.4 " " "	0.198
B10	T13	D	2596	2760	2717	13.2per 1/2"Stud	0.268
B11	T13	D	2556	2760	2717	12.8per 1/2"Stud	0.199
B12	T13	D	2626	2760	2717	13.6per 1/2"Stud	0.170
Bridge	T1	C	16740	16455	-	13.4per 1/2"Stud	0.028
B21S	T1	C	12678	11920	-	50.8per 6"of4[5.4	0.0108

Table 2 Summary of Test Results (Cont'd)

Member	Test See Fig.1	Failure Type	Max.Test Mom.M (kip")	M_u (kip")	M_u' (kip")	Apparent Max Connector Force Kips per Connector	Max. End Slip at M
B21W	T1	C	10057	11480	9589	91.7per 4"of4[5.4	0.0775
B24S	T1	A	14100	13600	-	54.3per 6"of4[5.4	0.0068
B24W	T1	A	13690	13710	-	51.4per 4"of4[5.4	0.0092
#1	T12	C	2572	2150	-	17.0per 1/2" spiral	0.0068
#2	T12	A	2362	2150	-	15.6per 1/2" spiral	0.0074
#3	T12	A	2272	2150	-	15.0per 1/2" spiral	0.0040
#4	T12	A	2402	2150	-	15.9per 1/2" spiral	0.0096

Table 3 Comparison of Test Results with Ultimate Strength Theories of Complete and Incomplete Interaction

Beam Test	Type of Failure	$\Sigma q_u/C$	Complete Interaction M/M_u	Incomplete Interaction $\Sigma M/M_u$
BI-T3	C	3.78	1.030	-
BII-T4	C	1.21	1.061	-
BIII-T6	D	0.760	0.951	1.032
B3-T7	D	0.772	0.873	0.950
B4-T8	D	0.722	0.951	1.049
B5-T11	B	0.437	0.838	1.006
B6-T2	D	0.473	0.838	0.991
B7-T4	C	0.897	0.936	0.950
B8-T9	C	0.717	0.913	0.976
B9-T10	B	0.807	0.922	0.958
B10-T13	D	0.888	0.941	0.956
B11-T13	D	0.888	0.926	0.944
B12-T13	D	0.888	0.952	0.968
Bridge	C	1.045	1.020	-
B21S	C	1.95	1.062	-
B21W	C	0.50	0.877	1.050
B24S	A	1.59	1.036	-
B24W	A	1.41	0.998	-
No.1	A	1.57	1.090	-
No.2	C	1.75	1.110	-
No.3	A	1.72	1.052	-
No.4	A	1.60	1.032	-

A Test stopped before failure

B Failure to carry additional load

C Crushing of concrete slab

D Tensile failure of connectors

E Failure by tensile cracking of slab

F Failure by connectors pulling out cone of concrete

Table 4

Table 4a Ultimate Strength of 1/2" Stud Connectors

Designation	Reference	Type of Test	H/d	Type of Failure	Concrete Strength (psi)	Max. Slip(in.)	Qult.	Stress ksi
B6-T2	2	Beam	4.5	D	3600	0.120	17.8	90.8
BIII-T6	1	Beam	4.5	D	5550	0.093	16.6	84.7
B4-T8	2	Beam	4.5	D	3600	0.126	16.6	84.7
B3-T7	2	Beam	4.5	D	3600	0.092	15.7	80.2
B8-T9	3	Beam	6.0	C	3337	0.129	15.4	78.5
3	6	Pushout	4.5	D	5000	-	14.5	74.0
4A	7	"	8.0	D	3840	0.163	14.4	73.5
4B	7	"	8.0	D	4390	0.170	13.9	70.9
B12-T13	3	Beam	4.5	D	3595	0.170	13.6	69.4
Bridge	5	"	3.8	C	3280	-	13.4	68.4
B10-T13	3	"	4.5	D	3595	0.198	13.2	67.3
B7-T4	3	"	4.5	C	3337	0.139	13.0	66.4
2	6	Pushout	4.5	D	5000	-	12.9	65.8
B11-T13	3	Beam	4.5	D	3595	0.199	12.8	65.4
P5	2	Pushout	4.5	D	3600	0.265	12.1	61.7
P6	2	"	4.5	D	3600	0.290	12.1	61.7
P8	3	"	4.5	D	3063	0.335	12.1	61.7
BII-T4	1	Beam	4.5	C	5580	0.045	12.1	61.7
P1	2	Pushout	4.5	D	3600	0.200	11.0	56.1
P4	2	"	4.5	D	3600	0.190	10.4	53.0
B1-T3	1	Beam	4.5	C	5800	0.004	7.0	35.6
P7	3	Pushout	4.5	D	3060	0.335	6.8	34.7

