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BEAM WEB CONNECTIONS

WITH COPED TOP FLANGES

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by

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ABSTRACT

A summary and evaluation of recent bolted connection research by Fisher and Struik[1] has led to recommendations of higher allowable bearing stresses based on a new bearing strength model. To verify the applicability of the new bearing strength criteria to web shear connections, the American Institute of Steel Construction sponsored experimental research of double-angle, beam web-column connections at the University of Texas.

This study uses the experimental data and observations obtained in the University of Texas tests to generate and to test the validity of various behaviour models. The ultimate shear strengths predicted by the behaviour models are compared to resistances calculated from current CSA S16.1-1974 specifications. Recommendations are made for design limits which can be used to achieve a rational design of beam web connections with coped top flanges.

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NOMENCLATURE

Explanation of the symbols used in the sections of this thesis appears in those specific sections. In addition, the following symbols are included here for convenience.

AN = Net Area

AG = Gross Area

D = Bolt Diameter

EH = Horizontal End Distance

EV = Vertical End Distance

Fs = Shear Stress

Fy = Yield Strength

Fu = Ultimate Tensile Strength

G = Bolt Gauge

m = Number of Vertical Lines of Bolts

n = Number of Bolts

P = Bolt Pitch

t = Web Thickness

Tu = Ultimate Shear Stress

V = Shear Force

1. INTRODUCTION

1.1 HISTORICAL NOTES

Recent bolted connection research summarized and evaluated by Fisher and Struik[1] led to the incorporation of higher allowable bearing stresses by the Canadian Standards Association S16.1-1974, "Steel Structures for Buildings - Limit States Design"[2] and into the "Specification for Structural Joints Using ASTM A325 or A490 Bolts"[3]. Because these recommendations were based upon experimental data which were collected entirely from the testing of simple tensile plate splices, the Canadian Institute of Steel Construction (CISC) arranged for tests of simple double-angle, single vertical line of bolts, beam-column connections to verify the applicability of the new bearing strength criteria to web shear connections. These tests were carried out at the University of Toronto.

While a reasonable margin of safety was found for a connection where the beam was uncoped, coping of the top flange resulted in a significant reduction in the connection strength. The test results suggested that the bearing strength criteria as presented, did not represent the critical mode of failure and that a shearing out of a block of web, a "block shear" failure, occurred before the theoretical bearing strength of the web was reached.

Figure 1 illustrates this failure model where the resistance to "block shear" is provided by the tensile resistance of the web across plane AA and the shear resistance of the web along plane BB. This model analytically yielded connection strengths which closely match the experiment results for single vertical line of bolts connections with top flanges coped.

The "block shear" model was incorporated into the Commentary on the AISC Specification, Section 1.5.1.2, and into a Technical Memorandum issued by the CISC in May 1978. However, while the AISC and CISC gave guidelines for connections with one line of bolts, and a coped top flange, no recommendations were given for the case where two vertical lines of bolts are present. Whether or not the simple "block shear" model would be valid for an increased eccentricity was questioned.

To further investigate the behaviour of connections with coped top flanges, the American Institute of Steel Construction sponsored experimental research at the University of Texas at Austin. Twenty-three tests were conducted on bolted shear connections with framing angles on both sides of the web, as shown in Appendix D.

The results of the research showed that while current specifications are adequate for determining the strength of connections with one line of bolts, they are not satisfactory for connections having two lines of bolts. The "block shear" failure model predicted the ultimate shear well for connections with one line of bolts, but overestimates the capacity of connections with two lines of bolts. The ratio of test load to allowable load, calculated according to current provisions of the AISC specification, ranged from 2.50 to 4.37 for connections with one line of bolts and from 1.63 to 1.96 for connections with two lines of bolts[4]. Thus the immediate conclusions reached in the University of Texas research suggest that the "block shear" failure model is not applicable to connections with two lines of bolts and that further work is necessary to develop a design model for such connections.

1.2 OBJECTIVES AND SCOPE

A major disadvantage of the "block shear" failure model is that it does not consider the effect of eccentricity on the ultimate shear strength of a connection. For connections with only one line of bolts the eccentricity is small and thus this deficiency does not appear to be significant for usual connection proportions. However, for connections with two lines of bolts in the web, the application of a model which does not account for eccentricity results in an overestimation of connection strength.

Observation of test specimens at the University of Texas showed the appearance of yield lines associated with bending stress in the web of the beam as the ultimate shear was reached; also the ultimate strength was reached when a tear commenced at the end of a beam. This behaviour suggested that the moment created by eccentricity significantly affected the strength of the connection.

The objectives of this paper are:

(1) To develop a design model which considers eccentricity and is applicable to connections with either one or two lines of bolts.

(2) To investigate the validity and accuracy of the current CSA S16.1 "Steel Structures for Building - Limit States Design".

(3) To develop a design aid for use in predicting the ultimate shear strength of a connection with two lines of bolts, when the top flange is coped.

The experimental data and observations obtained in the University of Texas study for AISC are used to assist the development of analytical models, and to test the validity of the models.

2. MOMENT MODEL FOR WEB STRENGTH

2.1 DEVELOPMENT OF THE MOMENT MODEL

In a bolted clip angle type of connection there is a moment created by the eccentricity of the connection. This moment can be simply expressed as:

$$Me = V \times e$$

where V = vertical shear on connection

e = eccentricity, the distance from the column face to the centoid of the bolt pattern in the beam web.

This expression ignores any moment which may exist at the column face. In order to investigate the means by which the web resists this moment, a section bounded by the dotted lines in Figure 2 will be analyzed. Figure 3 shows the forces which must be acting on this section to satisfy static equilibrium. The moment "Mw" could be created by a number of possible stress distributions. Several of the distributions which were considered are shown in Figure 4. Because of the large moment which must be resisted by the web for equilibrium, either distribution (c) or (d) were considered likely possibilities.

University of Texas tests on connections with two vertical lines of bolts indicated that failure at ultimate load occurred by rupture beginning at the beam end of plane AA and propagating along AA. On the basis of this evidence it was decided that the tensile stress on plane AA would approach the ultimate tensile stress, Fu, at failure. The ultimate shear stress which could occur would be in the range of 0.6 to 0.7 Fu. An arbitrary value of 0.65 Fu was chosen for the shear stress at ultimate load along plane AA.

The first model to be investigated in detail is shown in Figure 7. The horizontal shear, 'HS1', which occurs along plane AA can be expressed as:

$$HS1 = 0.65\ Fu\ (AN1) \qquad \text{eq. 1}$$

where AN1 = Net area along plane AA

Ultimate shear of the connection can be calculated by applying the equations of equilibrium to the model. Application of equilibrium in the 'x' direction will give:

$$Fy\ t\ (L2-YC-D-\frac{1}{8.}) - HS1 - Fy\ t\ (YC-\frac{D}{2.}-\frac{1}{16.}) = 0.0 \qquad \text{eq. 2}$$

$$\therefore\ YC = \frac{Fy\ t\ (L2 - \frac{D}{2.} - \frac{1}{16.}) - HS1}{2\ Fy\ t} \qquad \text{eq. 3}$$

where YC = distance from plane AA to the point 0, as shown in Figure 7.

Taking moments at point 0 will give the following expression for the ultimate shear strength, V;

$$V = \frac{1}{EC}\Big\{HS1(YC) + Fu\ t\ (EH-C2)\left[L1 - \frac{(EH-C2)}{2.0}\right] + \frac{Fu\ t\ (G-C1)\quad G}{2.0} + Fy\ t\ (EV-C2)\left[L2 - \frac{(EV-C2)}{2.0}\right] + \frac{Fy\ t\ (P1-YC-C2)^2}{2.0} + \frac{FY\ t\ (YC-C2)^2}{2.0}\Big\} \qquad \text{eq. 4}$$

After V has been calculated, the vertical shear, 'VS1', which occurs along plane BB can be found from vertical equilibrium;

$$VS1 = V - Fu\ (AN1) \qquad \text{eq. 5}$$

VS1 cannot exceed the maximum shear which the web can resist along the plane BB. Therefore,

$$VS1 \leq 0.65\ Fu\ (AN2) \qquad \text{eq. 6}$$

where AN2 = Net Area along plane BB.

If the value for VS1 calculated from equation 5 exceeds the limit for VS1 given by equation 6, then the ultimate shear capacity of the connection is limited by the shear capacity of the web along plane BB. The full bending moment capacity of the web will not be reached and thus since VS1 is known, equation 5 can be solved for V.

$$\text{i.e.}\quad V = VS1 + Fu\ (AN1) \qquad \text{eq. 7}$$

where VS1 is given by equation 6.

$$\text{Thus,}\ V = 0.65\ FU\ (AN2) + Fu\ (AN1) \qquad \text{eq. 9}$$

Results obtained from this model were conservative in comparison to the corresponding experimental values. It was felt that this was in part due to the tensile and compressive stresses along plane BB and the tensile stress along plane AA not being extended across the bolt holes. The clamping effect of bolt pretensioning plus bearing contact of the bolt will help to spread the stress to the web area surrounding the bolt hole. Figure 8 shows this revision.

Results obtained for the stress distribution shown in Figure 8 were an improvement over those obtained using Figure 7. All but one of the calculated ultimate shears were within 10% of the experimental result.

It was subsequently proposed that the tensile stress acting on plane AA may not reach the ultimate stress value along the total length of the plane. While the ultimate stress would be reached at the edge of the web, as shown in Figure 5, the maximum stress at the other end of the plane would be the yield stress. This revised mode showed poorer correlation with the experimental connection strength and thus was discarded.

Observation of the test specimens at failure showed that substantial yielding occurred in the region surrounding the bottom left hand side of the connections, as indicated in Figure 2. This observation led to the proposal that the maximum tensile stress value occurring along plane BB be equal to the ultimate tensile stress, as shown in Figure 6. While this revision did not have a significant effect on the results given by model, it was thought to give a more accurate representation of the observed failure mechanism.

The final form of the model used to represent the distribution of stresses at ultimate connection shear is shown in Figure 9. Equation 1 will calculate the horizontal shear, 'HS1', and application of equilibrium in the 'x' direction will give:

$$YC = \frac{Fy\ t\ (L2) - HS1}{1.5\ Fy\ t + 0.5\ Fu\ t} \qquad \text{eq. 9}$$

Taking moments at point 0 will give the ultimate shear strength as:

$$V = \frac{1}{EC}\left[HS1\ (YC) + \frac{Fu\ t\ (L1)^2}{2.0} + \frac{Fy\ t\ (YC)^2}{2.0} + \frac{Fy\ t\ (L2-YC)^2}{2.0} + \frac{(Fu-Fy)\ t\ (YC)^2}{3.0} \right] \qquad \text{eq. 10}$$

Vertical equilibrium will give the vertical shear long plane BB, i.e. 'VS2', as:

$$VS2 = V - FU\ t\ (L1) \qquad \text{eq. 11}$$

If VS2 exceeds the limit placed on vertical shear along plane BB, equation 6, then the ultimate shear strength of the connection will be limited to:

$$V = 0.65\ Fu\ (AN2) + Fu\ t(L1) \qquad \text{eq. 12}$$

When the moment model is used for connections having one line of bolts, equation 12 becomes the dominant expression. The shear resistance capacity becomes more important because of the reduced eccentricity decreasing the moment on the connection. It should be noted that equation 12 is similar to the expression given by the "block shear" failure model for the ultimate shear.

The ultimate shear capacity of connections with slotted holes can also be predicted by the "moment" model. However, because of the increase material removed along plane AA the tensile resistance will not be able to develop across the full width of the section. Figure 10 shows the recommended tensile stress distribution along plane AA.

The concepts used in the "moment" model for coped flanges can also be applied to connections with uncoped flanges. A connection with an uncoped flange will have a higher shear resistance for the following reasons:

(1) the additional shear component occurring along the top of the model, as shown in Figure 11.

(2) moment capacity of the flange.

The contribution from the flange can be shown to be small in comparison to that of the web and has been neglected (Appendix C). Horizontal shear, "TS1", at the top of the model is taken to occur at a distance 'k' below the top of the flange and is expressed as:

$$TS1 = 0.65\ Fu\ (L1)\ t \qquad \text{eq. 13}$$

This additional force will change the equations of equilbrium stated previously for the coped condition.

Fx = 0.0 becomes:

$$\begin{aligned} &Fy\ t\ (L2-YC) + TS1 - HS1 - Fy\ t\ (YC) \\ &- (Fu - Fy)\ t\ (YC)\ 0.5 = 0.0 \qquad \text{eq. 14} \end{aligned}$$

solving for YC one obtains:

$$YC = \frac{Fy\ t\ (L2) - HS1 + TS1}{1.5\ Fy\ t + 0.5\ Fu\ t} \qquad \text{eq. 15}$$

Now summing moments at "O" and solving for V yields:

$$\begin{aligned} V = \frac{1}{EC}\Big[&HS1\ (YC) + TS1\ (L2-YC) + \frac{Fut(L1)^2}{2.0} + \frac{Fyt(YC)^2}{2.0} \\ &+ \frac{Fy\ t\ (L2-YC)^2}{2.0} - \frac{(Fu - Fy)\ t\ (YC)^2}{3.0}\Big] \qquad \text{eq. 16} \end{aligned}$$

After finding V, and VS2, the shear resistance of the web can be calculated by following the procedure described for the coped condition.

2.2 COMPARISONS OF MODELED AND MEASURED ULTIMATE SHEAR STRENGTH

Ultimate shear capacity of the connections tested at the University of Texas was calculated using the "moment" model and the results recorded in Table 1. To gain a perspective on the relationship of the calculated to the measured capacity of the connections, the "moment" model solution was expressed as a percentage of the corresponding experimental result. The degree of correlation between predicted and measured strengths of coped beams is presented in Figures 12 and 13. For test numbers 18-8 & 18-25, which had a single line of bolts with slotted holes, the "moment" model overestimated the strength of the connection. Two factors which may account for this overestimation are:

(1) The "moment" model for slotted connections with two lines of bolts, Figure 10, suggests that the tensile stress along plane AA will be developed across the entire 'EH' distance. However, for only one line of bolts, this distribution may not be achieved, because the length of the slotted holes severely reduced the width of the existing web steel along the plane AA. For test case 18-8 if the tensile stress along plane AA was developed only across the existing web steel, (i.e. EH - BOLT HOLE RADIUS) then the calculated capacity would be reduced to 110% of the measured capacity.

(2) The quality of workmanship used in forming the slots is highly variable because of the variety of methods used by fabricators. Thus the workmanship may have had an effect on the strength of the web.

Good correlation was achieved with the test cases having either a single line of five bolts and no slotted holes, or two lines of bolts; the calculated strengths are within 7% of the measured values.

For the test cases having a single line of three bolts with a pitch of six inches, such as 18-9, 18-20, 18-21, 18-22, 18-23, 18-24 and 18-25, the shear pedicted by equation 12 cannot be reached because the bearing strength of the web limits the amount of vertical shear that can occur along the plane BB. Bearing strength can be approximated by summing the individual bolt loads. The top bolt is limited by the load required for fastener tearout, i.e.:

$$Bu = 2t \left(EV - \frac{d}{2} - \frac{1}{16}\right) Tu \qquad \text{eq. 18}$$

where EV = vertical end distance

Tu = ultimate shear stress = 0.65 Fu

d = bolt diamter

The load on the remaining bolts is limited by the simple bearing strength of the plate material, i.e.:

$$Fu = 3\ t\ d\ Fu \qquad \text{eq. 19}$$

Application of equations 18 and 19 to test case 18-9 reduced the predicted strength from 157% to 91.4% of the measured value.

The degree of correlation between predicted and measured strengths of the uncoped test cases is presented in Figure 14. Overestimation of the strength of the connections by the "moment" model results because:

(1) the "moment" model not being the critical mode of failure for beams without copes.

(2) full web shear, (i.e. 0.66 x Fy x Aw) is developed by the connection before the load predicted by the "moment" model is reached.

The "moment" model provides a design procedure to check that particular mode of failure. This mode may not be critical and thus the connection resistance would be limited by other requirements.

2.3 COMPARISON OF THE 'MOMENT' MODEL AND OTHER MODELS

Ultimate shear capacities of coped connections tested at the University of Texas were calculated by using a "block shear" model and a "tension resistance" model. The "block shear" model, Figure 1, expresses ultimate shear as the sum of shear resistance along plane BB plus tensile resistance along plane AA.

i.e. $V = 0.65\ Fu\ (AN2) + Fu\ (AN1)$ eq. 20

where AN2 = Net area along plane BB

AN1 = Net area along plane AA

The "tension resistance" model assumes that the maximum tensile resistance which can occur along plane AA, over the gross area of the web will give an approximate estimate of the ultimate shear.

i.e. $V = Fu\ (AG)$ eq. 21

where AG = Gross Area = $(G + EH)\ t$

G = bolt gage, Figure 7

EH = horizontal end distance, Figure 7

Results obtained from the "block shear" and "tension resistance" models are recorded in Tables 2 and 3. In Table 4, these results are expressed as a percentage of the experimental value. The degree of correlation between predicted and measured results is presented in Figures 16 and 17. While the "block shear" model gives good results for connections with only one line of bolts, it greatly overestimates the strength of connections with two lines of bolts. Some correlation with the experimental results is shown by the "tension resistance" model for connections with two lines of bolts, provided that the bolt holes are not slotted. Slotting of the holes resulted in the "tension resistance" model overestimating the strength of the connection, as shown by test specimen 18-11. For connections having a single line of bolts, the "tension resistance model" greatly underpredicts the strength.

As was indicated earlier, the "moment" model will give satisfactory results for connections with either one or two lines of bolts.

2.4 COMPARISON OF CURRENT CSA S16.1-1974 SPECIFICATION REQUIREMENTS WITH EXPERIMENTAL BEHAVIOUR

Specifications currently require five design checks to be completed to satisfy the adequacy of shear capacity of a bolted double angle beam web to column connection. The five design checks are:

(1) Shear capacity of the beam web.

(2) Bearing resistance, considering vertical end distance.

(3) Bearing resistance, considering horizontal end distance.

(4) Bolt shear including eccentric load effect.

(5) "Block Shear" model.

Shear resistance of a beam web is expressed in S16.1-1974, Clause 13.4.1 as:

$$Vr = \phi \; Aw \; Fs \qquad \text{eq. 22}$$

where $\phi = 0.90$

Aw = shear area = $h \times w$

Fs = shear stress = $0.66 \times Fy$

Bearing resistance of a connection is expressed in S16.1-1974, Clause 13.10 (c) as:

$$Br = \phi \; t \; n \; e \; Fu \qquad \text{eq. 23}$$

where $\phi = 0.67$

n = number of bolts

e = end distance (either horizontal or vertical)

Resistance of a bolt group to an eccentric load can be calculated by using the tables provided in Part 3 of the "Limit States Design Steel Manual". The CISC computer program, from which the tables were established, was used here for all eccentric load effects.

In a technical memorandum issued in May 1978, the CISC suggested the following rule to check for block shear.

$$V_r = \phi \, w \left[L-(n-\tfrac{1}{2}) \, d \right] 0.53 \, Fu + \phi \, w \, (e_o-\tfrac{1}{2}d) \, Fu \qquad \text{eq. 24}$$

where ϕ = 0.90

w = web thickness

L = length of shear plane BB shown in Figure 1

e_o = edge distrance from centre of bolt hole to edge of web taken horizontally

Each of these five design checks were applied to the coped connections tested at the University of Texas and the results recorded in Tables 5, 6, 7 and 8.

In table 6 the bearing resistance with regard to both vertical and horizontal end distance is calculated by using equation 23. The performance factor, ϕ, used in equation 23 is equal to 0.67, instead of 0.9. This reflects a desire to cause member rather than connection behaviour to govern the structural behaviour at ultimate. Therefore, to relate the ultimate bearing resistance allowed by the specifications to the measured ultimate shear, the value found from equation 21 will be divided by 0.9 rather than 0.67, as shown in Table 6. In order to calculate the ultimate shear predicted by the remaining design checks the performance factor will be deleted by dividing the formula for each of the checks by 0.9.

The CISC computer program for eccentric load on bolt groups produces a coefficient 'C' dependent on geometry of the group and eccentricity of the load. To determine the capacity of a given connection, for a given eccentricity, the appropriate coefficient is multiplied by the smaller of (1) shear resistance of a bolt and (2) the bearing capacity of the connected material. Table 8 contains a list of the coefficients and the capacities of the test connections. Because of the small end distance present in the test connections, the bearing resistance of the beam web is well below the shear capacity of the bolts.

A summary of the design checks is presented in Table 9 and the critical design check is expressed as a percentage of the experimental result. In Table 9, it is shown that for all the test cases the capacity predicted by the eccentric load design check is critical.

For connections with a single line of bolts the eccentric load criteria gave very low estimates of the ultimate shear strength. The average calculated shear expressed as a percentage of the measured shear was only 42.1%. Although the ultimate shears predicted by the block shear model had the best correlation with the measured shear, the eccentric load formula always gave lower ultimate shears and thus was the critical design check.

For connections with two lines of bolts, the eccentric load criteria gave very low strength estimates when one or both of the end distances were equal to the minimum value. In such cases the entire connection strength was limited in direct proportion to the weakness in one fastener. Improved correlation with the experimental results was obtained for connections which had both horizontal and vertical end distances equal to 2.0 inches (i.e. tests 18-12, 18-17 and 18-19).

The results obtained from application of the "moment" model, shown in Figures 17 and 18, provide better correlation with experimental results than those calculated using the critical resistance according to the current CSA S16.1 specification. As was mentioned previously, the appropriate performance factors were included in the design checks required by CSA S16.1 and the results divided by 0.9.

3. MODIFIED BEARING STRENGTH CRITERIA

3.1 ULTIMATE STRENGTH CRITERIA

While the "moment" model gives a good explanation of the failure mode for coped connections, it is rather complex for use as a design aid. An alternate attempt to develop a more convenient design approach was the revision of the existing CISC computer program for eccentric loads on bolt groups to include the effects of bearing and end or edge tear out.

Recall that the CISC computer program for eccentric loads on bolt groups produces a coefficient 'C' dependent on the geometry of the group and the eccentricity of the load. To determine the capacity of a given connection, for a given eccentricity, the appropriate coefficient is multiplied by the smaller of (1) shear resistance of a bolt and (2) the bearing capacity of the connected material. This approach considers the nonlinear load-deformation response of a single fastener as a basis for determining the ultimate strength of a fastener group. The ultimate strength of the fastener group is assumed to be reached when the ultimate strength of the fastener farthest from the instantaneous center is reached.

For a given fastener configuration and eccentricity a trial and error procedure is used to locate the instantaneous center. First, a trial location of the instantaneous center is selected, then the fastener located farthest from the instantaneous center is assumed to have the maximum fastener deformation. The deformation occurring at each fastener is assumed to vary linearly with its distance from the instantaneous center. Thus, the deformation of other fasteners can be determined from:

$$\Delta_i = \frac{r_i}{r_{max}} \Delta_{max} \qquad \text{eq. 25}$$

where Δ_{max} = maximum fastener deformation

Δ_i = deformation of "i" fastener

r_{max} = distance from instantaneous center to farthest fastener

r_i = distance from instantaneous center to "i" fastener

The fastener load corresponding to Δ_i is calculated by using Crawford and Kulak load deformation relationship[1].

$$\text{i.e.} \quad R = R_{ult} \, (1 - e^{-u\Delta})^{\lambda} \qquad \text{eq. 26}$$

where R = shear force on the fastener

R_{ult} = ultimate shear load of the fastener

Δ = deformation of the fastener

u, λ = regression coefficient

e = base of natural logarithms

Equilibrium of horizontal and vertical forces, plus summation of moments around the instantaneous center can now be used to check the assumed location of the instantaneous center. Once the location of the instantaneous center has been established, then the strength of the fastener configuration is known and thus the "C" coefficient can be calculated.

To include the effect of bearing and end or edge tear out, it was decided that if the fastener shear, "R", calculated from equation 26, exceeded the load required for fastener tearout, or excessive material deformation, then the fastener shear force would be reduced to the tearout or excessive deformation load. For end or edge tearout limiting the fastener shear force, it was assumed that the fastener shear load acts parallel to the direction of tearout, i.e. either horizontally or vertically. Although there will be some variation in the fastener load direction from the horizontal or vertical direction, in order to simplify the calculations this variation will be neglected.

The following three revisions were made to the CISC computer program for eccentric loads on bolt groups:

(1) The load on the bolt located at the bottom of the line of bolts nearest the end of the beam is limited to the force required for horizontal tearout of the web material at that location.

(2) Revision 1 with the additional feature that the load on the highest bolt in the line of bolts located farthest from the end of the beam is limited to the force required for vertical tearout of the web material at that location.

(3) Revision 2 with the further limit on the maximum load which can be applied to any bolt is limited to 3.0 t d Fu, where: t = web thickness, d = bolt diameter and Fu = ultimate tensile stress of web.

Thus, in the third form, all of current specification requirements for bearing have been added to the eccentric load program which had been developed for fastener shear. The revisions will adjust the coefficients produced by the computer program to reflect a bearing mode of failure when it is critical. The ultimate strength of a connection can then be calculated by choosing the appropriate coefficient, 'C', from the computer output and multiplying it by the shear strength of the bolts;

$$B_r = C\ 0.6\ m\ (AB)\ (FUB) \qquad \text{eq. 27}$$

where m = number of shear planes

AB = cross-sectional area of one bolt

FUB = ultimate tensile stress of the bolt

Each of the revisions was used to calculate the ultimate shear capacity of the connections tested at the University of Texas and the results recorded in Table 10. Figures 19 and 20 illustrate the degree of correlation between the ultimate shear strengths calculated using the proposed revisions and the measured values.

A study of the Texas tests on connections with two lines of bolts showed that connections with an unsymmetrical arrangement of bolts, such as test specimens 18-10, 18-11, 18-12, 18-18 and 18-19, showed no increase in strength over similar symmetrical connections with fewer bolts. For example, test 18-10 had 3 bolts at 3 inch pitch in the bolt lines closest to the end of the beam and 2 bolts at 6 inch pitch in the line farthest from the end of the beam. And test 18-16 had the same end distances as test 18-10 but only two bolts at 6 inch pitch in both lines. Although the analysis predicted that test 18-10, with its additional bolt, will have an ultimate shear strength of 183. kips, as compared to 94.1 kips for test 18-16, the measured strength was the same for both tests. An investigation of test specimens 18-10, 18-11, 18-12, 18-18 and 18-19 after failure found no significant material deformation around the bolt located between the upper and lower rows of bolts. Because of the ineffectiveness of this bolt to increase the strength of a connection, the contribution of the bolt to the strength of a connection is insignificant and the presence of this bolt could be neglected when calculating the strength of a connection. For tests 18-16 and 18-17, having two bolts per line, the predicted strength showed good correlation with the measured values.

Figure 19 illustrates the good correlation achieved between the strengths calculated using Revision 3 and the measured values for single line of bolts connections having five bolts with a pitch of 3.0 inches. However, for connections having a single line of three bolts with a pitch of six inches, the predicted strengths were conservative with respect to the measured values. It was felt that the large pitch of the bolts resulted in a connection with sufficient stiffness to reduce the effective eccentricity and prevent horizontal tearout of the bottom bolt from being part of the failure mode. Observation of the test specimens at failure showed no material deformation in the horizontal direction at the bolt locations. Figure 21 shows the improvement in predicted results achieved by removing horizontal tearout as a limiting factor, and assuming an effective eccentricity equal to half the distance between the centerline of the bolts and the column face. However, because the reason for revising the CISC computer program was to develop a convenient design approach, it was decided that this improvement in the analytical results did not compensate for the corresponding increase in the complexity of the design procedure and thus the improvement was not incorporated into the program.

The degree of correlation between revision 3 and the experimental results is presented in Figures 22 and 23. For tests 18-10, 18-11, 18-12, 18-18 and 18-19 revision 3 overestimates the strength of the connections. However, application of the "moment" model to these connections will give lower strength predictions, which have good correlation with the experimental values, as illustrated in Figure 18.

3.2 SERVICEABILITY CRITERIA

In the limit states design philosophy a serviceability limit state, such as deflection, may be the critical limit for the evaluation of the performance of a connection, rather than one of the ultimate limit states. An acceptable serviceability check would be to limit the amount of permanent deflection which occurs when the maximum specified load is applied. In an attempt to evaluate serviceability the limit on permanent live load deflection of a connection at the column face was taken arbitrarily as 0.10 inches.

The CISC computer program for eccentric loads on bolt groups was used to generate the load deflection curves for the following bolt group configurations:

(1) one line of five bolts at a pitch of 3.0 inches.

(2) two lines of two bolts each at a gage of 3.0 inches and a pitch of 6.0 inches.

For each of the two connection types, two connection strengths were calculated. First the connection strength as limited only by fastener shear was calculated; this would correspond to the real case of an uncoped beam with adequate end distance or a coped beam with adequate end and edge distance. Secondary, each of the connection analyses was modified, as described in Section 3.0, to correspond to the University of Texas tests which had end or edge distance values limiting the connection strength. In the case of the five bolt single line, tests 18-3 and 18-4 were simulated and in the two line four bolt connection, tests 18-16 and 18-17 were simulated. The analytical load-deflection curves for these bolt groups are shown in Figure 24. An estimate of service load for the bolt groups was made by dividing the ultimate strength by 2.0.

Recall that the CISC computer program for eccentric loads on bolt groups assume the strength of a fastener group is dependent upon the shear strength of the fasteners. The ultimate strength of the fastener group is assumed to be reached when the ultimate strength of the fastener farthest from the instantaneous center is reached.

In order to estimate the permanent live load deflection, a line parallel to the linear portion of the load deflection curve is projected from the point on the curve corresponding to one half of the ultimate load to the "x" axis of Figure 24. Figure 24 shows that the analytically obtained permanent deflection is less than the assumed limit of 0.10 inches.

While the vertical end distance for both test numbers 18-16 and 18-17 is 2.0 inches, for test number 18-16 the horizontal end distance is only 1.0 inches, as compared to 2.0 inches for test number 18-17. Because of the larger horizontal end distance, and thus larger permitted fastener load, the load-deflection curve for test number 18-17 is higher than that for test number 18-16. A direct comparison of the test data from tests 18-4 and 18-16, and the analyses shows reasonable correlation in Figure 25. In both cases the experimental data showed some "softer" behaviour above the proportional limit. For all cases, experimental and analytical, the 0.1 inch deflection limit would not have been exceeded after an application of half of the ultimate load.

While this assessment of the serviceability of two of the double angle connection configurations has been done arbitrarily with a tenth inch limit, it would seem, for the type of connections assessed, that the ultimate limit state would tend to control in design.

4.0 CONNECTION DESIGN

4.1 DESIGN PHILOSOPHY

It is the recommendation of this report, that in order to satisfy the adequacy of shear capacity of a bolted double angle beam web to column connections the following design checks must be completed;

(1) Shear capacity of the beam web.

(2) Shear capacity of the connection angles.

(3) Bearing capacity and/or bolt shear capacity (Developed in Section 3.1). The ultimate bearing strength, or shear strength, is computed by the revised CISC computer program for eccentric shear loads on bolt groups. A listing of the revised program is given in Appendix A.

(4) Web tearing capacity - an extension of the block shear concept for higher eccentric effects is called "moment model" herein. (Developed in Section 2.1). A computer program was formulated to facilitate the calculations involved in the "moment" model design procedure. A listing of this program is given in Appendix B.

4.2 DESIGN EXAMPLES

Example No. 1

A W18x45 subjected to a specified live load of 8.0 kips/foot and a dead load of 3.0 kips/foot, is required to span a distance of 12.0 feet. A cope of the top flange is required for erection.

Factored Live Load = 8.0 kips/ft x 1.5 = 12 kips/ft.

Factored Dead Load = 3.0 kips/ft x 1.25 = 3.75 kips/ft.

Total Factored Load = 15.75 kips/ft.

End Reaction = 15.75 kips/ft x 6 ft = 94.5 kips

V_r for W18x45 is 156 kips > 94.5 kips

Shear capacity of web is adequate

Horizontal and vertical end distances equal to 2.0 inches will be assumed.

Use A324 - 3/4 inch diameter H.S. bolts

Bolt shear resistance = 0.60 ϕ m (AB) (FUB)

$= 0.60 \times 0.67 \times 2 \times 0.4418 \text{ in}^2 \times 120 \text{ ksi}$

$= 42.6$ kips

The revised CISC computer program for eccentric loads on bolt groups is used to list coefficients based on connection geometry, web thickness, ultimate tensile strength of the web, and end distances.

$$\text{Coefficient required} = \frac{\text{End Reaction}}{\text{Bolt Shear Resistance}} = \frac{94.5}{42.6}$$

$$= 2.21$$

From computer output, pp. 32, the coefficient for a single line of five bolts with a 3.0 inch pitch is 2.78, at an eccentricity of 2.5 inches, which is greater than 2.21 required.

W18*45
WEB THICKNESS =0.3350 INCHES ULTIMATE TENSILE STRESS = 65.00 KSI
HORIZONTAL END DISTANCE = 2.00 INCHES
VERTICAL END DISTANCE = 2.00 INCHES

ECCENTRIC LOADS ON BOLT GROUPS
COEFFICIENTS C

PITCH B INCHES	NO. OF BOLTS	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	NO. OF BOLTS	PITCH B INCHES
					MOMENT ARM,E, INCHES										
	2	.95	.87	.80	.74	.69	.64							2	
	3	1.69	1.61	1.53	1.45	1.37	1.29							3	
	4	2.41	2.35	2.27	2.19	2.11	2.03							4	
	5	3.12	3.06	3.00	2.93	2.85	2.78							5	
6	6	3.82	3.77	3.71	3.65	3.58	3.51							6	6
	2	.64	.55	.49	.43	.39	.35							2	
	3	1.29	1.14	1.02	.91	.81	.73							3	
	4	2.03	1.87	1.72	1.58	1.45	1.34							4	
	5	2.73	2.61	2.45	2.29	2.14	2.00							5	
3	6	3.51	3.36	3.19	3.03	2.87	2.71							6	3

ECCENTRIC LOADS ON BOLT GROUPS
COEFFICIENTS C

2 VERTICAL LINES AT A SPACING, D OF 3 INCHES

PITCH B INCHES	NO. OF BOLTS	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	NO. OF BOLTS	PITCH B INCHES
					MOMENT ARM,E, INCHES										
	1	.46	.41	.37	.34	.31	.28							1	
	2	2.02	1.88	1.74	1.62	1.51	1.41							2	
	3	3.50	3.34	3.18	3.02	2.86	2.70							3	
	4	4.94	4.81	4.66	4.51	4.34	4.18							4	
	5	6.36	6.24	6.11	5.97	5.82	5.67							5	
6	6	7.75	7.65	7.54	7.42	7.28	7.14							6	6
	1	.46	.41	.37	.34	.31	.28							1	
	2	1.55	1.39	1.25	1.14	1.04	.96							2	
	3	2.78	2.51	2.27	2.07	1.90	1.75							3	
	4	4.22	3.92	3.63	3.37	3.13	2.92							4	
	5	5.69	5.37	5.06	4.76	4.47	4.20							5	
3	6	7.15	6.85	6.53	6.21	5.90	5.59							6	3

.˙. Use a single line of five bolts, pitch equal to 3.0 inches, and a clip angle eccentricity of 2.5 inches or less and a minimum edge distance of 2 inches.

The moment model computer program is used to check the capacity of this connection. It's output is shown on page 34. Ultimate shear capacity is equal to 188 kips. Applying a performance factor of 0.67 to this value gives: V_r = 0.67 x 1.88 kips = 126 kips, which is greater than the end reaction of 94.5 kips.

.˙. Connection is adequate.

Example No. 2

For the second example suppose that the connection designed in Example No. 1 was fabricated such that the 2.0 inch end distances were reduced to 1.0 inch.

From the output of the revised CISC program, pp. 35, for minimum end distances, the coefficient would be reduced to 1.95.

.˙. V_r = 1.95 x 42.6 kips = 83.1 kips

Ultimate shear capacity of the connection with minimum end distances is given by the "moment" model program as 152 kips. (output page 36).

.˙. V_r = 0.67 x 152 kips = 102 kips > 83.1 kips

.˙. Present strength of connection = 83.1 kips

The factored resistance of connection as calculated previously, Example No. 1, is 118 kips.

Thus a reduction in strength of $\frac{118 - 83.1}{118}$ = 29.6% was caused by improper fabrication.

W18*45

CONNECTION DATA

DIAMETER OF BOLTS = 0.7500 INCH
THICKNESS OF WEB = 0.3350 INCH
CLIP ANGLE ECCENTRICITY = 2.5000 INCHES
YIELD STRENGTH OF WEB = 44.000 KSI
ULTIMATE TENSILE STRENGTH OF WEB = 65.000 KSI
NUMBER OF ROWS OF BOLTS = 1
GAGE = 0.0000

HORIZONTAL END DISTANCE = 2.0000 INCHES
VERTICAL END DISTANCE = 2.0000 INCHES

NUMBER OF BOLTS IN ROW 1 = 5 PITCH = 3.0000

XC = 0.0000 INCH YC = 5.5733 INCH

SOLUTION

ULTIMATE SHEAR RESISTANCE IS LIMITED BY THE WEB
SHEAR CAPACITY OF THE MODEL

VSM2	TS1	VSB2
144.61	43.55	240.89

ULTIMATE SHEAR RESISTANCE = 188.16

W18*45
WEB THICKNESS =0.3350 INCHES ULTIMATE TENSILE STRESS = 65.00 KSI
HORIZONTAL END DISTANCE = 1.00 INCHES
VERTICAL END DISTANCE = 1.00 INCHES

ECCENTRIC LOADS ON BOLT GROUPS
COEFFICIENTS C

PITCH B INCHES	NO. OF BOLTS	MOMENT ARM,E, INCHES 2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	NO. OF BOLTS	PITCH B INCHES
	2	.34	.31	.29	.27	.25	.23							2	
	3	.91	.83	.75	.66	.59	.53							3	
	4	1.61	1.53	1.46	1.38	1.31	1.24							4	
	5	2.31	2.25	2.18	2.10	2.02	1.95							5	
6	6	3.01	2.96	2.89	2.82	2.75	2.68							6	6
	2	.23	.20	.17	.16	.14	.13							2	
	3	.53	.44	.38	.33	.30	.27							3	
	4	1.24	1.11	.99	.90	.81	.75							4	
	5	1.95	1.79	1.64	1.50	1.38	1.27							5	
3	6	2.68	2.52	2.36	2.20	2.05	1.92							6	3

ECCENTRIC LOADS ON BOLT GROUPS
COEFFICIENTS C

2 VERTICAL LINES AT A SPACING, D OF 3 INCHES

PITCH B INCHES	NO. OF BOLTS	MOMENT ARM,E, INCHES 2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	NO. OF BOLTS	PITCH B INCHES
	1	.17	.15	.13	.12	.11	.10							1	
	2	1.40	1.30	1.21	1.12	1.05	.98							2	
	3	2.74	2.60	2.46	2.32	2.18	2.04							3	
	4	4.15	4.02	3.88	3.73	3.58	3.43							4	
	5	5.56	5.44	5.31	5.17	5.03	4.87							5	
6	6	6.95	6.85	6.74	6.61	6.47	6.33							6	6
	1	.17	.15	.13	.12	.11	.10							1	
	2	1.07	.96	.87	.79	.72	.67							2	
	3	2.11	1.89	1.70	1.55	1.42	1.31							3	
	4	3.48	3.20	2.95	2.73	2.53	2.35							4	
	5	4.90	4.60	4.30	4.02	3.75	3.50							5	
3	6	6.34	6.04	5.73	5.42	5.13	4.94							6	3

PRCGRAM FOR EVALUATION OF ULT..ATE SHEAR WHEN
TOP FLANGE IS COPED

W18*45

CONNECTION DATA

DIAMETER OF BOLTS = 0.7500 INCH
THICKNESS OF WEB = 0.3350 INCH
CLIP ANGLE ECCENTRICITY = 2.5000 INCHES
YIELD STRENGTH OF WEB = 44.000 KSI
ULTIMATE TENSILE STRENGTH OF WEB = 65.000 KSI
NUMBER OF ROWS OF BOLTS = 1
GAGE = 0.0000

HORIZONTAL END DISTANCE = 1.0000 INCHES
VERTICAL END DISTANCE = 1.0000 INCHES

NUMBER OF BOLTS IN ROW 1 = 5 PITCH = 3.0000

XC = 0.0000 INCH YC = 5.5621 INCH

SOLUTION

ULTIMATE SHEAR RESISTANCE IS LIMITED BY THE WEB
SHEAR CAPACITY OF THE MODEL

VSM2	TS1	VSB2
130.24	21.77	212.14

ULTIMATE SHEAR RESISTANCE = 152.02

Example No. 3

In this example, suppose that a steel fabricator wishes to use a standard connection for W18x45 beams with a coped top flange. The connection consists of two vertical lines of bolts in the web spaced at 3.0 inches, with two A325 - 3/4 inch diameter H.S. bolts per line and a pitch of 6.0 inches. The resistance of this connection, with end distances of 2.0 inches is evaluated as follows:

Moment Arm for connection

$$= \frac{\text{Bolt Gauge}}{2.} + \text{Eccentricity to first line of bolts}$$

$$= \frac{3.0}{2.} \text{ inches} + 2.5 \text{ inches} = 4.0 \text{ inches}$$

From the output of the "bearing" model program, pp. 32, the coefficient for this connection is 1.62.

$\therefore$ V_r for the beam web = 1.62 x 42.6 kips

= 69.0 kips

Checking the connection using the "moment" model program gives the output shown on pp. 38 i.e. ultimate shear capacity = 124 kips

$\therefore$ V_r = 0.67 x 124 kips = 83.1 kips > 69.0 kips

Thus the shear strength of the beam web for this fastener configuration would be 69.0 kips.

If a check of the clip angles showed that their resistance was equal to, or greater than, 69.0 kips, then the shear resistance of the connection would be assigned a value of 69 kips.

PROGRAM FOR EVALUATION OF ULTIMATE SHEAR WHEN
TOP FLANGE IS COPED

W18*45

CONNECTION DATA

DIAMETER OF BOLTS = 0.7500 INCH
THICKNESS OF WEB = 0.3350 INCH
CLIP ANGLE ECCENTRICITY = 2.5000 INCHES
YIELD STRENGTH OF WEB = 44.000 KSI
ULTIMATE TENSILE STRENGTH OF WEB = 65.000 KSI
NUMBER OF ROWS OF BOLTS = 2
GAGE = 3.0000

HORIZONTAL END DISTANCE = 2.0000 INCHES
VERTICAL END DISTANCE = 2.0000 INCHES

NUMBER OF BOLTS IN ROW 1 = 2 PITCH = 6.0000

NUMBER OF BOLTS IN ROW 2 = 2 PITCH = 6.0000

XC = 0.0000 INCH YC = 1.9676 INCH

SOLUTION

MB11	MB12	MB1	MB2	MB3	MB	TS1
104.27	272.19	376.46	305.81	0.00	682.26	108.87

ULTIMATE SHEAR RESISTANCE = 124.05

Example No. 4

A W18x41, subjected to uniform loading, is required to span a distance of twelve feet and a cope of the top flange is required for erection. One inch in diameter, A325, H.S. bolts will be used in the end connections. This example is presented to show a case where "moment" model is critical.

Mr for W18x41 = 260 kips/foot

$$Mr = \frac{w\ell^2}{8} = 2\,\frac{(12.0\ ft)^2}{8} = 260\ kips/ft$$

$\therefore\ w = 14.4$ kips/ft

and End Reaction = 14.4 kips/ft x 6 ft = 86.4 kips

Use minimum end distances, i.e. 1.25 inches.

Shear Resistance of 1 inch diameter bolt is equal to 0.67 x 0.60 x 2 x 0.785 in^2 x 120 ksi = 75.8 kips

$$\text{Coefficient required} = \frac{\text{End Reaction}}{\text{Bolt Shear Resistance}}$$

$$= \frac{86.4}{75.8} = 1.14$$

From output of "bearing" model program, pp. 41, for a single line of four bolts with a 3.0 inch pitch and a clip angle eccentricity of 2.5 inches, the coefficient is 1.99.

$1.99 > 1.14\ \therefore$ adequate

Assume a single line of four bolts with a 3.0 inch pitch.

Ultimate shear capacity predicted by the "moment" model is 112 kips, (pp. 42).

$V_r = 0.67 \times 112$ kips $= 75.0$ kips < 86.4 kips

$\therefore$ connection is not adequate

Try a single line of five bolts with a 3.0 inch pitch. Ultimate shear capacity by the "moment" model is 138 kips (pp. 43).

$V_r = 0.67 \times 138$ kips $= 92.5$ kips > 86.4 kips

$\therefore$ The connection is adequate and the "moment" model type of failure was the critical design check.

W18*41

WEB THICKNESS =0.3190 INCHES   ULTIMATE TENSILE STRESS = 65.00 KSI
HORIZONTAL END DISTANCE = 1.25 INCHES
VERTICAL END DISTANCE = 1.25 INCHES

ECCENTRIC LOADS ON BOLT GROUPS
COEFFICIENTS C

PITCH B INCHES	NO. OF BOLTS	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	NO. OF BOLTS	PITCH B INCHES
	2	.40	.37	.34	.31	.29	.27							2	
	3	1.10	.98	.86	.76	.68	.62							3	
	4	1.99	1.90	1.80	1.70	1.61	1.52							4	
	5	2.88	2.80	2.71	2.62	2.52	2.42							5	
6	6	3.77	3.70	3.62	3.53	3.44	3.34							6	6
	2	.27	.23	.20	.18	.16	.15							2	
	3	.62	.52	.44	.39	.34	.31							3	
	4	1.52	1.35	1.21	1.09	.99	.91							4	
	5	2.42	2.22	2.02	1.84	1.68	1.54							5	
3	6	3.34	3.14	2.94	2.74	2.56	2.37							6	3

ECCENTRIC LOADS ON BOLT GROUPS
COEFFICIENTS C

2 VERTICAL LINES AT A SPACING, D OF 3 INCHES

PITCH B INCHES	NO. OF BOLTS	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	NO. OF BOLTS	PITCH B INCHES
	1	.19	.17	.16	.14	.13	.12							1	
	2	1.74	1.62	1.50	1.40	1.30	1.21							2	
	3	3.44	3.25	3.08	2.88	2.69	2.52							3	
	4	5.23	5.06	4.88	4.69	4.50	4.31							4	
	5	7.01	6.86	6.70	6.52	6.33	6.14							5	
6	6	8.78	8.65	8.50	8.34	8.17	7.99							6	6
	1	.19	.17	.16	.14	.13	.12							1	
	2	1.34	1.20	1.08	.98	.90	.83							2	
	3	2.63	2.36	2.13	1.94	1.77	1.64							3	
	4	4.36	3.99	3.67	3.38	3.13	2.90							4	
	5	6.17	5.78	5.39	5.02	4.69	4.38							5	
3	6	8.00	7.62	7.23	6.83	6.43	6.06							6	3

W18*41

CONNECTION DATA

DIAMETER OF BOLTS = 1.0000 INCH
THICKNESS OF WEB = 0.3190 INCH
CLIP ANGLE ECCENTRICITY = 2.5000 INCHES
YIELD STRENGTH OF WEB = 44.000 KSI
ULTIMATE TENSILE STRENGTH OF WEB = 65.000 KSI
NUMBER OF ROWS OF BOLTS = 1
GAGE = 0.0000

HORIZONTAL END DISTANCE = 1.2500 INCHES
VERTICAL END DISTANCE = 1.2500 INCHES

NUMBER OF BOLTS IN ROW 1 = 4 PITCH = 3.0000

XC = 0.0000 INCH YC = 4.2793 INCH

SOLUTION

ULTIMATE SHEAR RESISTANCE IS LIMITED BY THE WEB
SHEAR CAPACITY OF THE MODEL

VSM2	TS1	VSB2
86.39	25.92	205.43

ULTIMATE SHEAR RESISTANCE = 112.31

PROGRAM FOR EVALUATION OF ULTIMATE SHEAR WHEN
TOP FLANGE IS COPED

W18*41

CONNECTION DATA

DIAMETER OF BOLTS = 1.0000 INCH
THICKNESS OF WEB = 0.3190 INCH
CLIP ANGLE ECCENTRICITY = 2.5000 INCHES
YIELD STRENGTH OF WEB = 44.000 KSI
ULTIMATE TENSILE STRENGTH OF WEB = 65.000 KSI
NUMBER OF ROWS OF BOLTS = 1
GAGE = 0.0000

HORIZONTAL END DISTANCE = 1.2500 INCHES
VERTICAL END DISTANCE = 1.2500 INCHES

NUMBER OF BOLTS IN ROW 1 = 5 PITCH = 3.0000

XC = 0.0000 INCH YC = 5.6194 INCH

SOLUTION

ULTIMATE SHEAR RESISTANCE IS LIMITED BY THE WEB
SHEAR CAPACITY OF THE MODEL

VSM2	TS1	VSB2
112.05	25.92	267.64

ULTIMATE SHEAR RESISTANCE = 137.97

5. CONCLUSIONS AND RECOMMENDATIONS

The findings of this report are summarized as follows:

(1) While the "block shear" failure model will give accurate results for connections with only one line of bolts, for connections with two lines of bolts it overestimates the ultimate connection strength because it does not consider the effect of eccentricity.