Table 4b Ultimate Strength at 5/8" Stud Connectors

5A	7	Pushout	6.3	D	3790	0.319	23.8	76.8
5B	7	"	6.3	D	4250	0.279	22.5	72.7

Table 4c Ultimate Strength of 3/4" Stud Connectors

6F	7	Pushout	6.7	D	4900	0.364	34.8	79.1
6B	7	"	5.2	D	4240	0.246	32.5	73.9
6A	7	"	5.2	D	3870	0.382	32.0	72.8
6G	7	"	9.3	D	4590	0.276	31.5	71.5
B9-T10	3	Beam	4.0	B	3337	0.198	31.4	71.4
6C	7	Pushout	4.0	E	4200	0.227	26.2	59.5
6D	7	"	4.0	E	3940	0.250	25.2	57.2
1	6	"	5.3	E	5000	-	24.6	55.8
6E	7	"	2.7	F	4730	0.138	23.8	54.0
6B4	7	"	5.3	E	3260	0.079	22.5	51.2
6A4	7	"	5.3	E	3360	0.099	21.2	48.3
P3	2	"	4.0	E	3600	0.220	21.2	48.1
P9	3	"	4.0	E	3063	0.180	16.0	36.3

Table 4 (cont'd)

Table 4d Ultimate Strength of 7/8" Stud Connectors

Designation	Reference	Type of Test	H/d	Type of Failure	Concrete Strength (psi)	Max. Slip(in.)	$Q_{ult.}$	Stress ksi
7H	7	Pushout	10.0	D	3440	0.278	45.0	81.3
7K	7	"	4.7	E	5340	0.257	37.6	67.3
7M	7	"	4.8	E	3000	0.160	29.8	57.5
7J	7	"	2.3	F	5380	0.102	29.0	48.7
7L	7	"	2.4	F	2480	0.147	20.9	40.3

Table 4e Ultimate Strength of 1" Stud Connectors

Designation	Reference	Type of Test	H/d	Type of Failure	Concrete Strength (psi)	Max. Slip(in.)	$Q_{ult.}$	Stress ksi
8B			4.0	E	4230	0.138	45.0	57.7
8A			4.0	E	3760	0.090	42.0	53.9

Table 5 Summary of Tensile Tests of Stud Connectors

Specimen	Reference	Stud Diameter	Ult. Stress (ksi)	Connector Force per stud
4A	7	1/2	70,700	13.9
4B	7	"	70,800	13.9
6	2	"	66,100	13.0
7	2	"	67,400	13.2
1	3	"	66,900	13.1
2	3	"	67,700	13.3
Average	-	1/2	-	13.4
5A	7	5/8	68,000	20.8
5B	7	5/8	63,300	19.4
Average	-	5/8	-	20.1
6A	7	3/4	69,900	30.8
6B	7	"	70,400	31.0
6A4	7	"	67,700	29.9
6B4	7	"	69,300	30.6
6C	7	"	72,800	32.1
6D	7	"	66,600	29.4
6F	7	"	71,700	31.6
6G	7	"	73,200	32.3
4	2	"	73,000	32.2
3	3	"	76,200	33.6
Average	-	3/4	-	31.4
7H	7	7/8	82,200	49.6
7K	7	"	67,100	46.6
7M	7	"	65,000	39.3
Average	-	7/8	-	45.2
8A	7	1	73,600	57.8
8B	7	1	73,600	57.8
Average	-	1	-	57.8