(2) The "moment" model design procedure was developed to consider eccentricity and is applicable to connections with either one or two lines of bolts. For connections with only one line of bolts, the "moment" model degenerates into a "block shear" mode of failure.

(3) Results obtained from the use of the "moment" model to predict the ultimate shear capacity of the connection configuration tested at the University of Texas showed good correlation with the measured values.

(4) The "tension resistance" model, while exhibiting correlation with certain types of bolt group configurations, does not give satisfactory correlation with all configurations.

(5) Current CSA S16.1-1974, "Structures for Buildings - Limit States Design", require the following design checks to be completed to satisfy the adequacy of shear capacity of a bolted angle beam web to column connection.

1. Shear capacity of the beam web.
2. Bearing resistance of the web.
3. Bolt shear.
4. "Block Shear" model.

(6) When considering the effect of eccentric loading, S16.1-1974 requires that the capacity of a given connection be determined by multiplying the appropriate coefficient "C" by the smaller of (1) shear resistance of a bolt, and (2) bearing capacity of the connected material. Clause 13.10 (c) of S16.1-1974, which gives an expression for the bearing capacity of a material (i.e. Br = ϕtneFu), does not state what end, or edge, distance should be used in this expression. An interpretation of "e" to be equal to the smallest of either horizontal, or vertical, end distances led to very conservative estimates for the ultimate shear strengths of the connections tested at the University of Texas.

(7) A design aid for use in predicting the ultimate shear strength of a connection with either one or two lines of bolts, when the top flange is coped, was developed by revising the CISC computer program for eccentric loads on bolt groups to include fastener tearout and ultimate bearing strength as additional limits on fastener shear load, (i.e. the "Bearing" model).

(8) For the bolt group configurations tested at the University of Texas, an arbitrary serviceability limit state permanent on deflection was found not to be critical for the design of such connections.

It is the recommendation of this study that for double angle beam web to column connections, the design procedure for the connection should consist of the following three design limits when the top flange is coped.

1. Gross shear capacity of the beam web.
2. "Bearing" model (includes fastener shear)
3. "Moment" model.

LIST OF TABLES

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TABLE 1 - COMPARISON OF MEASURED SHEAR TO THEORETICAL VALUE

Test Number	Experimental Result (kips)	Moment Model (kips)	% of Test
18-1	205.	231.	113.
18-2	205.	204.	99.
18-3	212.	213.	100.
18-4	201.	200.	99.5
18-5	173.	185	107.
18-6	161.	186.	116.
18-7	201	214	106.
18-8	145.	173.	119.
18-9	152.	139.	91.4
18-10	111.	117.	105.
18-11	101.	94.3	93.4
18-12	152.	146.	96.
18-13	140.	160.	114.
18-16	111.	110.	99.2
18-17	131.	136.	103.
18-18	101.	94.2	93.3
18-19	134.	136.	101.
18-20	167.	162.	96.7
18-21	142.	129.	90.9
18-22	185.	162.	87.1
18-23	157.	129.	82.3
18-24	178.	161.	90.4
18-25	142.	161.	113.

TABLE 2 - ULTIMATE SHEAR STRENGTH CALCULATED BY "BLOCK SHEAR" MODEL

Test Number	Number of lines of bolts	F_u (ksi)	$An2^b$ (in^2)	$An1^a$ (in^2)	Shear Resistance $0.66 \times F_u \times An2$ (kips)	Tensile Resistance $Fu \times An1$ (kips)	Ultimate Shear (kips)
18-3	1	60.3	4.05	0.671	161.2	40.4	202.
18-4	1	60.3	4.36	0.247	173.6	14.9	189.
18-5	1	60.3	3.98	0.247	158.3	14.9	173.
18-8	1	60.3	3.89	0.165	154.9	9.9	165.
18-9	1	60.3	4.70	0.660	187.1	39.8	227.
18-10	2	60.3	2.91	1.15	115.7	69.5	185.
18-11	2	60.3	2.81	1.05	111.8	63.2	175.
18-12	2	60.3	2.91	1.62	115.7	97.6	213.
18-16	2	58.6	2.81	1.13	108.7	66.2	175.
18-17	2	58.6	2.81	1.55	108.7	90.8	200.
18-18	2	58.6	2.02	1.13	78.1	66.2	144.
18-19	2	58.6	2.44	1.55	94.4	90.8	185.
18-20	1	58.6	4.96	0.446	191.8	26.1	218.
18-21	1	58.6	4.54	0.236	175.6	13.8	189.
18-22	1	58.6	4.96	0.656	191.8	38.4	230.
18-23	1	58.6	4.54	0.656	175.6	38.4	214.
18-24	1	58.6	4.90	0.630	189.5	36.9	226.
18-25	1	58.6	4.90	0.210	189.5	12.3	202.

a - An1 Net Area along plane AA

b - An2 Net Area along plane BB

TABLE 3 ULTIMATE SHEAR STRENGTHS CALCULATED BY "TENSION RESISTANCE" MODEL

Test Number	Number of lines of bolts	Ultimate Shear = Fu x Ag (kips)
18-3	1	52.2
18-4	1	26.5
18-5	1	26.5
18-8	1	23.2
18-9	1	51.4
18-10	2	104.
18-11	2	130.
18-12	2	132.
18-16	2	98.5
18-17	2	123.
18-18	2	98.5
18-19	2	123.
18-20	1	36.9
18-21	1	24.6
18-22	1	49.2
18-23	1	49.2
18-24	1	47.7
18-25	1	23.1

Ag = gross area of web along plane AA

TABLE 4 COMPARISON OF CALCULATED AND MEASURED ULTIMATE SHEARS

Test Number	Number of lines of bolts	Measured ultimate shear	BLOCK SHEAR MODEL		TENSION RESISTANCE		MOMENT MODEL	
			Ultimate shear (kips)	% of Test	Ultimate shear (kips)	% of Test	Ultimate shear (kips)	% of Test
18-3	1	212.	202.	95.3	52.2	24.6	213.	100.
18-4	1	201.	189.	94.9	25.5	13.2	200.	99.5
18-5	1	173.	173.	100.	26.5	15.3	185.	107.
18-8	1	145.	165.	114.	23.2	16.0	173.	119.
18-9	1	152.	139.[b]	91.4	51.4	33.8	139.[b]	91.4
18-10	2	111.	185.	167.	104.	93.7	117.	105.
18-11	2	101.	175.	173.	130.	129.	94.3	93.4
18-12	2	152.	213.	140.	132.	86.8	146.	96.1
18-16	2	111.	175.	158.	98.5	88.7	110.	99.2
18-17	2	131.	200.	153.	123.	93.9	136.	103.
18-18	2	101.	144.	143.	98.5	97.5	94.2	93.3
18-19	2	134.	185.	138.	123.	91.8	136.	101.
18-20	1	167.	162.[b]	96.7	36.9	22.1	162.[b]	96.7
18-21	1	142.	129.[b]	90.9	24.6	17.3	129.[b]	90.9
18-22	1	185.	162.[b]	87.1	49.2	25.5	162.[b]	87.1
18-23	1	157.	129.[b]	82.3	49.2	31.3	129.[b]	82.3
18-24	1	178.	161.[b]	90.4	47.7	26.8	161.[b]	90.4
18-25	1	142.	161.[b]	113.	23.1	16.3	161.[b]	113.

b - bearing strength of the web is critical

TABLE 5 SHEAR RESISTANCE OF BEAM WEB CSA S16.1

Test Number	Web Thickness (inches)	Depth of Section (inches)	Web Area "Aw" (in^2)	Fy (ksi)	V_r = 0.66xFyxAw (kips)
18-3	0.447	18.38	8.21	38.5	209.
18-4	0.439	18.31	8.04	38.5	204.
18-5	0.439	18.38	8.07	38.5	205.
18-8	0.440	18.47	8.13	38.5	207.
18-9	0.440	18.38	8.09	38.5	205.
18-10	0.439	18.31	8.04	38.5	294.
18-11	0.430	18.31	7.87	38.5	200.
18-12	0.439	18.38	8.07	38.5	205.
18-16	0.420	18.38	7.72	36.6	186.
18-17	0.420	18.38	7.72	36.6	186.
18-18	0.420	18.38	7.72	36.6	186.
18-19	0.420	18.38	7.72	36.6	186.
18-20	0.420	18.38	7.72	36.6	186.
18-21	0.420	18.38	7.72	36.6	186.
18-22	0.420	18.38	7.72	36.6	186.
18-23	0.420	18.38	7.72	36.6	186.
18-24	0.420	18.38	7.72	36.6	186.
18-25	0.420	18.38	7.72	36.6	186.

TABLE 6 BEARING RESISTANCE - CSA S16.1

Test Number	Number of lines of bolts	Number of bolts 'η'	Fu (ksi)	Web Thickness (in.)	EV (in.)	EH (in.)	$B_r = \frac{\phi \cdot t \cdot \eta \cdot EV \cdot Fu}{0.9}$ (kips)	$B_r = \frac{\phi \cdot t \cdot \eta \cdot EH \cdot Fu}{0.9}$ (kips)
18-3	1	5	60.3	0.447	1.0	1.938	100.	194.
18-4	1	5	60.3	0.439	1.875	1.0	185.	98.5
18-5	1	5	60.3	0.439	1.0	1.0	98.5	98.5
18-8	1	5	60.3	0.440	1.063	0.875	105.	86.4
18-9	1	3	60.3	0.440	1.0	1.938	59.3	115.
18-10	2	5	60.3	0.439	2.0	0.938	197.	92.4
18-11	2	5	60.3	0.430	2.0	1.0	193.	96.5
18-12	2	5	60.3	0.439	2.0	2.0	197.	197.
18-16	2	4	58.6	0.420	2.0	1.0	147.	73.3
18-17	2	4	58.6	0.420	2.0	2.0	147.	147.
18-18	2	5	58.6	0.420	1.0	1.0	91.6	91.6
18-19	2	5	58.6	0.420	2.0	2.0	183.	183.
18-20	1	3	58.6	0.420	2.0	1.5	110.	82.5
18-21	1	3	58.6	0.420	1.0	1.0	55.0	55.0
18-22	1	3	58.6	0.420	2.0	2.0	110.	110.
18-23	1	3	58.6	0.420	1.0	2.0	55.0	110.
18-24	1	3	58.6	0.420	2.0	2.0	110.	110.
18-25	1	3	58.6	0.420	2.0	1.0	110.	55.0

TABLE 7 'BLOCK SHEAR' RESISTANCE - CSA S16.1

Test Number	m	η	Fu (ksi)	Av (in^2)	A_t (in^2)	V = 0.53xFuxAv +FuxAt (kips)
18-3	1	5	60.3	4.18	0.684	175.
18-4	1	5	60.3	4.49	0.261	159.
18-5	1	5	60.3	4.10	0.261	147.
18-8	1	5	60.3	4.02	0.193	140.
18-9	1	3	60.3	4.83	0.674	195.
18-10	2	2	60.3	2.95	1.19	166.
18-11	2	2	60.3	2.85	0.90	145.
18-12	2	2	60.3	2.95	1.66	194.
18-16	2	2	58.6	2.85	1.17	157.
18-17	2	2	58.6	2.85	1.59	182.
18-18	2	3	58.6	2.09	1.17	133.
18-19	2	3	58.6	2.51	1.59	171.
18-20	1	3	58.6	5.03	0.459	183.
18-21	1	3	58.6	4.61	0.249	158.
18-22	1	3	58.6	5.03	0.669	195.
18-23	1	3	58.6	4.61	0.669	182.
18-24	1	3	58.6	4.96	0.643	192.
18-25	1	3	58.6	4.96	0.223	167.

m = number of lines of bolts

η = number of bolts in left hand side line

$$Av = w\left[L - (n - \tfrac{1}{2})\, d \right]$$

For one line of bolts:

$$At = w\,(e_o - \tfrac{1}{2}\, d)\ Fu$$

For two lines of bolts:

$$At = w\,(e_o + G - (m - \tfrac{1}{2})\, d)$$

TABLE 8 ECCENTRIC LOADS ON BOLT GROUP - CSA S16.1

Test Number	Number of Bolts	Coefficient 'c'	Eccentric Load on Bolts	
			Shear* (kips)	Bearing** (kips)
18-3	5	4.21	268.	84.2
18-4	5	4.21	268.	82.9
18-5	5	4.21	268.	82.9
18-8	5	4.21	268.	72.7
18-9	3	2.65	169.	52.4
18-10	5	3.19	203.	59.0
18-11	5	2.97	189.	57.3
18-12	5	3.19	203.	126.
18-16	4	2.45	156.	44.9
18-17	4	2.45	156.	89.8
18-18	5	2.39	152.	43.8
18-19	5	2.39	152.	87.6
18-20	3	2.65	169.	72.9
18-21	3	2.65	169.	48.6
18-22	3	2.65	169.	97.2
18-23	3	2.65	169.	48.6
18-24	3	2.65	169.	97.2
18-25	3	2.65	169.	48.6

* - Shear based on ultimate shear capacity of 3/4 inch diameter A325 H.S. bolt. (i.e. 63.6 kips)

** - Bearing based on bearing resistance of the connected material (see Table 6)

TABLE 9 SUMMARY OF DESIGN CHECKS - CSA S16.1

Test Number	Web Shear (kips)	Bearing Vertical (kips)	Bearing Horizontal (kips)	Eccentric Load (kips)	Block Shear (kips)	Critical Design check	% of Test
18-3	209.	100.	194.	84.2	175.	Eccentric Load	39.7
18-4	204.	185.	98.5	82.9	159.	Eccentric Load	41.2
18-5	205.	98.5	98.5	82.9	147.	Eccentric Load	47.9
18-8	207.	105.	86.4	72.7	140.	Eccentric Load	50.1
18-9	205.	59.3	115.	52.4	195.	Eccentric Load	34.5
18-10	204.	197.	92.4	59.0	166.	Eccentric Load	53.2
18-11	200.	193.	96.5	57.3	145.	Eccentric Load	56.7
18-12	205.	197.	197.	126.	194.	Eccentric Load	82.9
18-16	186.	147.	73.3	44.9	157.	Eccentric Load	40.5
18-17	186.	147.	147.	89.8	182.	Eccentric Load	68.5
18-18	186.	91.6	91.6	43.8	133.	Eccentric Load	43.3
18-19	186.	183.	183.	87.6	171.	Eccentric Load	66.9
18-20	186.	110.	82.5	72.9	183.	Eccentric Load	43.7
18-21	186.	55.	55.	48.6	158.	Eccentric Load	34.0
18-22	186.	110.	110.	97.2	195.	Eccentric Load	52.5
18-23	186.	55.	110.	48.6	182.	Eccentric Load	31.0
18-24	186.	110.	110.	97.2	192.	Eccentric Load	54.6
18-25	186.	110.	55.	48.6	167.	Eccentric Load	34.2

TABLE 10 COMPARISON OF REVISIONS FOR CISC COMPUTER PROGRAM

Test Number	Experimental Result (kips)	CISC COMPUTER PROGRAM RESULT					
		Revision 1 (kips)	% of Test	Revision 2 (kips)	% of Test	Revision 3 (kips)	% of Test
18-3	212	252.	119.	202.	95.4	185.	87.3
18-4	201	220.	109.	199.	98.8	179.	89.2
18-5	173	220.	127.	167.	96.4	150.	86.8
18-8	145	215.	148.	159.	110.	144.	99.6
18-9	152	151.	99.3	197.	70.3	99.9	65.7
18-10	111.	221.	199.	210.	189.	183.	164.
18-11	101.	221.	219.	207.	205.	177.	176.
18-12	152.	251.	165.	240.	158.	216.	142.
18-16	111.	120.	108.	112.	101.	94.1	85.1
18-17	131.	141.	108.	134.	103.	116.	88.5
18-18	101.	190.	188.	154.	152.	128.	127.
18-19	134.	216.	161.	203.	151.	176.	131.
18-20	167.	137.	82.	118.	70.7	108.	64.6
18-21	142.	123.	86.	69.3	48.8	65.6	46.2
18-22	185.	150.	81.1	132.	71.4	122.	65.5
18-23	157.	150.	95.5	104.	66.2	94.1	60.0
18-24	178.	148.	38.1	130.	73.	120.	67.7
18-25	142.	121.	85.2	102.	71.8	92.9	65.6

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23 Comparison of Revision 3 to Current CSA Specifications, Double Line of Bolts Connections

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25 Comparison of Analytical and Experimental Load-Deflection Curves

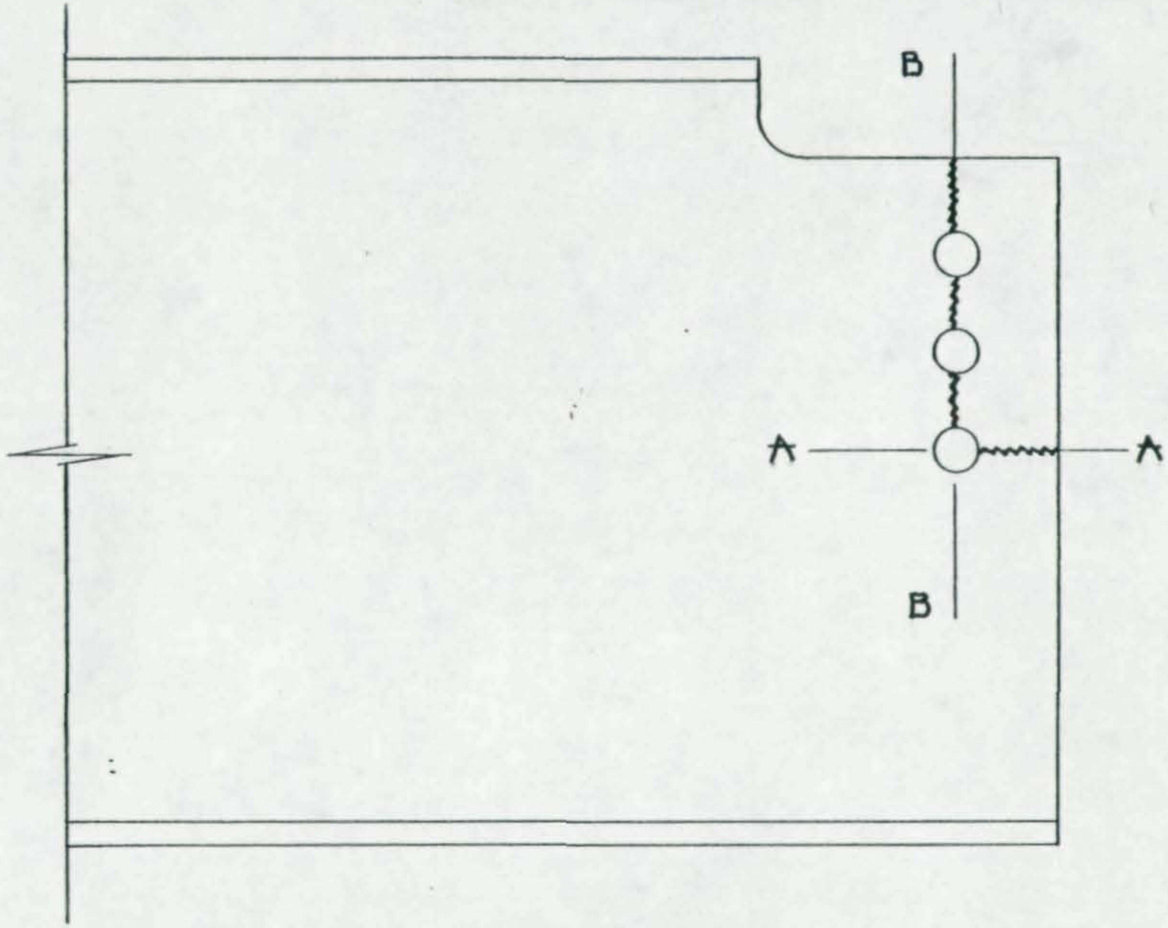


FIGURE 1 BLOCK SHEAR MODEL OF FAILURE

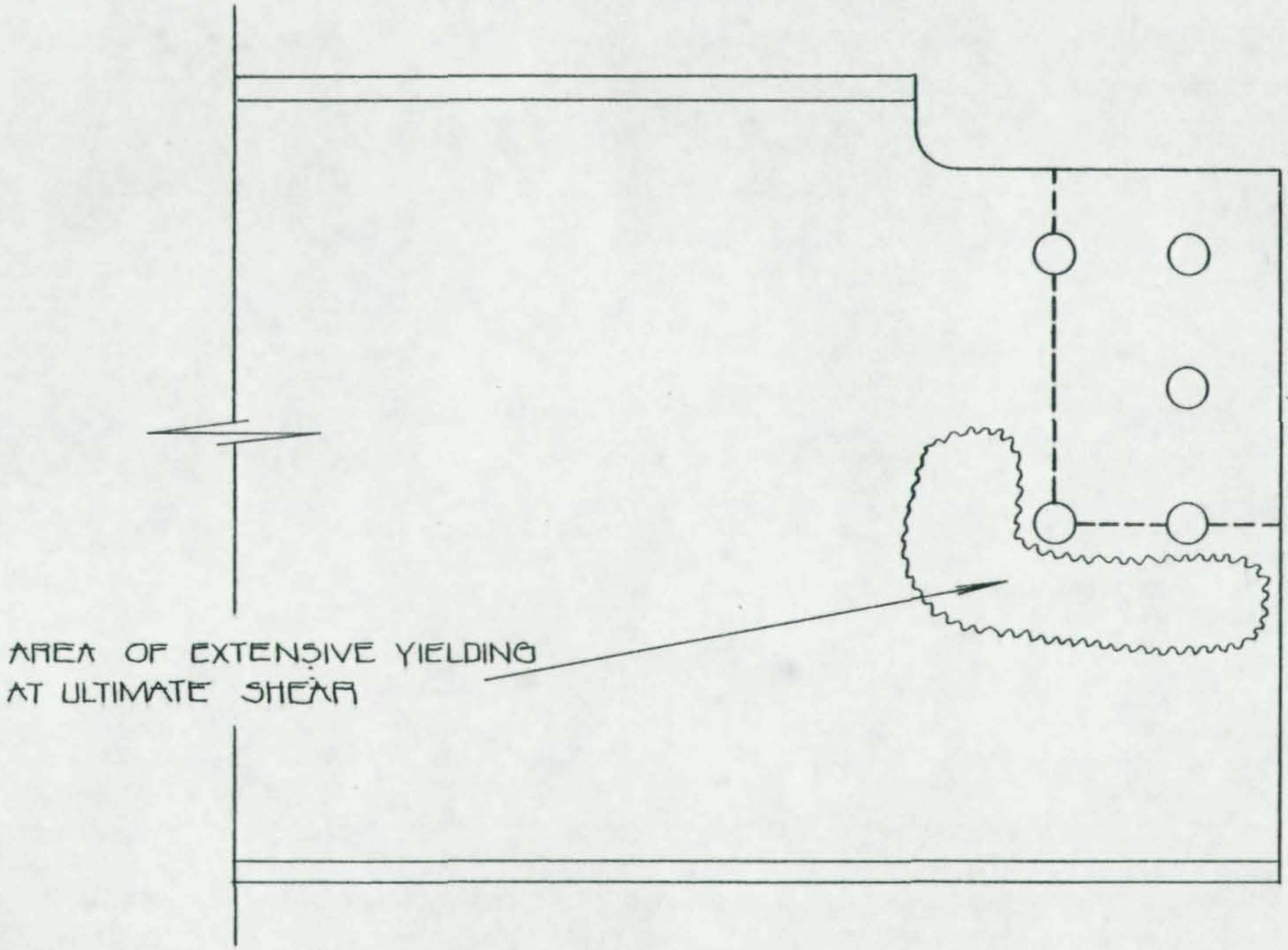


FIGURE 2 BEAM WITH COPED TOP FLANGE AT CONNECTION

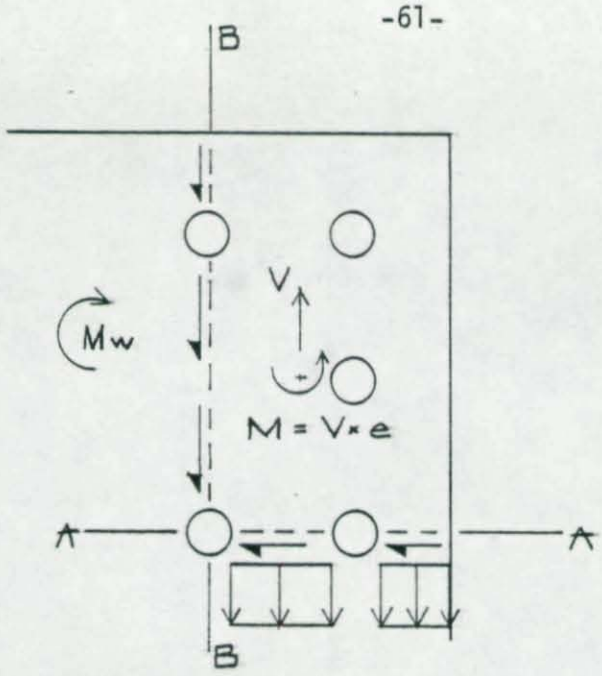


FIGURE 3 FORCES ON WEB

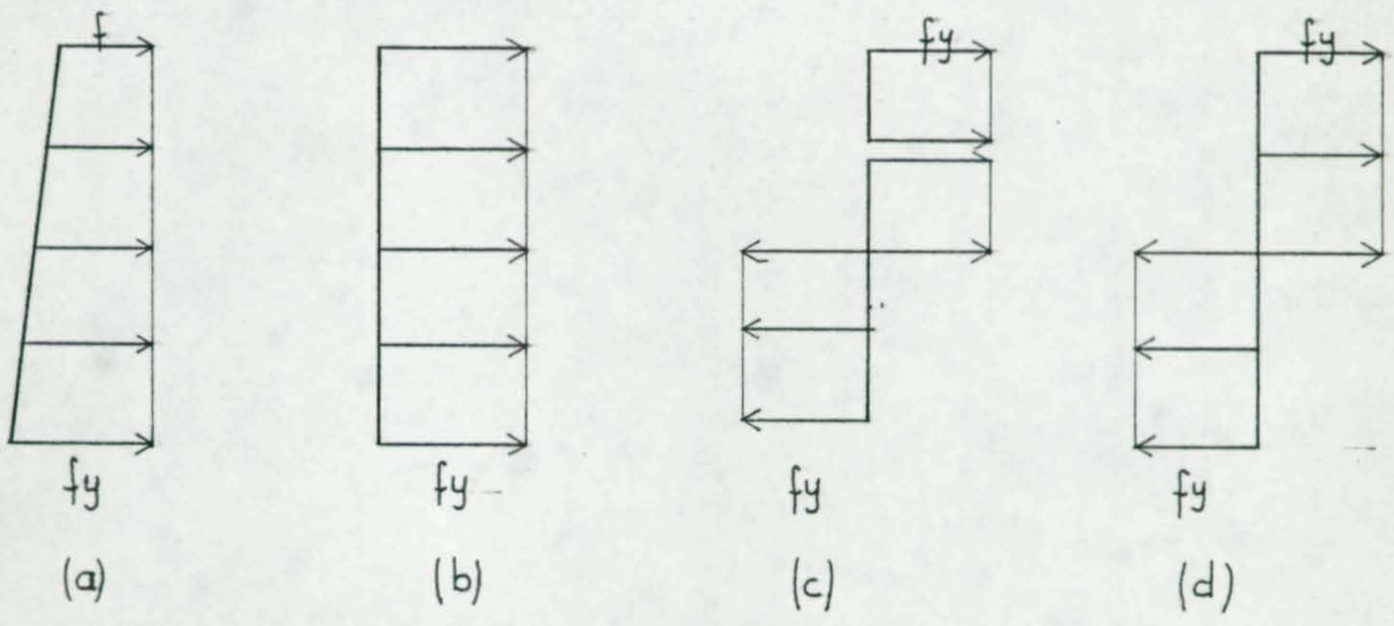


FIGURE 4 POSSIBLE STRESS DISTRIBUTIONS ALONG PLANE B-B,

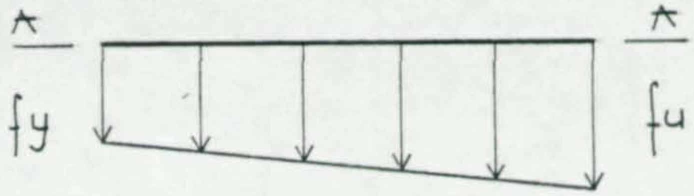


FIGURE 5 ALTERNATIVE DISTRIBUTION OF TENSILE STRESS ALONG PLANE A-A

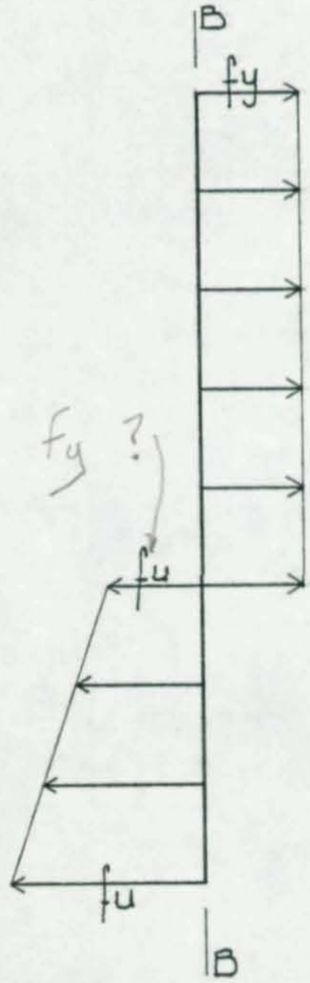


FIGURE 6 DISTRIBUTION OF STRESS ALONG PLANE B-B

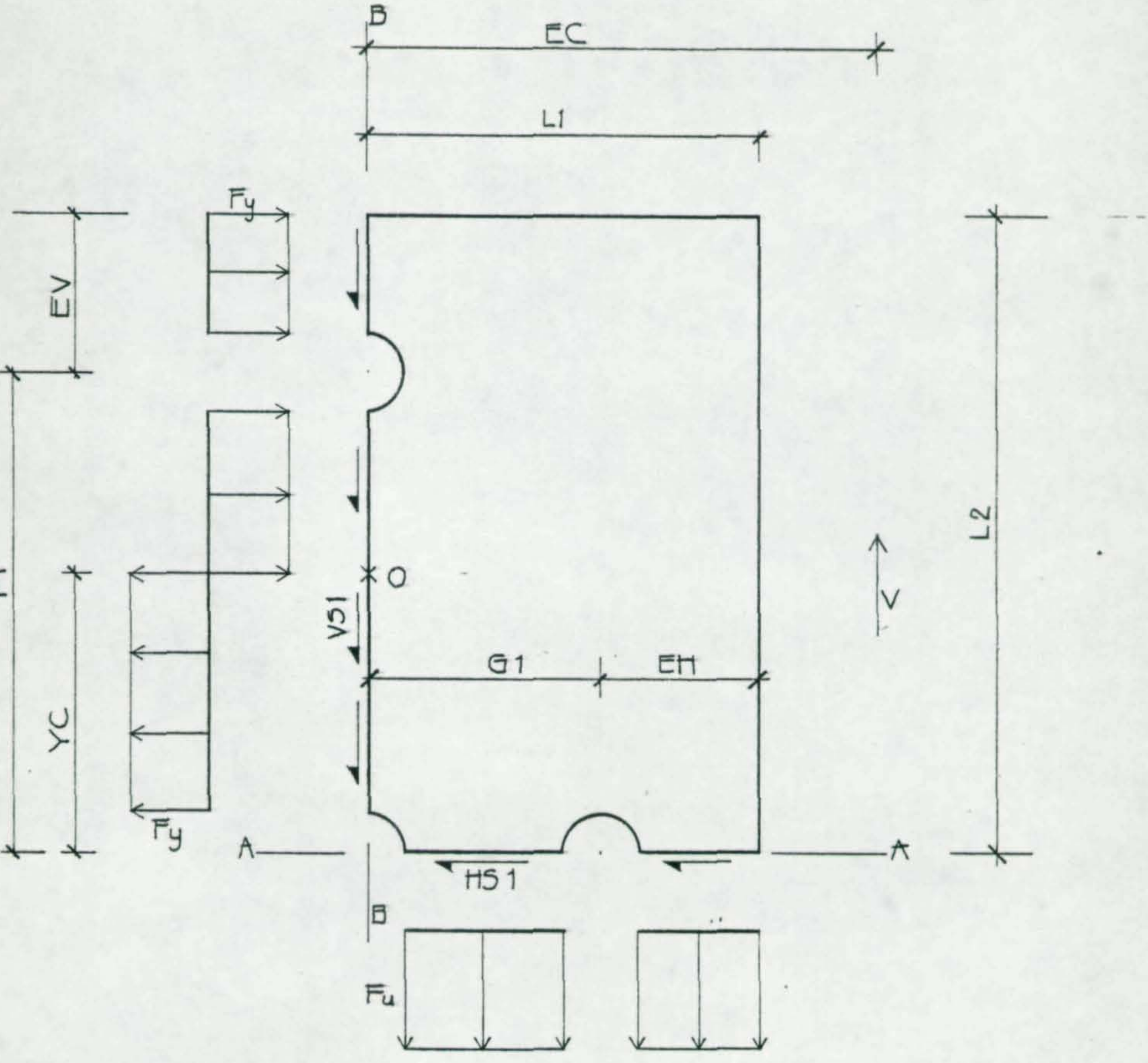


FIGURE 7 - DISTRIBUTION OF STRESSES - TRIAL 1

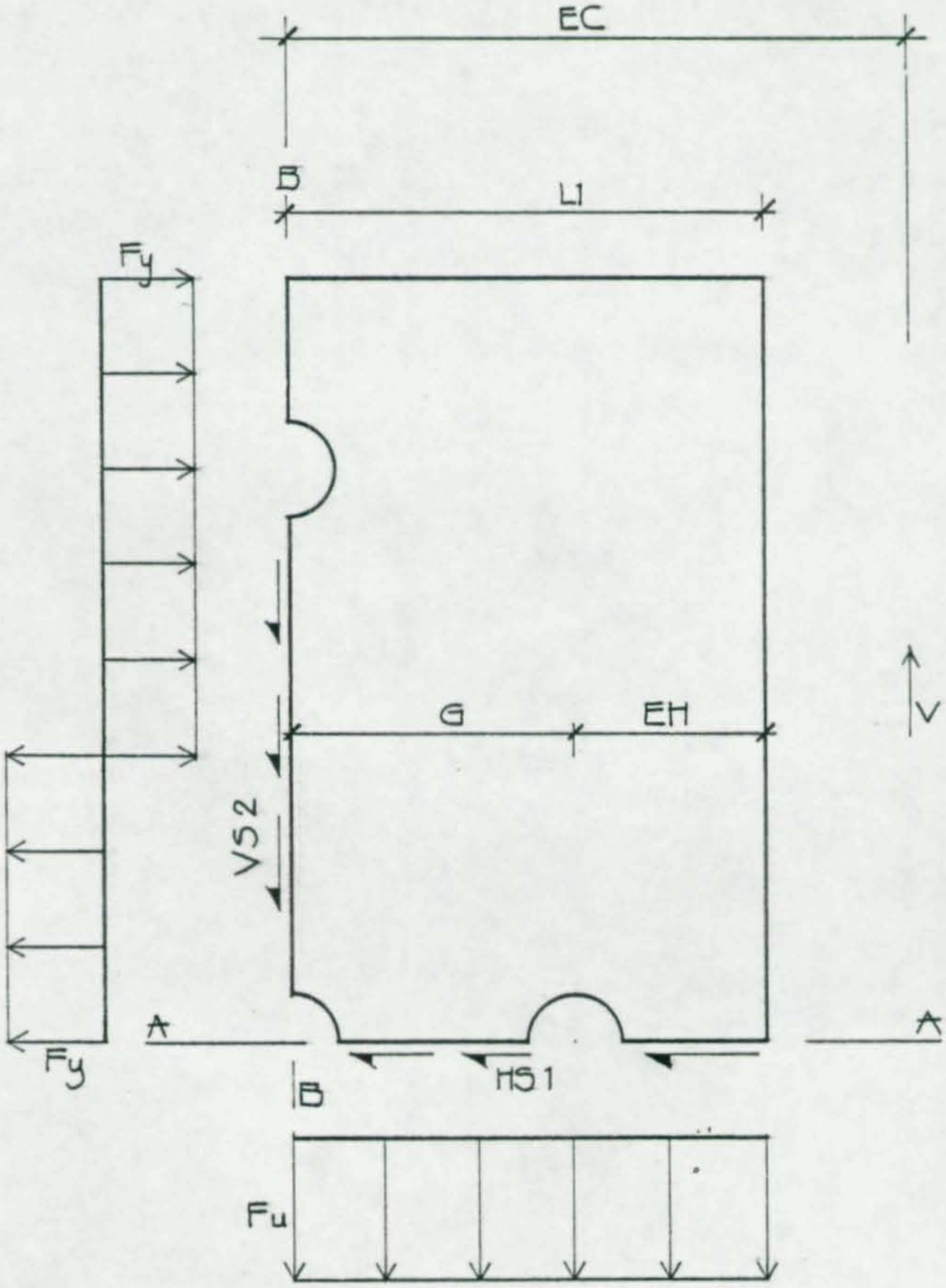


FIGURE 8 DISTRIBUTION OF STRESSES - TRIAL 2

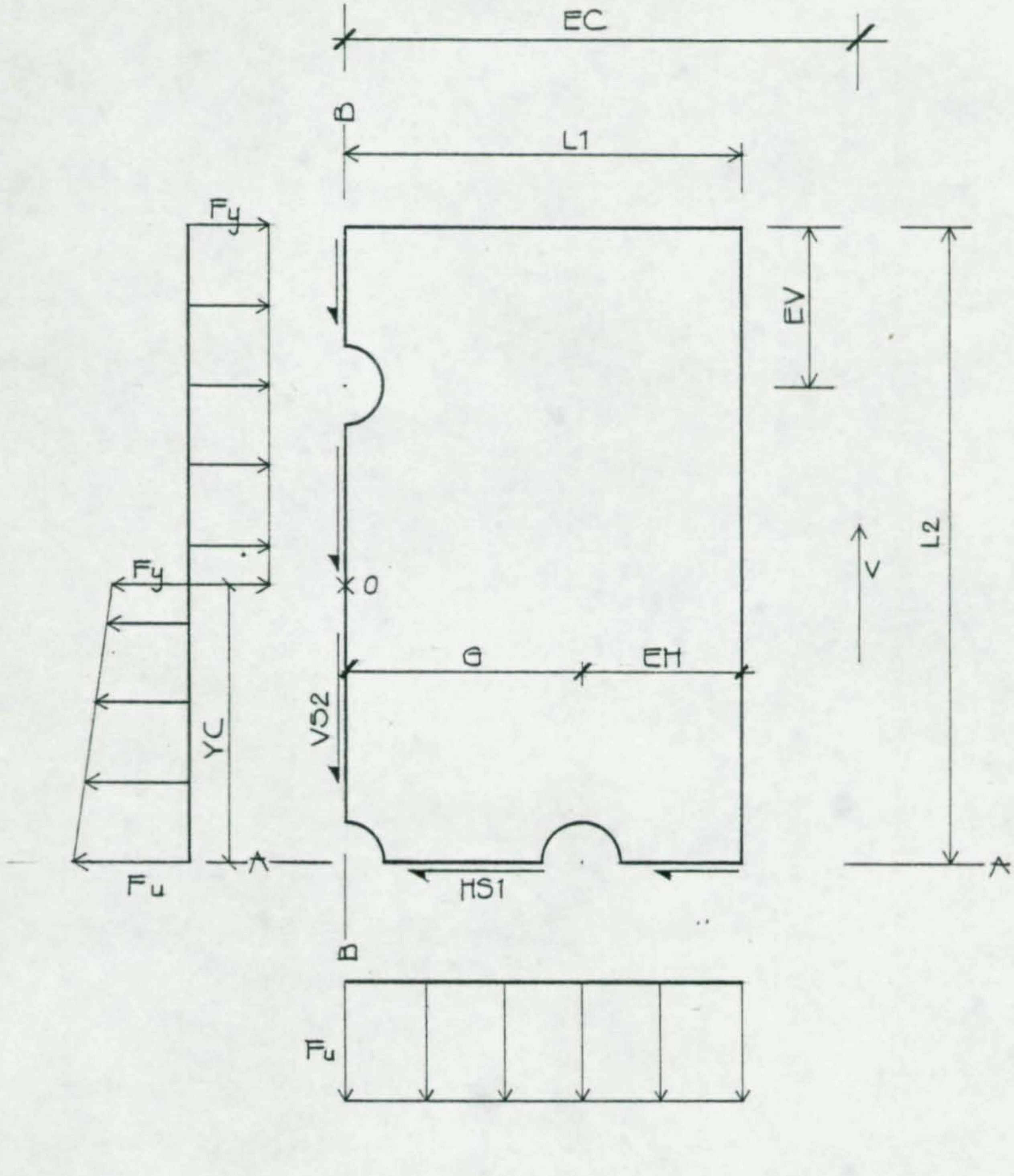


FIGURE 9 MOMENT MODEL, COPED FLANGE

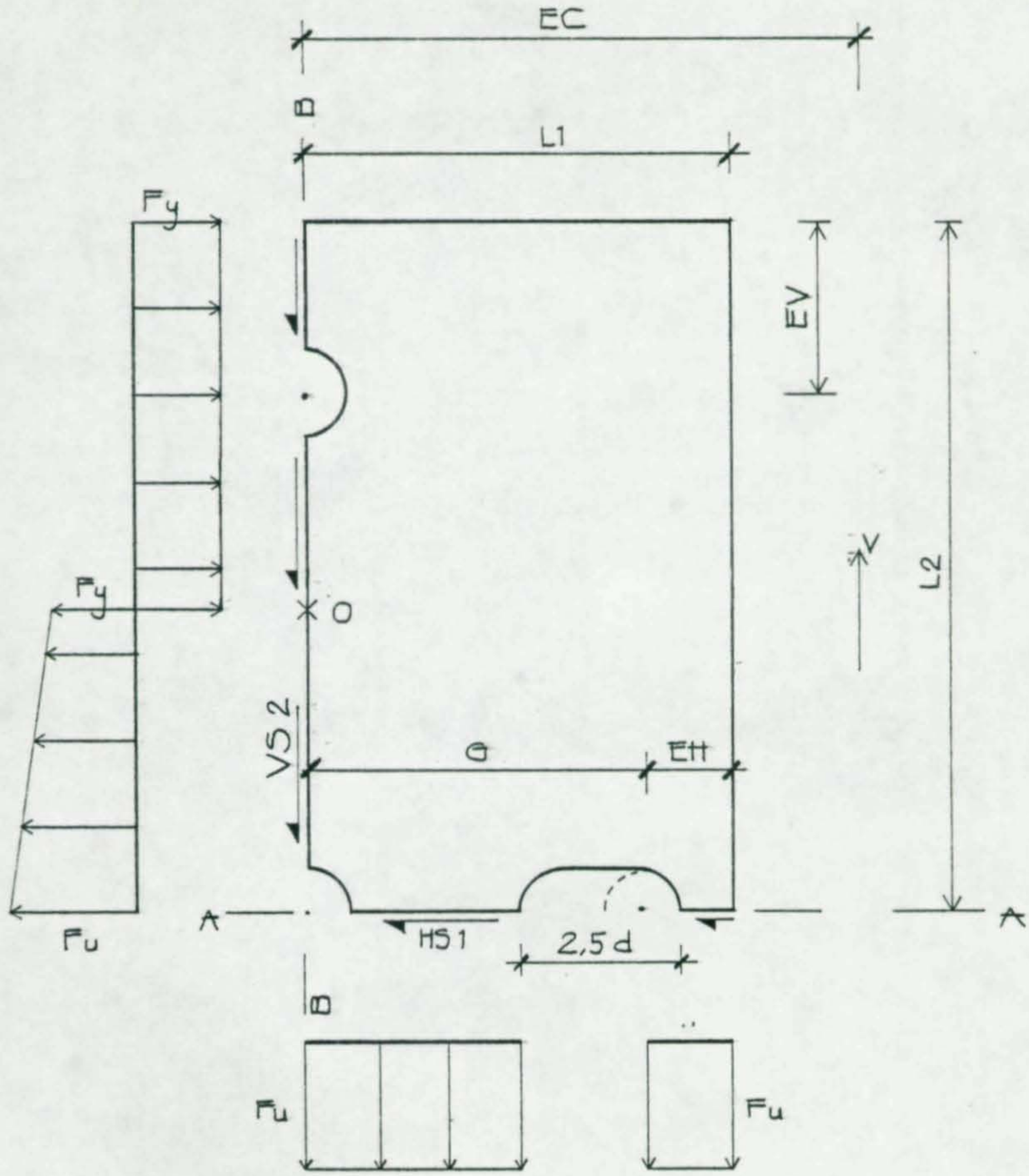


FIGURE 10 MOMENT MODEL, COPED FLANGE, SLOTTED HOLES

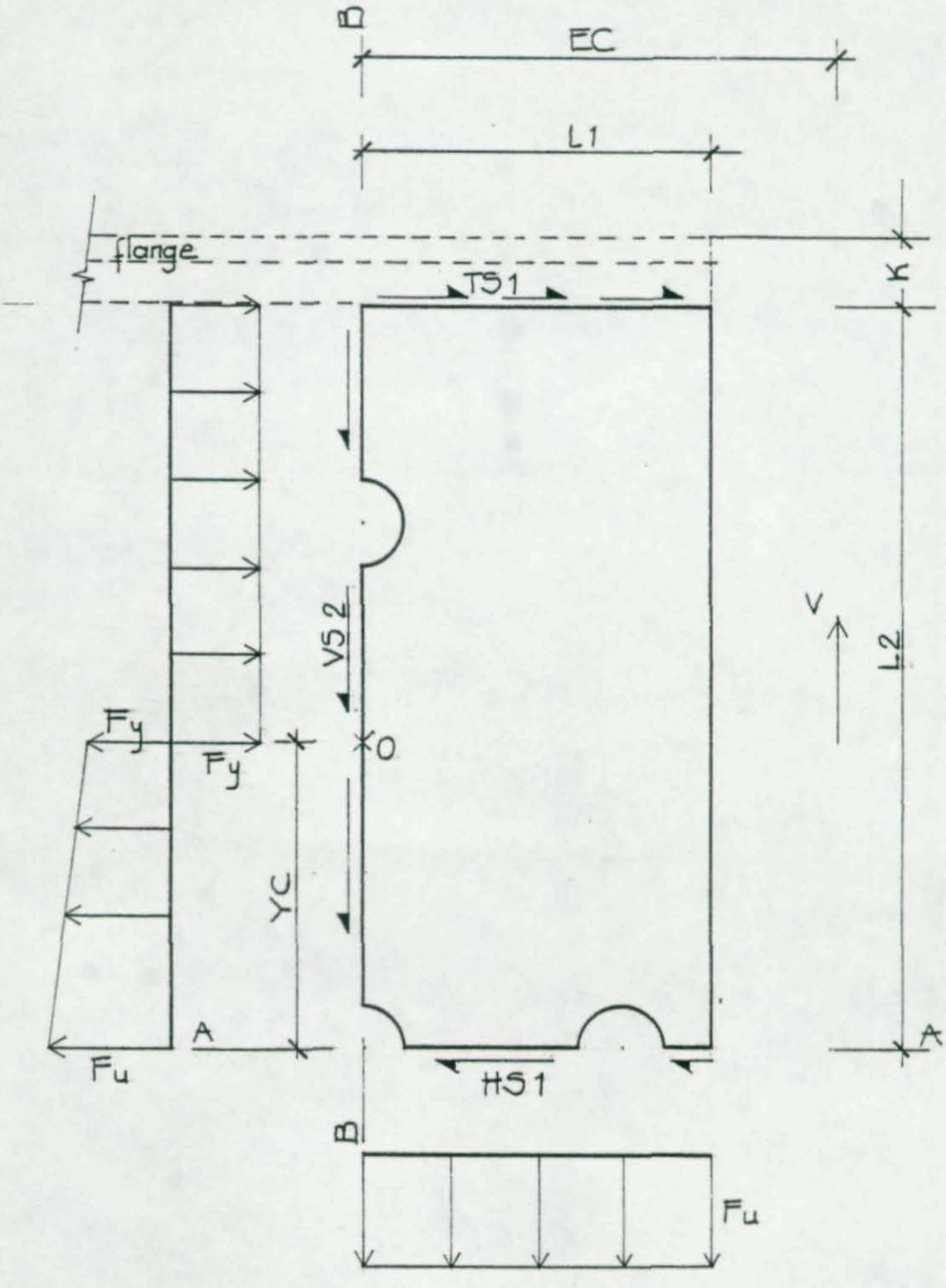


FIGURE 11 - MOMENT MODEL - UNCOPED FLANGE

FIGURE 12 PERCENTAGE OF EXPERIMENTAL RESULT VERSUS TESTS OF COPED BEAMS WITH SINGLE LINE OF BOLTS CONNECTIONS, UNIV, OF TEXAS - AISC PROJECT