Table 6 Ultimate Strength of Spiral Connectors

Designation	Ref.	Type of Test	Type of Failure	Concrete Strength	Size of Spiral	$Q_{ult.}$ per turn	Max. Slip
4A	8	Pushout	D	2990	1/2	34.5	0.250
4B	8	"	D	2990	1/2	29.3	0.247
Average	-	-	-	-	1/2	31.9	-
5B	8	Pushout	E	3520	5/8	44.0	0.139
5A	8	"	E	3520	5/8	43.7	0.190
2-1	12	"	D	4540	5/8	42.9	0.0465
2-2	12	"	D	3080	5/8	38.5	0.0675
Average	-	-	-	-	5/8	42.3	-
1-1	12	Pushout	D	5120	3/4	58.3	0.0225
6B	8	"	E	3250	3/4	54.9	0.075
6A	8	"	E	3250	3/4	52.3	0.088
1-2	12	"	E	2965	3/4	51.1	0.0340
Average	-	-	-	-	3/4	54.2	

NOTE

tension	8	Ult. Strength of material	1/2	12.4 x 2.0 = 24.8 kip/turn
"	8	" " "	5/8	19.0 x 2.0 = 38.0 kip/turn
"	8	" " "	3/4	29.2 x 2.0 = 58.4 kip/turn

Table 7 Ultimate Strength of Channel Connectors

Designation	Ref.	Type of Test	Size of Channel	Concrete Strength	Load per inch	Type of Failure
B5	2	Beam	3 [4.1	3600	18.1	C
3C3H3	7	Pushout	"	3920	14.9	D
3C3H2	7	"	"	3310	12.6	D
P2	2	"	"	3600	11.9	E
3C3H1	7	"	"	2810	10.5	D
B21W	7	Beam	4 [5.4	5580	22.9	C
4C3W2	7	Pushout	"	4430	20.4	D
4C3C11	7	"	"	6320	19.7	D
4C3C9	7	"	"	5340	19.4	D
4C3C10	7	"	"	5740	18.7	D
4C3C7	7	"	"	4140	17.1	D
4C3C8	7	"	"	4770	16.4	D
4C3F4	7	"	"	4690	16.2	D
4C3C6	7	"	"	3500	15.8	D
4C3C5	7	"	"	3470	15.2	D
4C3F3	7	"	"	4600	15.1	D
4C3S2	7	"	"	1970	15.0	E
4C3W1	7	"	"	2810	15.0	D
4C3C4	7	"	"	3140	13.2	D
B24W	7	Beam	"	5500	12.8	A
4C3C1	7	Pushout	"	2070	12.5	D
4C3F2	7	"	"	2650	12.4	D
4C3F5	7	"	"	3080	12.3	D
4C3C2	7	"	"	2300	12.1	D
4C3D2	7	"	"	3370	11.6	D
4C3C3	7	"	"	2570	11.2	D
4C3D1	7	"	"	2990	9.9	D
4C3F1	7	"	"	2580	9.6	D
4CS1	7	"	"	1340	8.0	E
4C5T8	7	"	4 [7.25	5050	21.8	D
4C5T7	7	"	"	4360	17.1	D
4C4T	7	"	"	4010	16.4	D
4C5F	7	"	"	2170	16.4	D
4C5T6	7	"	"	3530	15.8	D
4C5T3	7	"	"	3130	15.1	D
4C5T2	7	"	"	2910	14.5	D
4C5T4	7	"	"	3190	14.2	D
4C5T5	7	"	"	3310	14.1	D
4C5S	7	"	"	2720	14.0	D
4C5T1	7	"	"	2300	13.2	D
5C3H2	7	"	5 [6.7	3260	15.2	D
5C3H1	7	"	"	3170	14.9	D

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