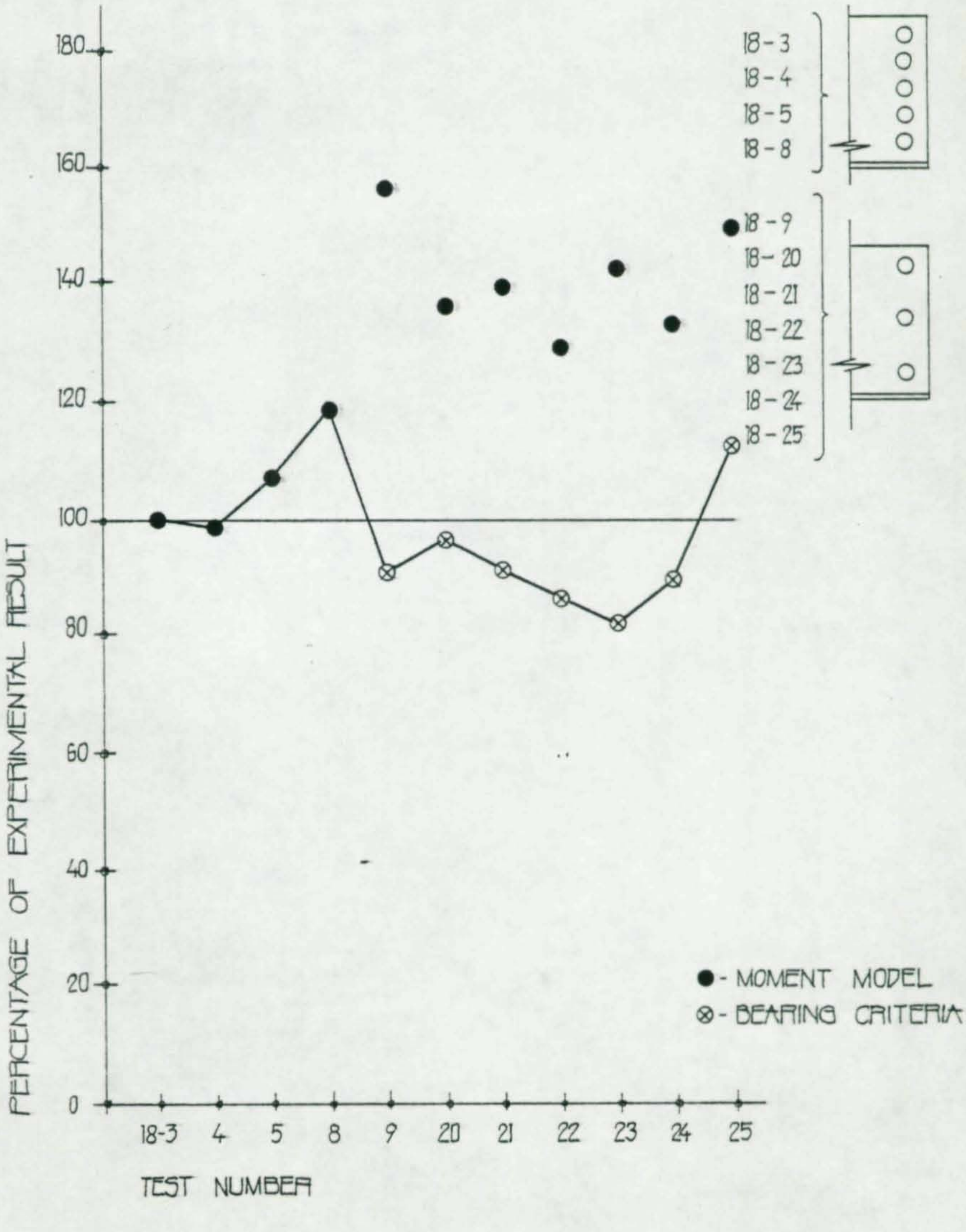


FIGURE 13 PERCENTAGE OF EXPERIMENTAL RESULT VERSUS TESTS OF COPED BEAMS WITH TWO LINES OF BOLTS CONNECTIONS, UNIV. OF TEXAS, AISC PROJECT

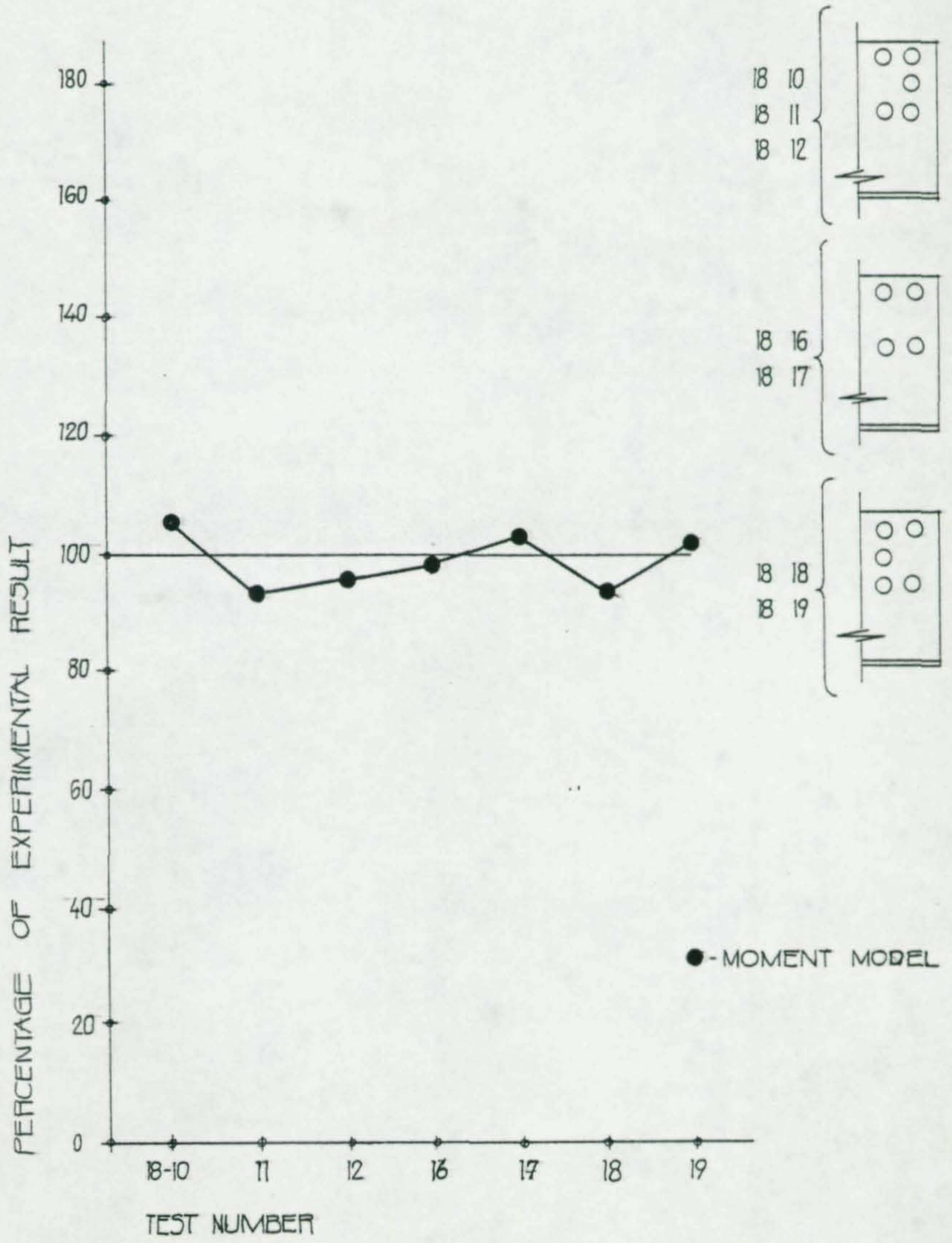


FIGURE 14 PERCENTAGE OF EXPERIMENTAL RESULT VERSUS TESTS OF UNCOPED BEAMS, UNIV. OF TEXAS - -AISC PROJECT,

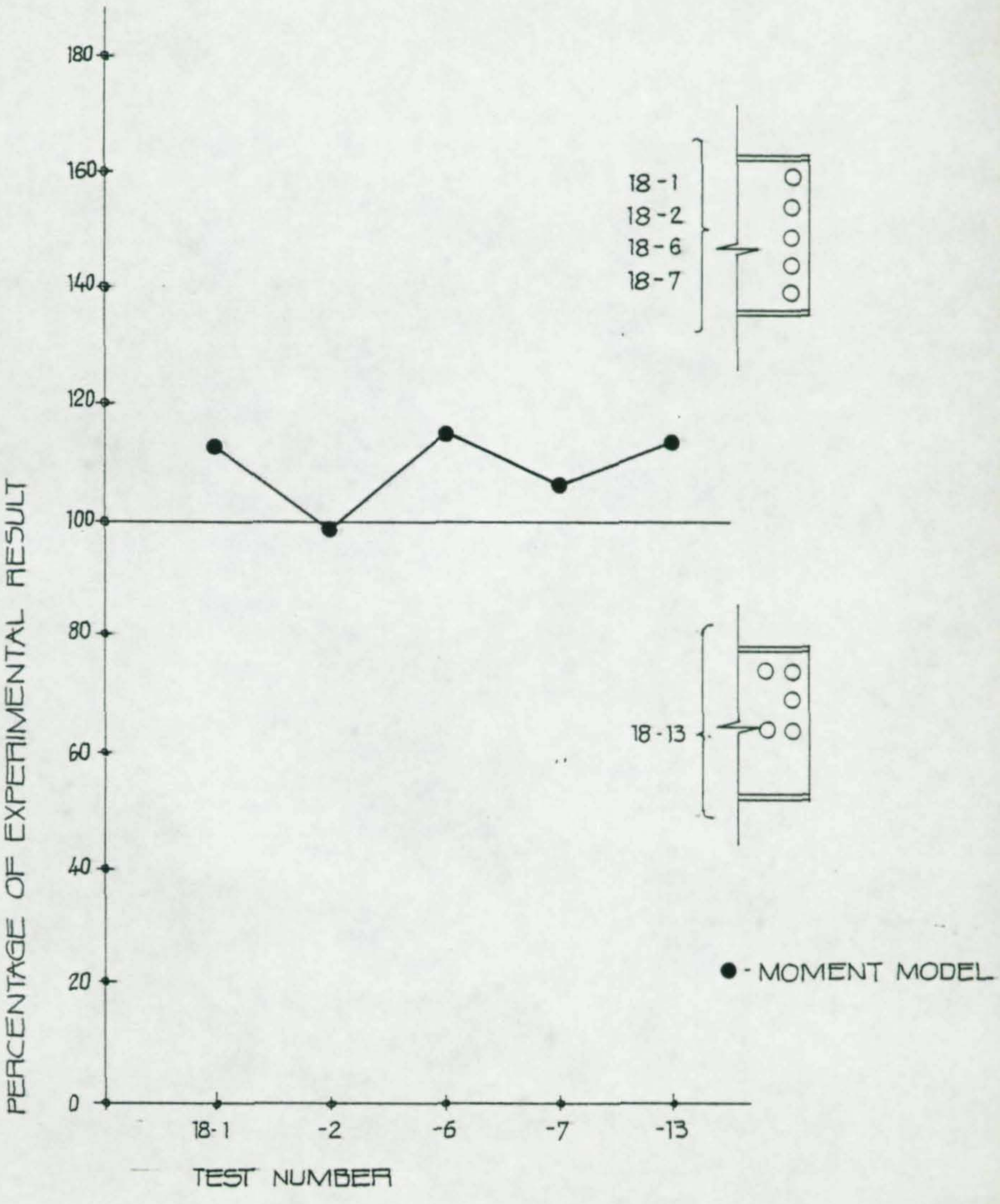


FIGURE 15 COMPARISON OF MODELS FOR COPED BEAMS WITH SINGLE LINE OF BOLTS CONNECTIONS,

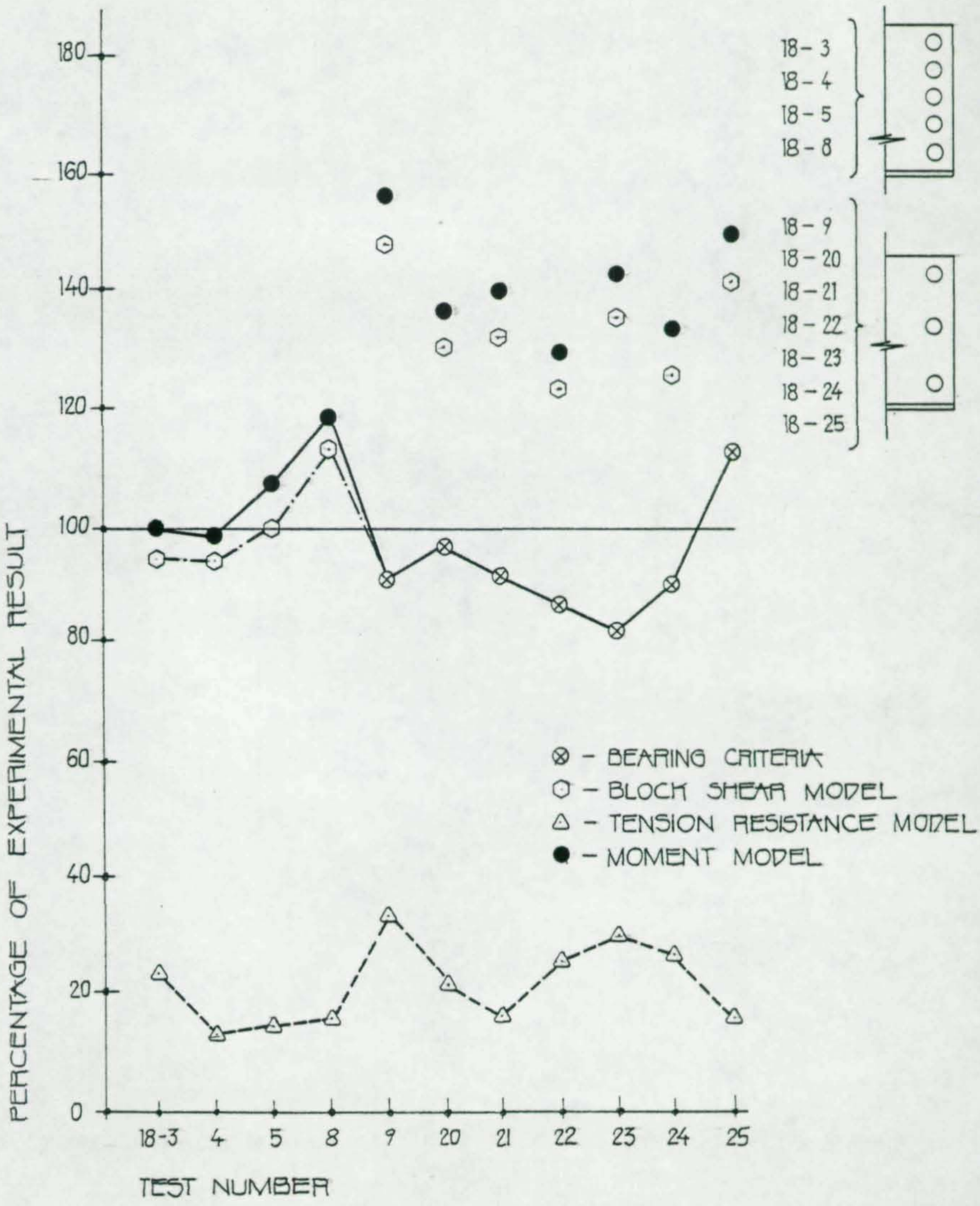


FIGURE 16 COMPARISON OF MODELS FOR COPED BEAMS WITH TWO LINES OF BOLTS CONNECTIONS.

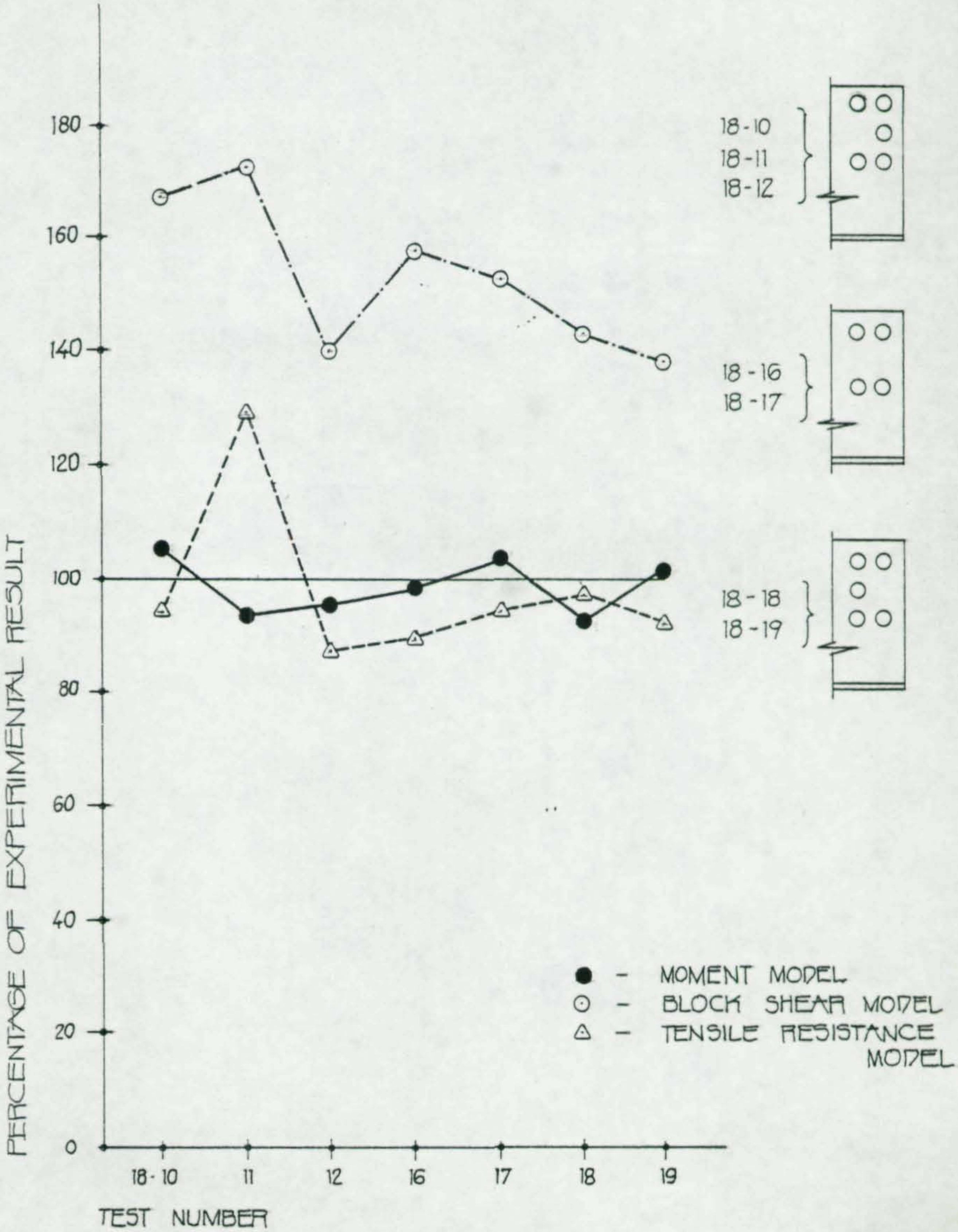


FIGURE 17 COMPARISON OF CSA SPECIFICATION TO "MOMENT" MODEL FOR COPED BEAMS WITH SINGLE LINE OF BOLTS CONNECTIONS

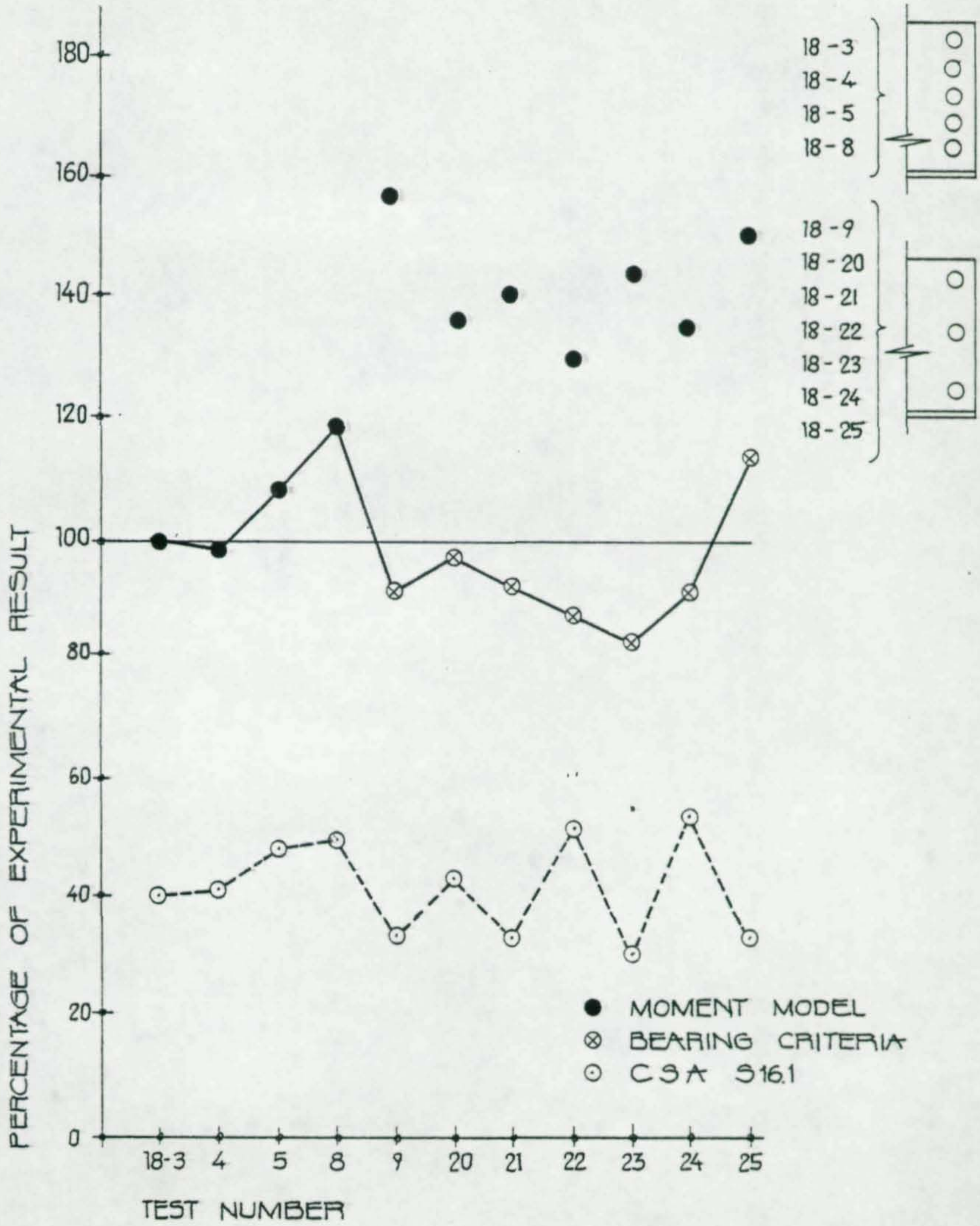


FIGURE 18 COMPARISON OF CSA SPECIFICATION TO "MOMENT" MODEL FOR COPED BEAMS WITH TWO LINES OF BOLTS CONNECTIONS,

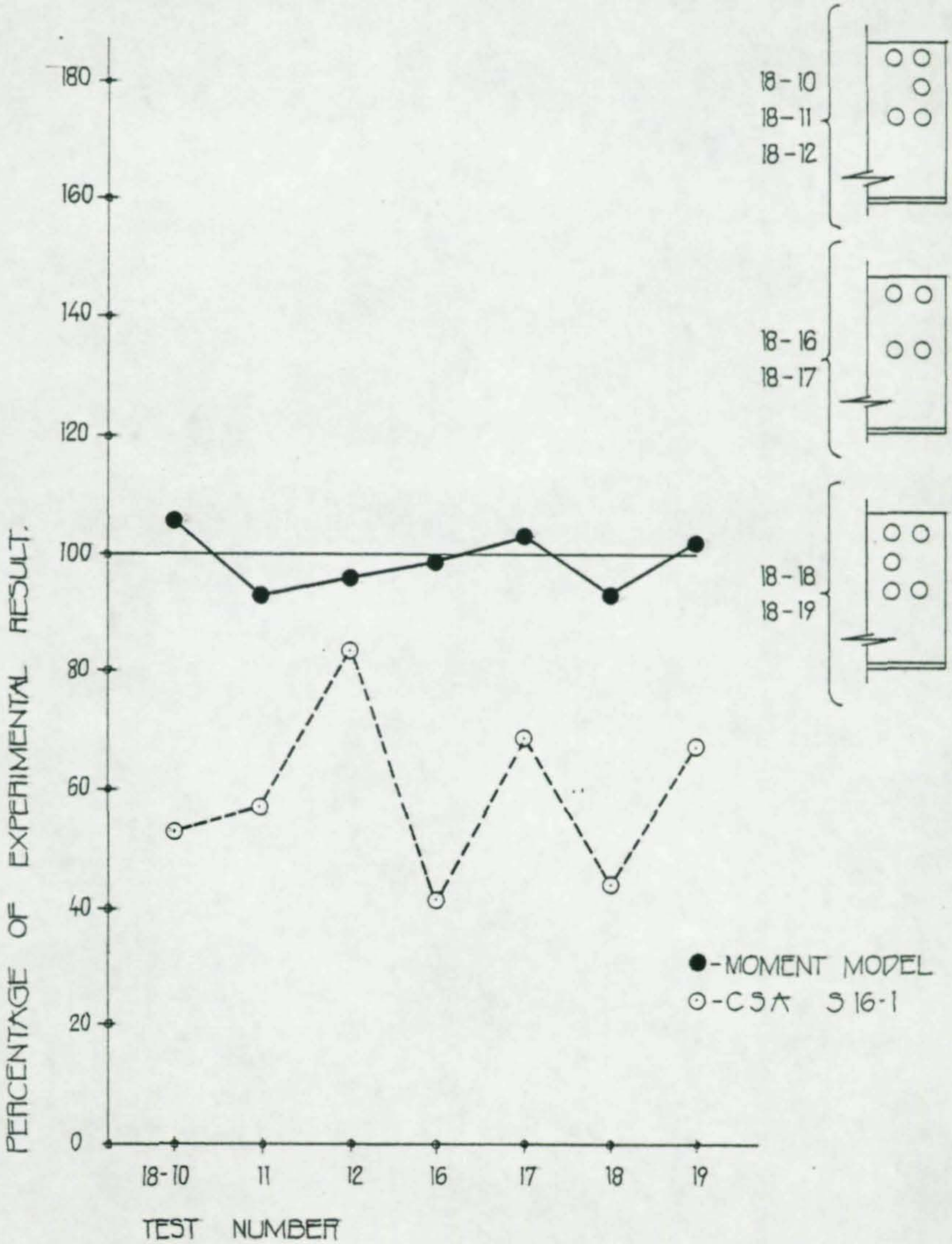


FIGURE 19 COMPARISON OF REVISIONS FOR CISC COMPUTER PROGRAM, SINGLE LINE OF BOLTS CONNECTIONS.

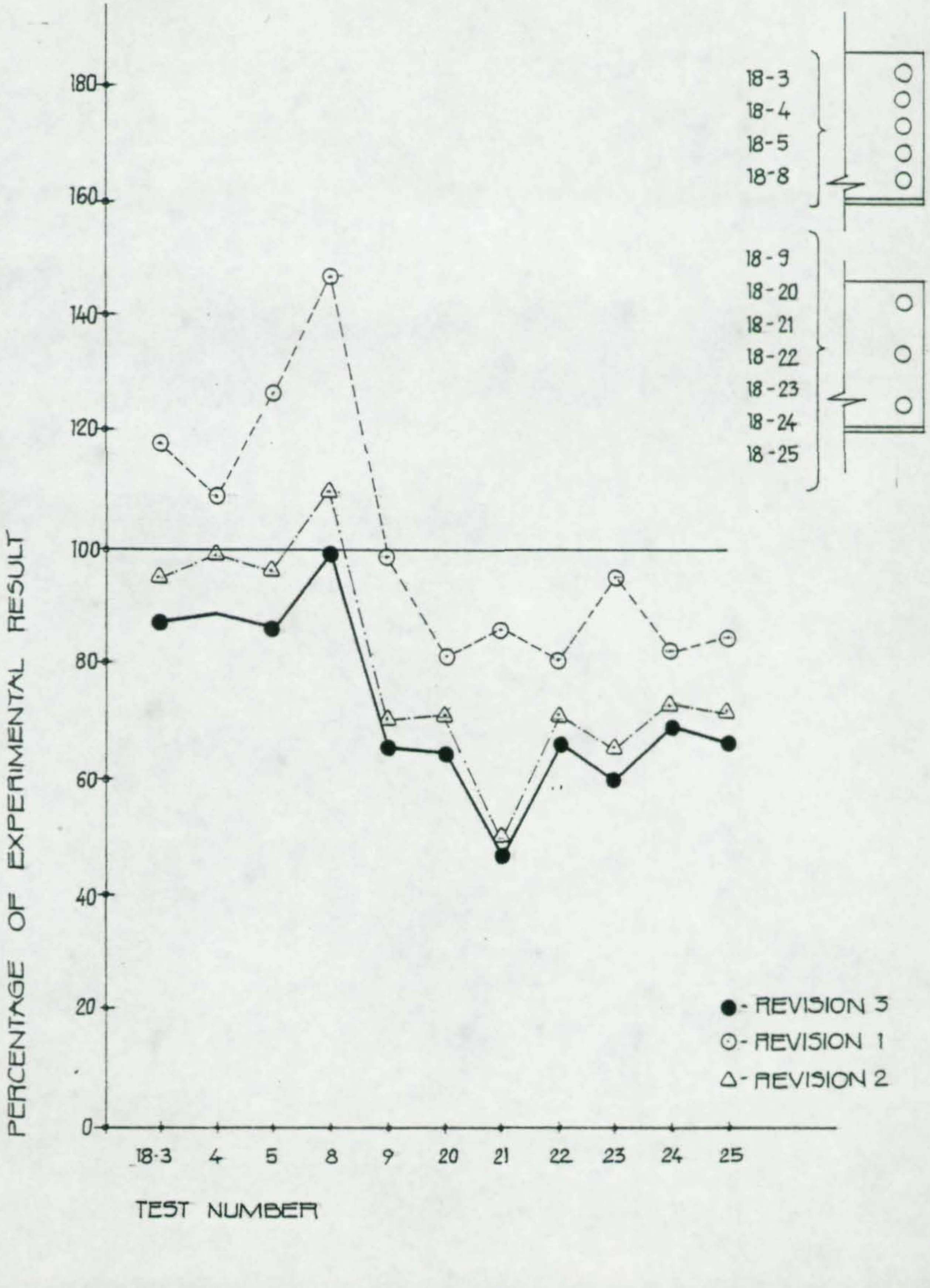


FIGURE 20 COMPARISON OF REVISIONS FOR CISC COMPUTER PROGRAM, TWO LINES OF BOLTS CONNECTIONS

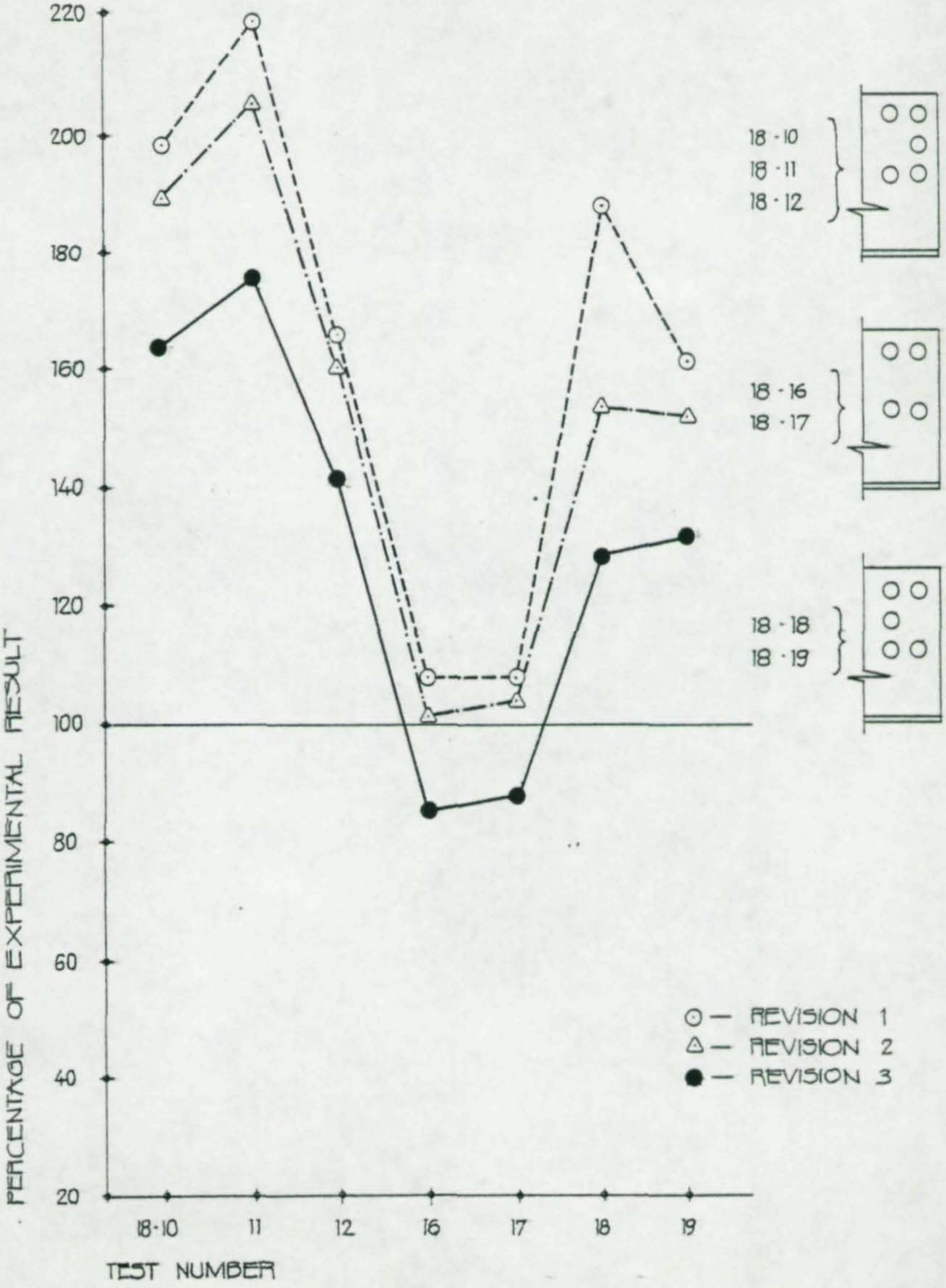
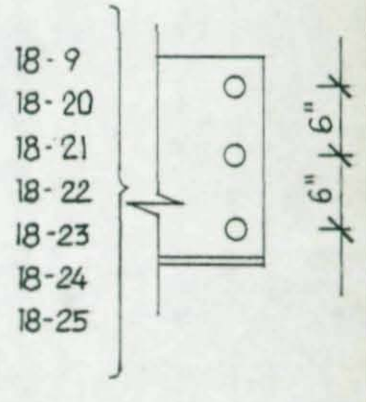


FIGURE 21 COMPARISON OF RESULTS OBTAINED BY REDUCING ECCENTRICITY AND REMOVING END TEAR-OUT LIMIT.



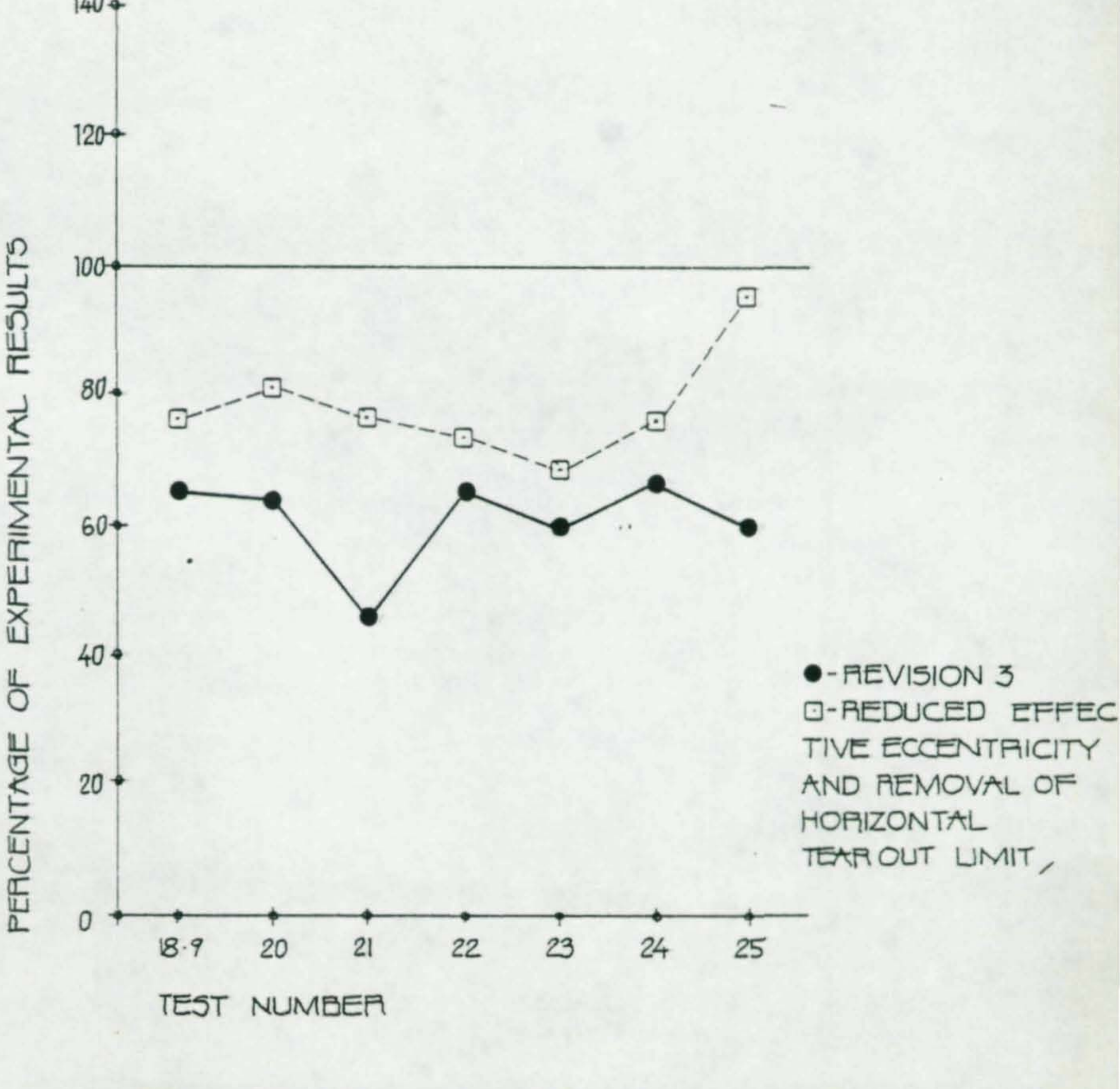


FIGURE 22 COMPARISON OF REVISION 3 TO CURRENT CSA SPECIFICATION ; SINGLE LINE OF BOLTS CONNECTIONS,

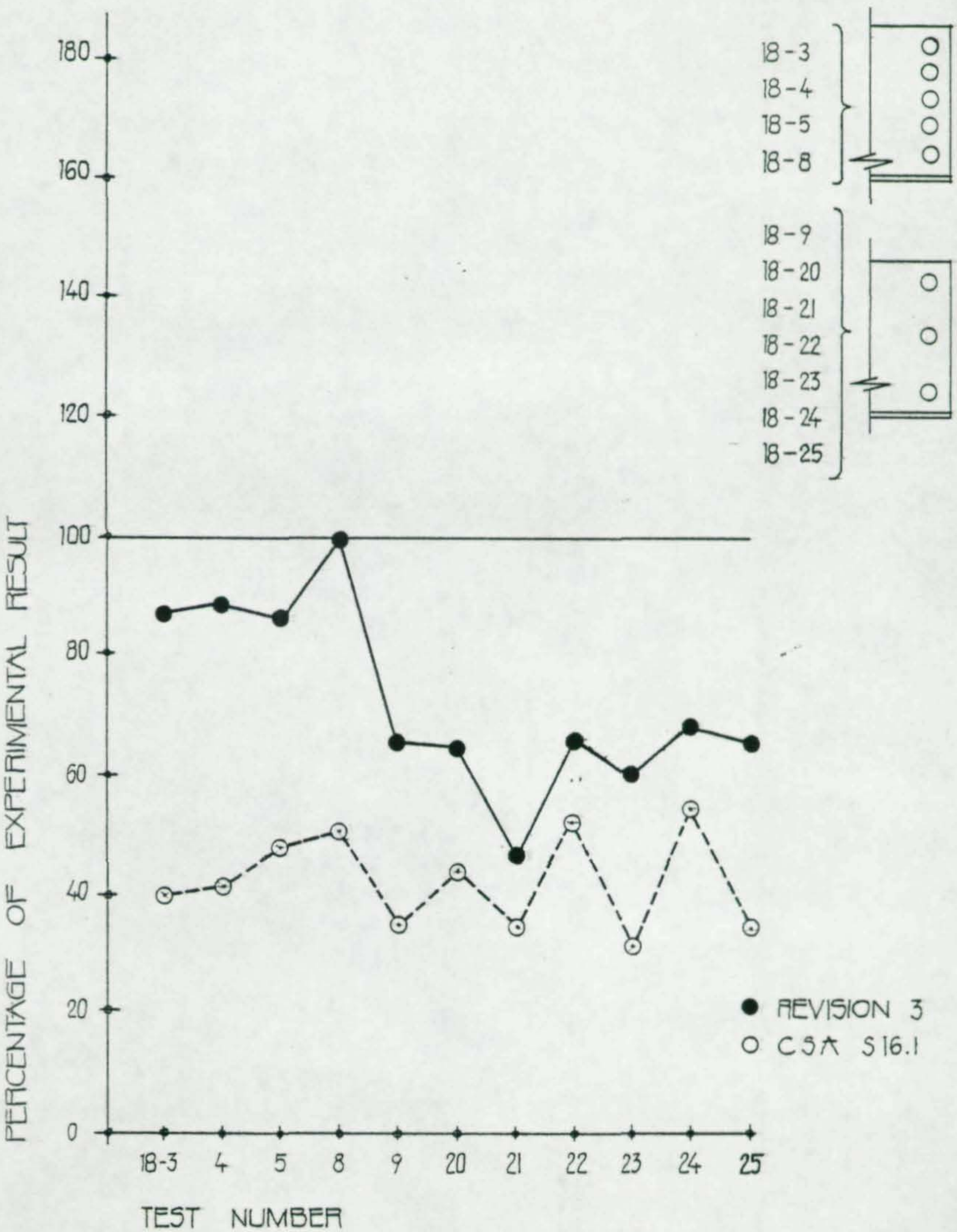


FIGURE 23 COMPARISON OF REVISION 3 TO CURRENT CSA SPECIFICATION; DOUBLE LINE OF BOLTS CONNECTIONS

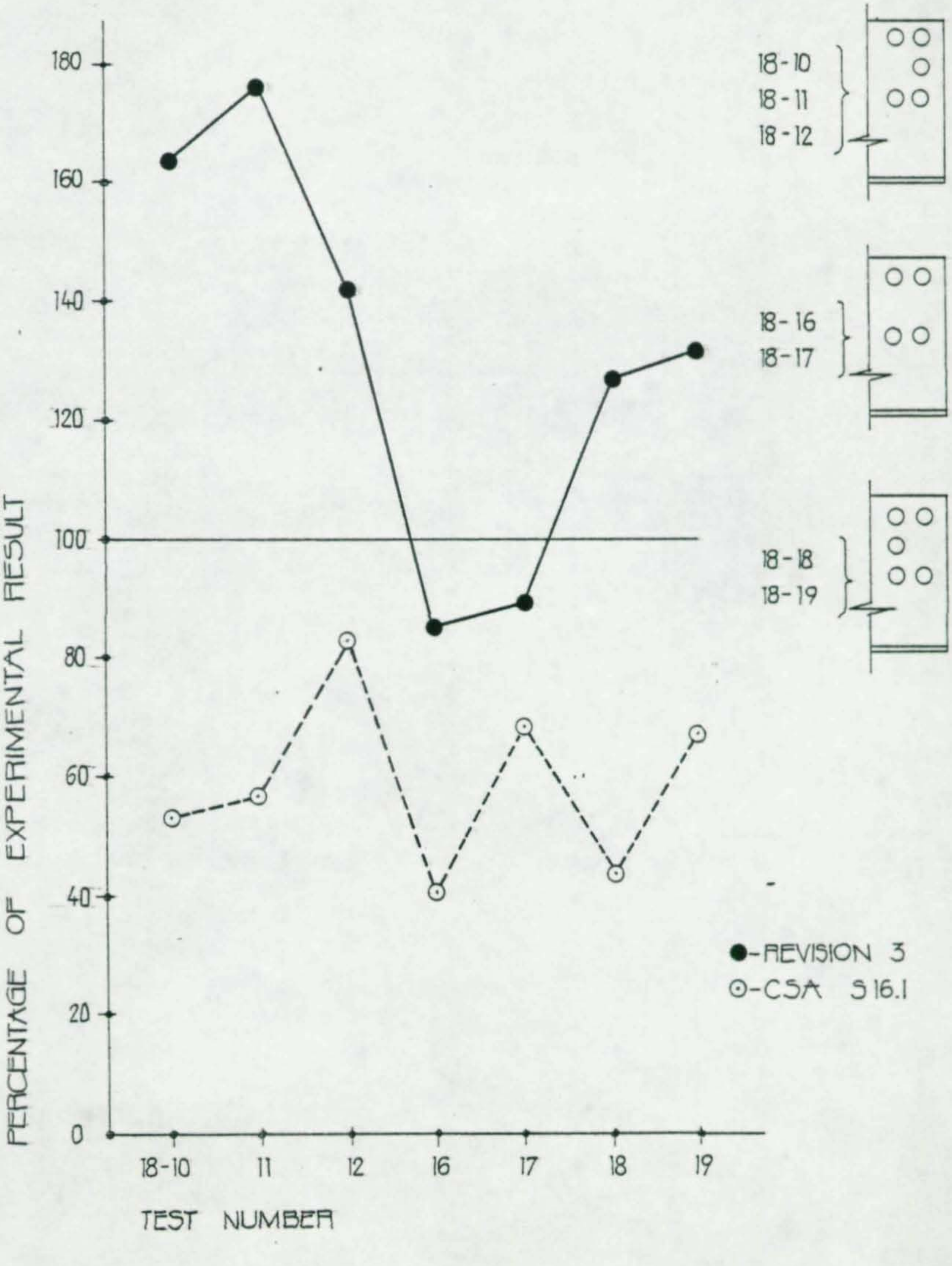
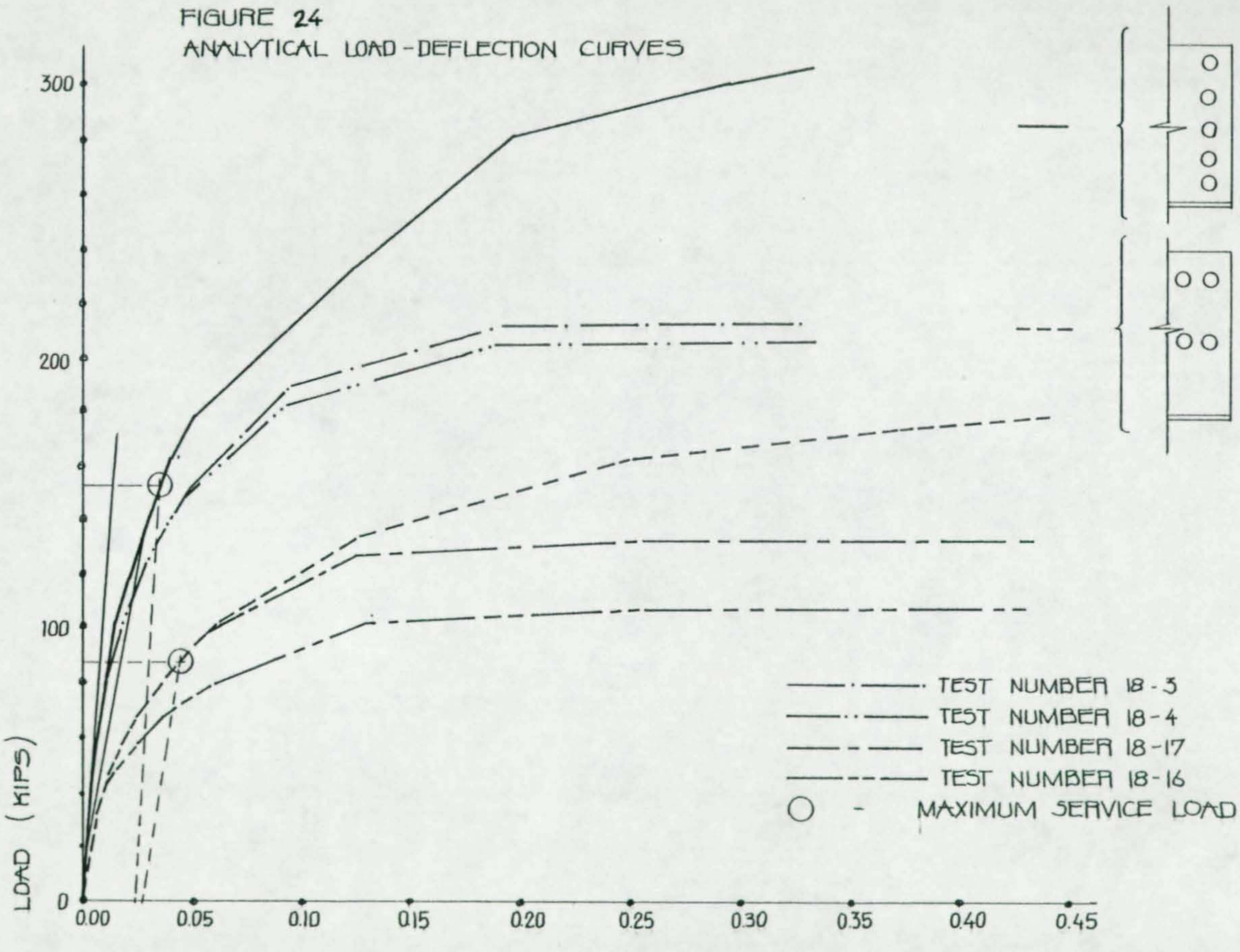


FIGURE 24

ANALYTICAL LOAD-DEFLECTION CURVES



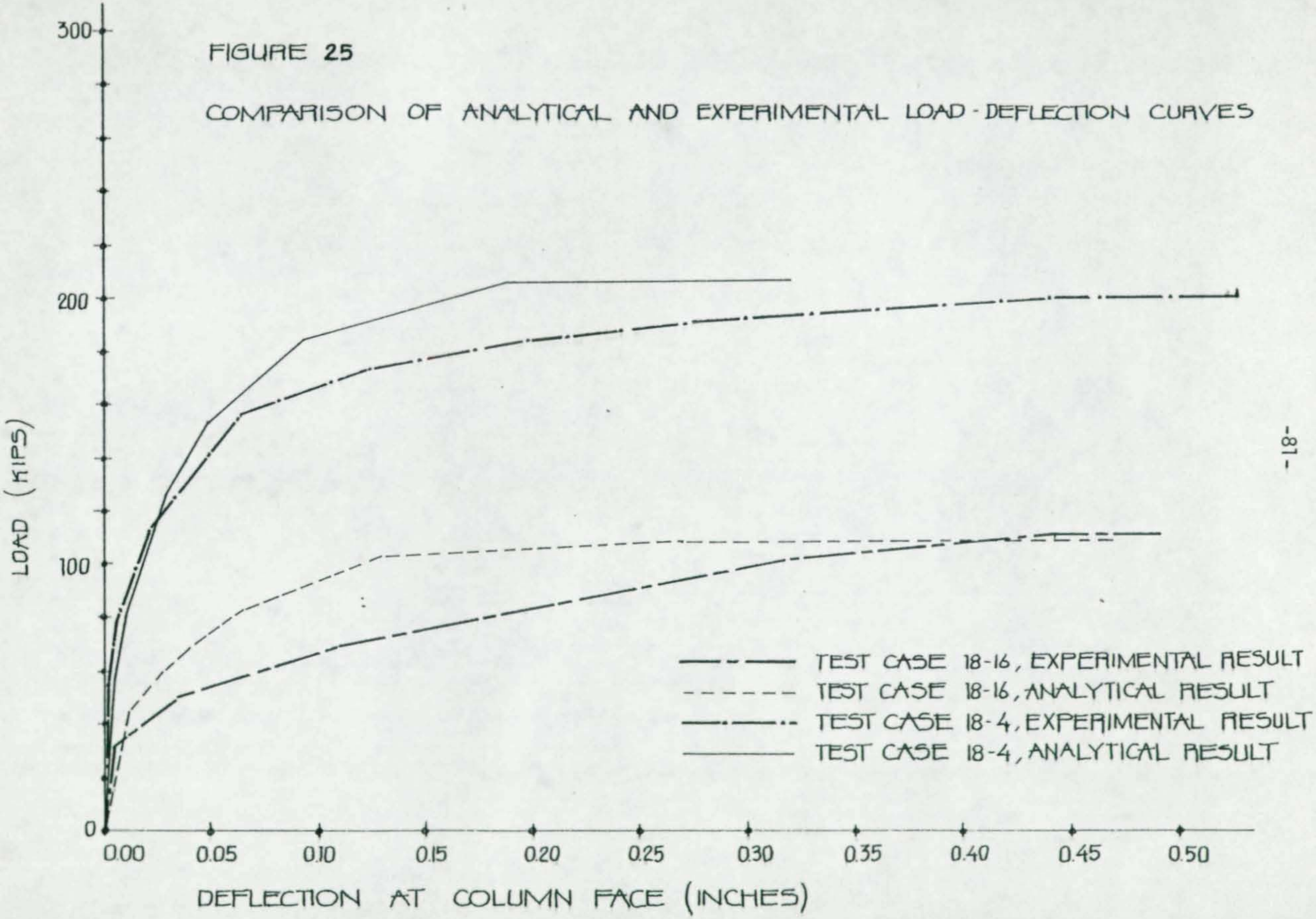


FIGURE 25

COMPARISON OF ANALYTICAL AND EXPERIMENTAL LOAD-DEFLECTION CURVES

REFERENCES

1. Fisher, J.W. and Struik, J.H., "Guide to Design Criteria for Bolted and Riveted Joints", John Wiley and Sons, New York, 1974.

2. "Steel Structures for Buildings - Limit States Design", CSA Standard S16.1-1974, Canadian Standards Association, Rexdale, Ontario, 1974.

3. "Specifications for Structural Joints Using ASTM A325 or A490 Bolts", Research Council on Riveted and Bolts Structural Joints, February, 1976.

4. Yura, J.A, Birkemoe, P.C., and Ricles, J.M., "Beam Web Shear Connections - An Experimental Study", Beam-to-Column Building Connections: State of the Art, 1980 ASCE Spring Convention, Portland, Oregon, April, 1980, pp. 46-66.

5. Birkemoe, P.C. and Gilmor, M.I., "Behaviour of Bearing Critical Double Angle Beam Connections", Engineering Journal, American Institute of Steel Construction, Vol. 15, No. 4, 1978, pp. 109-115.

6. "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", February 1969 and November 1978, American Institute of Steel Construction, New York.

7. Hogan, T.J., and Thomas, I.R., "Bearing Stress and Edge Distance Requirements for Bolted Steelwork Connections", Journal of the Australian Institute of Steel Construction, Vol. 13, No. 3, 1979, AT ISSN 0049-205.

8. de Back, J. and de Jong, A., "Measurements on Connections with High Strength Bolts, Particularly in View of the Permissible Arithmetical Bearing Stress", Report 6-68-3 Stevin Laboraotry, Delft University of Technology, Delft, The Netherlands, 1968.

APPENDIX A

REVISED CISC COMPUTER PROGRAM FOR ECCENTRIC LOADS ON BOLT GROUPS

```
*******************************************************************************************
* WATFIV  VER1 LEV5.                                              MONITOR,VERSION 3C      *
*                                      H WILSON                                           *
* MONDAY AUGUST 18, 1980.                                                        5:02 PM  *
*******************************************************************************************

$JOBW  ID=' H WILSON ' , T=3
        C       L.S.D. HANDBOOK
        C       ----> COMPUTE C FOR ECCENTRIC LOADS ON BOLT GROUPS
        C          VERSION 2 - EXACT METHOD       22 JANUARY 1976
        C
   1            INTEGER LINE(131),BLANK
   2            INTEGER UNITS
   3            REALB(5),D(3),DC,E(12),C(12),ICR(12)
   4            REAL RU(2),DELTA(2),LAMDA(2),MU(2),X(12,4),Y(12,4),R(12,4) ,INC
   5            REAL XSTART(12)
   6            REAL BSI(5)
   7            DIMENSION JOBID(20)
   8            DATA RU/74.0,00.0/,DELTA/0.34,0.00/,LAMDA/0.55,0.00/,MU/10.0,0.00/
   9            DATA BSI/80.,90.,100.,120.,160./
  10            DATA B/6.,3.,4.,5.,0./,BLANK/' '/
  11            DATA ISI,IMP/1,2/
        C
        C       ADDITIONAL DATA CARDS
  12            DATA M/2/              ,AB/0.4418/,DB/0.75/,DSB/0.8750/
        C       M=NUMBER OF SHEAR PLANES
        C       AB=CROSS-SECTIONAL AREA OF BOLT
        C       DB=BOLT DIAMETER
        C       DSB=DIAMETER OF SLOTTED BOLT HOLE
        C       DBB=BOLT HOLE DIAMETER
        C---->FOLLOWING DATA CARD IS REQUIRED ONLY IF SERVICEABILITY CRITERIA
        C       IS TO BE INVESTIGATED.
  13            DATA INBV/2/,NBV/1/,NEV/4/,IS/1/
        C       INBV=NUMBER OF BOLT PER ROW FOR THE TEST CASE UNDER CONSIDERATION
        C       NBV=PITCH OF BOLTS FOR THE TEST UNDER CONSIDERATION
        C       NEV=MOMENT ARM FOR THE TEST UNDER CONSIDERATION
        C       IF 'IS' = 2, THEN THE ROWS OF BOLTS ARE NOT SYMMETRICAL
        C
        C        UNITS = ISI FOR SI
        C              = IMP FOR IMPERIAL
        C        NOTE; DUE TO ORIGINAL NATURE OF THIS PROGRAM, IF UNITS = ISI
        C              THE INTERNAL COMPUTATIONS ARE DONE IN IMPERIAL UNITS
        C              AND CONVERTED TO METRIC FOR PURPOSES OF OUTPUT
        C
  14            UNITS=ISI
  15            UNITS=IMP
  16            IG=5
  17            IO=6
        C
  18            READ(IG,8001) JOBID
  19      8001  FORMAT(20A4)
  20            WRITE(IO,9001) JOBID
  21            READ(IG,8002) NB1,NB2,LMB,ISH,ISR
  22      8002  FORMAT(5I5)
        C---->IF THE ROWS OF BOLTS ARE SYMMETRICAL THE DATA CARD CAN BE BLANK
        C       IF SLOTTED HOLES ARE USED , THEN LET 'ISH' BE GREATER THAN 0
        C---->IF SERVICEABILITY REQUIREMENT IS TO BE CHECKED,THEN LET ISR=1
        C       NB1=NUMBER OF BOLTS IN ROW 1
        C       NB2=NUMBER OF BOLTS IN ROW 2
        C       NOTE: ROW 2 IS THE ROW CLOSEST TO THE END OF THE BEAM
```

```
C       LMB=LOCATION OF MISSING BOLT
        IF(ISR.EQ.1) WRITE(IO,9002)
C
        IF(IS.NE.2) GO TO 1002
        WRITE(IO,9006)
        IF(NB1.LT.NB2) WRITE(IO,9004) NB2,NB1
        IF(NB1.GT.NB2) WRITE(IO,9005) NB1,NB2
   1002 CONTINUE
C
C       CALCULATE TEAROUT LOAD
C
        READ(IG,8003) ED,T,FUW,EV
   8003 FORMAT(4F10.0)
C       ED=HORIZONTAL END DISTANCE
C       T=WEB THICKNESS
C       FUW=ULTIMATE TENSILE STRENGTH OF WEB
C       EV=VERTICAL END DISTANCE
        WRITE(IO,9007) T,FUW,ED,EV
        DEB=DB
        IF(ISH.GT.0) DB=DSB
        ALT=2.0*T*(ED-(DB+1.0/8.0)/2.0)*0.66*FUW
        ALTV=2.0*T*(EV-(DB+1.0/8.0)/2.0)*0.66*FUW
C       ALT=HORIZONTAL TEAROUT LOAD
C       ALTV=VERTICAL TEAROUT LOAD
C
        LFMAX=1500
        KODE=1
C       KODE=1 FOR A325 BOLTS
        IPT=2
        IPT=3
        IPT=4
        IPT=5
        IPT=6
        IPT=0
        DO 125 I=1,12
        ICR(I)=0.
        C(I)=0.
    125 CONTINUE
C           NL = NUMBER OF VERTICAL LINES
        NLMAX=4
        NLMAX=1
        NLMAX=2
C           NF = NUMBER OF FASTENERS (BOLTS)
        NFMAX=12
        NFMAX=6
C           NE = NUMBER OF MOMENT ARM VALUES
        NE=12
        NE=1
        NE=4
        NE=6
C           NB = NUMBER OF DIFFERENT PITCHES, B
        NB=5
        NB=4
        NB=1
        NB=2
        A=1.0
        E(1)=2.5
        DO 41 J=2,12
        E(J)=E(J-1)+0.5
     41 CONTINUE
```

```
C         E(I) IN SI = 75,100,125,150,175,200,225,250,300,400,500,600
C
C          SET EQUIVALENT IMPERIAL ECCENTRICITIES FOR METRIC VALUES
          IF(UNITS.EQ.IMP) GO TO 110
          DO 111 I=1,NE
      111 E(I)=E(I)*25./25.4
C          SET EQUIVALENT IMPERIAL PITCHES FOR METRIC VALUES
          DO 112 I=1,NB
      112 B(I)=BSI(I)/25.4
      110 CONTINUE
          IO=6
          DO 1014 NL=1,NLMAX
C            LOOP OVER NUMBER OF VERTICAL LINES
C            SET THE SPACING,D,BETWEEN VERTICAL LINES
          IF(UNITS.EQ.ISI) GO TO 120
          D(1)=4.
          D(1)=3.
          D(2)=4.
          D(3)=6.
          IF(NL.LE.2) D(2)=6.
          IF(NL.LE.2) D(3)=12.
C          SET EQUIVALENT IMPERIAL GAUGES FOR SI VALUES
          IF(UNITS.EQ.IMP) GO TO 121
      120 D(1)=80.
          D(2)=320.
          IF(NL.EQ.3) D(1)=160.
          IF(NL.EQ.4) D(1)=240.
          IF(NL.EQ.4) D(2)=480.
      121 CONTINUE
C            LOOP OVER NUMBER OF HORIZONTAL SPACINGS
C            ND = NUMBER OF HORIZONTAL SPACINGS
          ND=3
          ND=2
          ND=1
          IF(NL.EQ.1) ND=1
          DO 100 ID=1,ND
          IID=D(ID)
C            PRINT HEADINGS AT TOP OF A NEW PAGE
          DC=D(ID)
          WRITE(IO,601)
          IF(NL.EQ.1) GO TO 2
          IF(UNITS.EQ.ISI) IIID=IFIX(D(ID)*25.4+0.5)
          IF(UNITS.EQ.ISI) WRITE(IO,662)NL,IIID
          IF(UNITS.EQ.IMP) WRITE(IO,602) NL,IID
        2 CONTINUE
          IF(UNITS.EQ.ISI) WRITE(IO,663)(I,I=75,250,25),(J,J=300,600,100)
          IF(UNITS.EQ.IMP) WRITE(IO,603)(E(I),I=1,12)
C            LOOP OVER PITCH, B
          DO 200 IB=1,NB
          XDIM=0.
          DO 201 INIT=1,12
      201 XSTART(INIT)=-0.0
C            LOOP OVER NUMBER OF BOLTS
          DO 400 NF=1,NFMAX
C*************************************************************************
          IF(NF.EQ.1.AND.NL.EQ.1) GO TO 400
          DO 3 IJ=1,NL
          YDIM=0.
          DO 3 II=1,NF
          IF(II.EQ.1) GO TO 3
```

```
      YDIM=YDIM+B(IB)
    3 Y(II,IJ)=YDIM
      GO TO (11,12,13,14,99,99,99),NL
   11 XDIM=0.
      DO 20 II=1,NF
   20 X(II,1)=XDIM
      GO TO 15
   12 XDIM=D(ID)
      DO 21 II=1,NF
      X(II,1)=0.
   21 X(II,2)=XDIM
      GO TO 15
   13 XDIM=2.0*D(ID)
      DO 22 II=1,NF
      X(II,1)=0.
      X(II,2)=D(ID)
   22 X(II,3)=XDIM
      GO TO 15
   14 XDIM=2.0*D(ID)+DC
      DO 23 II=1,NF
      X(II,1)=0.
      X(II,2)=D(ID)
      X(II,3)=D(ID)+DC
   23 X(II,4)=XDIM
      GO TO 15
   99 WRITE(IO,604)
      STOP
   15 CONTINUE
C           CCMPUTE DISTANCE TO CENTRIOD CF BOLT GROUP
      CX=XDIM/2.0
      CY=YDIM/2.0
C         COMPUTE X & Y COORDINATES WRT CENTRIOD
      DO 24 II=1,NF
      DO 24 IJ=1,NL
      X(II,IJ)=X(II,IJ)-CX
      Y(II,IJ)=Y(II,IJ)-CY
   24 CONTINUE
      IF(IPT.EQ.2) WRITE(IO,600)((X(I,J),J=1,NL),(Y(I,J),J=1,NL),I=1,NF)
      DO 300 I=1,NE
C         INC=INCREMENT OF CHANGE OF I.C., INCHES
C                         ALCNG THE NEGATIVE X AXIS
      RMAX=0.
      XIC=XSTART(I)
      FACT=1.0
      INC=1.0
C         LCOP CVER I.C. VALUES
      DO 30 K=1,LPMAX
      INC=INC*FACT
      IF (ABS(INC).LE.0.005) GO TO 50
      XIC=XIC-INC
      IF(I.NE.NEV) GO TO 1039
      IF(NF.NE.INBV) GO TO 1039
      IF(IB.NE.NBV) GO TO 1039
      IF (IPT.EQ.6) WRITE(6,612) XIC
  612 FORMAT('0XIC= ',E10.3)
 1039 CCNTINUE
 1009 CCNTINUE
C        COMPUTE RADIUS
      DO 31 II=1,NF
      DO 31 IJ=1,NL
```

```
      R(II,IJ)=SQRT((XIC-X(II,IJ))**2+(0.0-Y(II,IJ))**2)
      IF(R(II,IJ).GT.RMAX) RMAX=R(II,IJ)
   31 CONTINUE
      IF(IPT.EQ.3) WRITE(IO,607)XIC, RMAX,((R(II,J),J=1,NL),II=1,NF)
      FTM=0.
C        COMPUTE DEFLECTION & LOAD ON EACH BOLT
      RTV=0.
      DO 35 II=1,NF
      DO 35 IJ=1,NL
      DEF=R(II,IJ)/RMAX*DELTA(KODE)
      F=RU(KODE)*(1.-EXP(-DEF*MU(KODE)))**LAMDA(KODE)
C
C
C
C     MATERIAL DEFORMATION LIMIT
C
      FBB=3.0*T*DBB*FUW
      IF(F .GT.FBB) F=FBB
C
C
C     CHECK ON BOLT TEAROUT
C
C
C     HORIZONTAL TEAROUT
C
      IF(II.NE.1) GO TO 1007
      IF(IJ.NE.NL    ) GO TO 1007
      IF(F.GT.ALT) F=ALT
 1007 CONTINUE
C
C     VERTICAL TEAROUT
C
C
      IF(II.NE.NF   ) GO TO 1011
      IF(IJ.NE.1) GO TO 1011
      IF(F.GT.ALTV) F=ALTV
 1011 CONTINUE
C
      IF(IS.NE.2) GO TO 1005
C
C     UNSYMMETRICAL ROWS OF BOLTS
      IF(NL.NE.2) GO TO 1005
      IF(NF.NE.3) GO TO 1005
      IF(NB1.LT.NB2) GO TO 1006
      IF(II.NE.LMB) GO TO 1005
      IF(IJ.NE.2) GO TO 1005
      R(II,IJ)=0.0
      F=0.0
      DEF=0.0
      GO TO 1005
 1006 CONTINUE
      IF(II.NE.LMB) GO TO 1005
      IF(IJ.NE.1) GO TO 1005
      R(II,IJ)=0.0
      F=0.0
      DEF=0.0
 1005 CONTINUE
C
      FM=F*R(II,IJ)
      IF (R(II,IJ).EQ.0.0) GO TO 42
      FV=ABS(XIC-X(II,IJ))/R(II,IJ)*F
```

```
          GO TO 43
       42 FV=0.0
       43 IF (X(II,IJ).LT.XIC) FV=-FV
C
C
          IF(I.NE.NEV) GO TO 1008
          IF(NF.NE.INBV) GO TO 1008
          IF(IB.NE.NBV) GO TO 1008
          IF (IPT.EQ.6) WRITE(6,611) DEF,F,FV
      611 FORMAT(1X,'DEF= ',E11.3,'  F= ',E11.3,'  FV= ',E11.3)
     1008 CONTINUE
C
C
          RTV=RTV+FV
          FTM=FTM+FM
       35 CONTINUE
C           PM =  FORCE DUE TO MCMENT
          PM=FTM/(E(I)-XIC )
          ERR=RTV-FM
          IF (ABS(ERR/PM).LE.0.005) GO TO 50
          IF (ERR.GT.0.0) GO TO 36
          IF (INC.LT.0.0) GO TO 37
          GO TO 38
       36 IF (INC.LT.0.0) GO TO 38
       37 FACT=-0.5
          GO TO 30
       38 FACT=1.0
          IF(I.NE.NEV) GO TO 30
          IF(NF.NE.INBV) GO TO 30
          IF(IB.NE.NBV) GO TO 30
          IF(IPT.EQ.4) WRITE(IO,608) DEF,F,FM,FV,RTV,FTM,PM,K,XIC
       30 CONTINUE
          WRITE(IO,610)NL,NF,E(I),D(ID),B(IB)
       50 CONTINUE
          ICR(I)=ABS(XIC)
C
C         CALCULATE DEFLECTION AT COLUMN FACE
          IF(ISR.NE.1) GO TO 1034
          IF(I.NE.NEV) GO TO 1034
          IF(NF.NE.INBV) GO TO 1034
          IF(IB.NE.NBV) GO TO 1034
          DCF=DELTA(KODE)*(E(I)-XIC)/RMAX
          RMAXR=RMAX
          DRR=DELTA(KODE)
          RLR=E(I)-XIC
          XICR=XIC
     1034 CONTINUE
C
          GO TO (51,52),KODE
C               REDUCE FROM 3/4-INCH A325 BOLT TO COEFFICENT C FOR TABLES
       51 C(I)= PM/(RU(1)*(1.-EXP(-DELTA(1)*MU(1)))**LAMDA(1))
          GO TO 55
C               REDUCE FROM             A490 BOLT TO C(I)
       52 C(I)= PM/(0.60    *  2.0*0.0000*150.)
       55 CONTINUE
          XSTART(I)=XIC
          IF (XIC.LT.1.0) XSTART(I)=0.0
       60 CONTINUE
          CALL SIGFIG(C(I),C(I),IR)
      300 CONTINUE
```

```
C          PRINT RESULTS
           ANF=NF
           DO 150 ICL=1,131
      150  LINE(ICL)=BLANK
           IF(NL.EQ.1.AND.NF.EQ.1) GO TO 400
           IF(NF.EQ.6 .AND.UNITS.EQ.ISI) CALL OUTLIN(LINE,BSI(IB),6,0)
           IF(NF.EQ.6 .AND. UNITS.EQ.IMP) CALL OUTLIN(LINE,B(IB),5,0)
           CALL OUTLIN(LINE,ANF,13,0)
           IP=13
           DO 151 I=1,NE
           IP=IP+6
           CALL OUTLIN(LINE,C(I),IP,2)
C
           IF(IB.NE.NBV) GO TO 1001
           IF(NF.NE.INBV) GO TO 1001
           IF(I.NE.NEV) GO TO 1001
           CCM=C(I)
     1001  CONTINUE
C
      151  CONTINUE
           CALL OUTLIN(LINE,ANF,92,0)
           IF(NF.EQ.6 .AND. UNITS.EQ.ISI) CALL OUTLIN(LINE,BSI(IB),100,0)
           IF(NF.EQ.6 .AND. UNITS.EQ.IMP) CALL OUTLIN(LINE,B(IB),99,0)
           WRITE(IO,605) LINE
           IF(IPT.EQ.5) WRITE(IO,609)ICR
      400  CONTINUE
           WRITE(IO,606)
      200  CONTINUE
      100  CONTINUE
     1014  CONTINUE
C
           IF(ISR.NE.1) GO TO 1035
           WRITE(IO,9008) JOBID,DCF
           WRITE(IO,9009) RMAXR,DRR,RLR,XICR
     1035   CONTINUE
C
           WRITE(IO,9003)
           STOP
      600  FORMAT( ' ',10X,8F10.2)
      601  FORMAT(1H-,70X,'ECCENTRIC LOADS ON BOLT GROUPS'/86X,'COEFFICIENTS
          1C')
      602  FORMAT(1H0,I2,'    VERTICAL LINES AT A SPACING, D OF ',I2,' INCHES'
          1)
      603  FORMAT(1H0,1X,'PITCH',4X,'NO.',76X,'NO.',3X,'PITCH'/
          1          1H ,3X,'B',6X,'OF',21X,'MOMENT ARM,E, INCHES',36X,'OF',6X,
          2 'B'/  1H ,1X,'INCHES',2X,'BOLTS',3X,F3.1,11(2X,F4.1),3X,'BOLTS',
          3 2X,'INCHES'/)
      604  FORMAT(1H0,//,10X,'**** ADD NEW STATEMENTS FOR BOLT GROUPS WITH ',
          1          'MORE THAN FOUR (4) VERTICAL LINES')
      605  FORMAT(1H ,131A1)
      606  FORMAT(1H )
      607  FORMAT('0',10X,2F10.2/' ',4F10.2)
      608  FORMAT('0',7F10.2,I10,F10.2)
      609  FORMAT(' ',15X,12F6.2)
      610  FORMAT('0',' $$ ITERATION FAILS FOR',I3,' LINES OF',I3, ' BOLTS',
          1 ' WITH E OF',F4.0,' D=',F4.0,' B=',F4.0)
      662  FORMAT(1H0,I2,'    VERTICAL LINES AT A SPACING, D OF ',I3,' MM')
      663  FORMAT(1H0,1X,'PITCH',4X,'NO.',76X,'NO.',3X,'PITCH'/
          1          1H ,3X,'B',6X,'OF',21X,'MOMENT ARM,E,    MM   ',36X,'OF',6X,
          2 'B'/  1H ,1X,'   MM   ',2X,'BOLTS',3X,I2,11(3X,I3),3X,'BOLTS',2X,
```

```
     3 '   MM   '/)
 9001 FORMAT('1',20A4)
 9002 FORMAT(' ',' SERVICEABILITY REQUIREMENT LOADS ')
 9003 FORMAT('1',' END OF OUTPUT ')
 9004 FORMAT(' ','   FOR THE CASE WHERE THE NUMBER OF BOLTS = ',I3,',',
     1          ' THE FIRST ROW OF BOLTS HAS ONLY ',I3,' BOLTS.')
 9005 FORMAT(' ','   FOR THE CASE WHERE THE NUMBER OF BOLTS = ',I3,',',
     1          ' THE SECOND ROW OF BOLTS HAS ONLY ',I3,' BOLTS.')
 9006 FORMAT(' ','   THE SECOND ROW OF BOLTS IS THE ONE CLOSEST TO  THE
     1END OF THE BEAM  ')
 9007 FORMAT(' ','       WEB THICKNESS =',F6.4,' INCHES',
     1          '     ULTIMATE TENSILE STRESS =',F6.2,' KSI'/
     1       ' ','       HORIZONTAL END DISTANCE =',F6.2,' INCHES'/
     1       ' ','       VERTICAL END DISTANCE =',F6.2,' INCHES'/)
 9008 FORMAT('-',20A4/
     1       ' ','    DEFLECTION AT COLUMN FACE =',F7.3)
 9009 FORMAT('0','RADIUS =',F9.3,'  BOLT DEFLECTION =',F7.3/
     1       ' ','HORIZ. DIST. TO FACE OF COLUMN = ',F9.3/
     1       ' ','DIST. TO I.C. =',F9.3)
      END

      SUBROUTINE SIGFIG(C,B,IERR)
      A = C
      IERR =0
      IF(C.LE.0.0) GO TO 99
      J = 0
   10 IF(A - 100.0) 20,50,30
   20 J= J-1
      A = A *10.0
      GO TO 10
   30 IF(A - 1000.0) 50,40,40
   40 J = J+1
      A = A/10.0
      GO TO 30
   50 H = FLOAT(IFIX(A))
      IF(A-H.GT.0.5) H=H+1.0
      IF(A-H-0.5) 70,60,70
   60 IF (FLOAT(IFIX(H/2.0))*2.0.NE.H) H=H+1.0
   70 B = H*10.0 **J
      GO TC 90
   99 IF(C.EQ.0.0) B = 0.0
   90 CONTINUE
      RETURN
      END

      SUBROUTINE OUTLIN(LINE,RNUM,IPOS,MAXD)                            OLN00020
C          MUST CALL S/R SIGFIG BEFORE USE                              OLN00030
C          FORMS A LINE OF OUTPUT WHERE:                                OLN00040
C                                    LINE=ARRAY OF CHAR. ( LE. 131)     OLN00050
C                                    RNUM=NUMBER TO BE ADDED            OLN00060
C                                    IPOS=POSITION IN LINE OF           OLN00070
C                                         DECIMAL POINT                 OLN00080
C                                    MAXD=NUMBER OF FIGURES AFTER DEC.OLN00090
C          IMPERIAL VERSION
      INTEGER LINE(131),INUM(11),ID(12)                                 OLN00100
      DATA ID/'1','2','3','4','5','6','7','8','9','0',' ','.'/          OLN00110
C        SKIP ZERO NUMBERS                                              OLN00120
      IF(RNUM.EQ.0.0) GO TO 50                                          OLN00130
C        CHECK SIZE OF NUMBER                                           OLN00140
      IF(RNUM.GT.10.**7.OR.MAXD.GT.3) GO TO 888                         OLN00150
```

```
      IF(IPOS.LE.0.OR.IPOS.GT.131) GO TO 999                            OLN00160
      INT=RNUM+0.5*10.**(-MAXD)                                         OLN00170
      IF(MAXD.EQ.0) INT=RNUM+0.5                                        OLN00180
      IFRAC=(RNUM*10.**MAXD-FLOAT(INT)*10.**MAXD)+0.5                   OLN00190
C        ALPHA - IZE INT AND IFRAC AND PUT IN INUM                      OLN00200
      DO 5 J=1,11                                                       OLN00210
    5 INUM(J)=ID(11)                                                    OLN00220
      DO 3 JJ=1,7                                                       OLN00230
      IF(INT.LT.(   10**(7-JJ)))GO TO 2                                 OLN00240
      IDIGIT=MOD(INT,10**(8-JJ))/10**(7-JJ)                             OLN00250
      IF(IDIGIT.EQ.0) IDIGIT=10                                         OLN00260
      INUM(JJ)=ID(IDIGIT)                                               OLN00270
      GO TO 3                                                           OLN00280
    2 INUM(JJ)=ID(11)                                                   OLN00290
    3 CONTINUE                                                          OLN00300
      INUM(8)=ID(12)                                                    OLN00310
      IF(IFRAC.LT.0) GO TO 6                                            OLN00320
    7 CONTINUE                                                          OLN00330
      IQ=MAXD                                                           OLN0034
      IQP1=IQ+1                                                         OLN00350
      IF(IQ.EQ.0) GO TO 6                                               OLN00360
      DO 4 JJ=1,IQ                                                      OLN00370
      IDIGIT=MOD(IFRAC,10**(IQP1-JJ))/10**(IQ-JJ)                       OLN00380
      IF(IDIGIT.EQ.0) IDIGIT=10                                         OLN00390
      INUM(JJ+8)=ID(IDIGIT)                                             OLN00400
      IF(MAXD.EQ.JJ) GO TO 6                                            OLN00410
    4 CONTINUE                                                          OLN00420
    6 CONTINUE                                                          OLN00430
C    FIND FIRST NON-ZERO CHARACTER                                      OLN00440
      DO 10 I=1,11                                                      OLN00450
C     DO 10 J=1,12                                                      OLN00460
      IF(INUM(I).NE.ID(11)) GO TO 15                                    OLN00470
   10 CONTINUE                                                          OLN00480
   15 ISIZE=I                                                           OLN00490
C    ISIZE REPRESENTS THE SIZE OF THE NUMBER                            OLN00500
C      FILL IN THE INTEGER PORTION OF THE NUMBER,IF ANY,IN LINE ARRAY   OLN00510
      IF(ISIZE.GE.8) GO TO 20                                           OLN00520
      IF(ISIZE.LE.5) GO TO 21                                           OLN00530
      LINE(IPOS)=INUM(8)                                                OLN00540
   21 K=8                                                               OLN00550
      IT=0                                                              OLN00560
   22 K=K-1                                                             OLN00570
      IF(K.LT.ISIZE) GO TO 20                                           OLN00580
      IP=IPOS-(8-K)-IT                                                  OLN00590
      LINE(IP)=INUM(K)                                                  OLN00600
      IF(K.EQ.5.OR.K.EQ.2) GO TO 23                                     OLN00610
      GO TO 22                                                          OLN00620
   23 CONTINUE                                                          OLN00630
C         PULL NEXT FORTRAN CARD 'GO TO 22'  IF SI METRIC STYLE OUTPUT  OLN00640
C         IS REQUIRED AS BLOCK OF CODE WILL PUT A BLANK BETWEEN 1000 &  OLN00650
C         100 DIGITS AND BETWEEN 1,000,000 & 100,000 DIGITS             OLN00660
      GO TO 22                                                          OLN00670
   24 CONTINUE                                                          OLN00680
      LINE(IP-1)=ID(11)                                                 OLN00690
      IT=IT+1                                                           OLN00700
      GO TO 22                                                          OLN00710
   20 IF(ISIZE.LE.5) GO TO 30                                           OLN00720
      LINE(IPOS)=INUM(8)                                                OLN00730
      LINE(IPOS+1)=INUM(9)                                              OLN00740
      IF(ISIZE.EQ.6) GO TO 30                                           OLN00750
```

```
393          LINE(IPOS+2)=INUM(10)                                        OLN00760
394          IF(ISIZE.EQ.7) GO TO 30                                      OLN00770
395          LINE(IPOS+3)=INUM(11)                                        OLN00780
396       30 CONTINUE                                                     OLN00790
397          IF(MAXD.EQ.0) LINE(IPOS)=ID(11)                              OLN00800
398          RETURN                                                       OLN00810
399      888 WRITE(6,601) RNUM,MAXD                                       OLN00820
400      601 FORMAT(1H0, 10X,F15.4,' OR MAXD ',I4,' EXCEEDS CAPACITY OF S/R OUTOLN00830
            *LIN')                                                        OLN00840
401          RETURN                                                       OLN00850
402      999 WRITE(6,600) RNUM                                            OLN00860
403      600 FORMAT('0',10X,'LINE LENGTH FOR',F11.4,' INCORRECT')         OLN00870
404       50 CONTINUE                                                     OLN00880
405          RETURN                                                       OLN00890
406          END
**WARNING**   UNREFERENCED STATEMENT     24 USED IN LINE     386 FOLLOWS A TRANSFER

        $DATA
```

APPENDIX B

COMPUTER PROGRAM FOR MOMENT MODEL DESIGN PROCEDURE

```
******************************************************************************************
* WATFIV  VERI LEV5.                                              MONITOR,VERSION 3C     *
*                                          H WILSON                                      *
* MONDAY AUGUST 18, 1980.                                                        5:17 PM *
******************************************************************************************

$JOBW ID=' H WILSON ' , T=3
      C
      C
      C 0.1    VARIABLE AND ARRAY SPECIFICATIONS
      C
      C        D=BOLT DIAMETER
      C        T=WEB THICKNESS
      C        QY=YIELD STRENGTH OF BEAM
      C        QU=ULTIMATE TENSILE STRENGTH OF BEAM
      C        N=NUMBER OF ROWS OF BOLTS
      C        G=GAGE
      C        BP=BOLT PITCH
      C        P=PITCH
      C        NLS=NUMBER OF CONNECTIONS TO BE ANALYZED
      C        DEC=DISTANCE FROM CENTERLINE OF RHS BOLT TO OUTSIDE EDGE OF
      C            CLIP ANGLE
      C        WS=WIDTH OF SLOT
      C        LS=LENGTH OF SLOT
      C
               REAL NL1,NL2,L1,L2,MB11,MB12,MB1,MB2,MVS2,MB,LS,MB3
             1      ,NTL1
               DIMENSION M(2),BP(2),JOBID(20)
      C---->CLIP ANGLE ECCENTRICITY OF 2.5 INCHES AND ULTIMATE TENSILE
      C        STRENGTH OF BOLT MATERIAL OF 120.0 KSI ARE GIVEN IN DATA
      C        STATEMENT ;
               DATA DEC/2.5/
      C
      C 0.2    ASSIGN INPUT/OUTPUT UNIT NUMBERS
      C
               IGET=5
               IPUT=6
      C
      C           1. INPUT AND PRINT CONNECTION DATA
      C
               READ (IGET,9104) NLS
          9104 FORMAT(I5)
               WRITE(IPUT,8104) NLS
          8104 FORMAT(' ','  NUMBER OF TEST CASES =',I4)
               DO 102 IJ=1,NLS
               READ(IGET,9103)  JOBID
          9103 FORMAT(20A4)
               WRITE(IPUT,8100) JOBID
          8100 FORMAT('1',20X,'PROGRAM FOR EVALUATION OF ULTIMATE SHEAR WHEN'/
             1        ' ',20X,'TOP FLANGE IS COPED'///
             1        ' ',20A4////
             1        '0','CONNECTION DATA')
      C---->IF BOLT HOLES ARE SLOTTED,PLACE A BLANK CARD AFTER THE DATA
      C        CARD CONTAINING ' JOBID '.
               READ(IGET,9100) D,T,QY,QU,N,G,IFLAG
          9100 FORMAT(4F10.0,I5,F10.0,I5)
      C---->IF IFLAG=0 , THE TOP FLANGE IS COPED.IF IFLAG=1 ,THE TOP FLANGE
      C        IS NOT COPED.
               IF(N.EQ.0) GO TO 120
               WRITE(IPUT,8101) D,    T,DEC,QY,QU,N,G
```

```
      8101 FORMAT('0','DIAMETER OF BOLTS =',F8.4,1X,'INCH'/
     1          ' ','THICKNESS OF WEB =',F8.4,1X,'INCH'/
     1          ' ','CLIP ANGLE ECCENTRICITY =',F8.4,1X,'INCHES'/
     1          ' ','YIELD STRENGTH OF WEB =',F8.3,1X,'KSI'/
     1          ' ','ULTIMATE TENSILE STRENGTH OF WEB =',F8.3,1X,'KSI'/
     1          ' ','NUMBER OF ROWS OF BOLTS =',I4/
     1          ' ','GAGE =',F8.4)
           IS=0
           GO TO 104
       120 CONTINUE
     C     SLOTTED HOLES USED IN CONNECTION
           IS=1
           READ(IGET,9200) D,WS,LS,T,QY,QU,N,G,IFLAG
      9200 FORMAT(6F10.0,I5,F10.0,I5)
     C---->IF IFLAG=0 , THE TOP FLANGE IS COPED.IF IFLAG=1 ,THE TOP FLANGE
     C     IS NOT COPED.
           WRITE(IPUT,8200) D,QUB,WS,LS,T,DEC,QY,QU,N,G
      8200 FORMAT('0','SLOTTED BOLT HOLES'/
     1          ' ','DIAMETER OF BOLTS =',F8.4,1X,'INCH'/
     1          ' ','ULTIMATE TENSILE STRENGTH OF BOLTS =',F8.4,1X,'KSI'/
     1          ' ','WIDTH OF SLOTS =',F8.4,1X,'INCH'/
     1          ' ','LENGTH OF SLOTS =',F8.4,1X,'INCH'/
     1          ' ','THICKNESS OF WEB =',F8.4,1X,'INCH'/
     1          ' ','CLIP ANGLE ECCENTRICITY =',F8.4,1X,'INCHES'/
     1          ' ','YIELD STRENGTH OF WEB =',F8.3,1X,'KSI'/
     1          ' ','ULTIMATE TENSILE STRENGTH OF WEB =',F8.3,1X,'KSI'/
     1          ' ','NUMBER OF ROWS OF BOLTS =',I4/
     1          ' ','GAGE =',F8.4)
       104 CONTINUE
     C---->INPUT HORIZONTAL END DISTANCE MEASURED FROM CENTERLINE OF BOLT TO
     C     EDGE OF WEB,(I.E. 'EH'), AND VERTICAL END DISTANCE MEASURED FROM
     C     CENTERLINE OF BOLT TO COPE,(I.E. 'EV').
           READ(IGET,9101) EH,EV
      9101 FORMAT(2F10.0)
           WRITE(IPUT,8102) EH,EV
      8102 FORMAT('-','HORIZONTAL END DISTANCE =',F8.4,1X,'INCHES'/
     1          ' ','VERTICAL END DISTANCE =',F10.4,1X,'INCHES')
     C---->INPUT NUMBER OF BOLTS PER ROW AND THEIR PITCH
           DO 101 I=1,N
           READ(IGET,9102) M(I),BP(I)
      9102 FORMAT(I5,F10.0)
           WRITE(IPUT,8103) I,M(I),BP(I)
      8103 FORMAT('0','NUMBER OF BOLTS IN ROW',I3,' =',I3,2X,' PITCH =',F8.4)
       101 CONTINUE
     C---->CALCULATE TOTAL VERTICAL LENGTH OF THE MODEL,'L2',
     C     NET VERTICAL LENGTH,'NL2',
     C     TOTAL HORIZONTAL LENGTH,'L1',
     C     NET HORIZONTAL LENGTH,'NL1'.
           IF(IS.EQ.1) GO TO 121
           C1=D+1.0/8.0
           C2=D/2.0+1.0/16.0
           L2=EV+(M(N)-1)*BP(N)
           NL2=L2-M(N)*C1+C2
           L1=EH+(N-1)*G
           NL1=L1-N*C1+C2
     C
     C     CALCULATION OF VERT. AND HORIZ. FORCES ON MODEL
           HS1=0.66*QU*NL1*T
           TS1=QU*L1*T
     C     VS2=V-TS1
```

```
      YC2=(QY*T*L2-HS1)/((1.5*QY*T)+(0.5*QU*T))
      GO TO 103
C
  121 CONTINUE
C     SLOTTED HOLES
      C1=WS+1.0/8.0
      C2=C1/2.0
      C3=LS+1.0/8.0
      L2=EV+(M(N)-1)*BP(N)
      NL2=L2-M(N)*C1+C2
      L1=EH+(N-1)*G
      NL1=L1-C2-C3*(N-1)
      NTL1=NL1+C2*N
C
C     CALCULATION OF VERTICAL AND HORIZONTAL FORCES ON MODEL
      HS1=0.66*QU*NL1*T
      TS1=QU*NTL1*T
C     VS2=V-TS1
      YC2=(QY*T*L2-HS1)/((1.5*QY*T)+(0.5*QU*T))
  103 CONTINUE
C
      IF(IFLAG.EQ.0) GO TO 105
C
C     TOP FLANGE NOT COPED
      HS3=0.66*QY*L1*T
      YC2=(QY*L2*T-HS1+HS3)/((1.5*QY*T)+(0.5*QU*T))
C
  105 CONTINUE
C
C        2. CALCULATION OF RESISTANCE MOMENT ALONG VERTICAL EDGE OF
C    1         MODEL,(I.E.'MB2'),DUE TO YIELDING OF THE WEB
C
C---->INPUT HORIZONTAL ECCENTRICITY,'XC',=DISTANCE FROM END OF CLIP
C     ANGLE TO CENTROID OF BOLT GROUP, AND VERTICAL ECCENTRICITY,
C     'YC',=DISTANCE FROM CENTERLINE OF LOWEST BOLT HOLE TO CENTROID
C     OF BOLT GROUP.
      XC=0.0
      YC=YC2
      WRITE(IPUT,8201) XC,YC
 8201 FORMAT('0','XC =',F8.4,1X,'INCH',5X,'YC =',F8.4,1X,'INCH'////
     1       ' ','SOLUTION')
      P=BP(N)
      MB2=QY*T*(YC)**2.0/2.0+QY*T*(L2-YC)**2.0/2.0
     1   +0.5*(QU-QY)*T*YC*0.6667*YC
C
C        3. CALCULATION OF TOTAL RESISTANCE MOMENT,'MB'.
C
C     CALCULATE MOMENT ALONG HORIZ. SIDE OF MODEL,'MB1'
      MB11=HS1*YC
      IF(IS.EQ.1) GO TO 301
      MB12=QU*T*(L1)**2.0/2.0
      GO TO 302
  301 CONTINUE
C     SLOTTED HOLES
      IF(N.LT.2) GO TO 303
      MB12=(EH)*T*QU*(L1-EH/2.0)
     1     +(N-1)*(G-C3+C2)**2.0*T*QU*0.5
      GO TO 302
  303 CONTINUE
      MB12=(EH)*T*QU*(L1-EH/2.0)
```

```
  302 CONTINUE
      MB1=MB11+MB12
      MB3=0.0
C
      IF(IFLAG.EQ.0) GO TO 304
C
C     TOP FLANGE NOT COPED
C---->CALCULATE RESISTANCE MOMENT DUE TO HORIZONTAL FORCE ALONG
C     SIDE 3 ,I.E. ,' MB3 '.
      MB3=HS3*(L2-YC)
  304 CONTINUE
C
C     CALCULATE TOTAL RESISTANCE MOMENT ='MB'.
      MB=MB1+MB2+MB3
      ARM=G+DEC
C
C        4. CALCULATE ULTIMATE SHEAR RESISTANCE
C
      V=MB/ARM
C---->CHECK THAT VS2 DOES NOT EXCEED THE WEB SHEAR CAPACITY ALONG
C     THE VERTICAL FACE OF THE MODEL
      VS2=V-TS1
C     SHEAR CAPACITY OF MODEL=' VSM2 '
      VSM2=0.66*QU*T*NL2
C
C---->ULTIMATE SHEAR RESISTANCE IS LIMITED BY THE SHEAR CAPACITY
C     OF THE WEB OF THE MODEL
C
C---->CALCULATE MAXIMUM VERTICAL SHEAR WHICH CAN BE TRANSMITTED
C     BY BOLTS ALONG SIDE 2 , I.E. ' VSB2 '
C
C     FASTENER TEAROUT LOAD FOR BOLT NEAREST COPE ,'FTL'
      FTL=2.0*T*(EV-C2)*0.66*QU
C     MAXIMUM BEARING FOR WEB , ' QB ' .
C---->IN ORDER TO LIMIT DEFORMATION OF THE HOLE , THE BEARING
C     RATIO QB/QU SHOULD NOT EXCEED 3.0 .
      QB=3.0*T*D*QU
      IF(FTL.GT.QB) FTL=QB
C     TOTAL MAXIMUM SHEAR , ' VSB2 '
      VSB2=FTL+(M(N)-1)*QB
      IF(N.LT.2) GO TO 405
      VSB2=VSB2+FTL+(M(1)-1)*QB
  405 CONTINUE
      IF(VS2.GT.VSM2) GO TO 404
      IF(V.LT.VSB2) GO TO 401
  404 CONTINUE
C
      WRITE(IPUT,8403) VSM2,TS1,VSB2
 8403 FORMAT('-','ULTIMATE SHEAR RESISTANCE IS LIMITED BY THE WEB '/
     1        ' ','SHEAR CAPACITY OF THE MODEL'//
     1        ' ',3X,'VSM2',5X,'TS1',5X,'VSB2'//
     1        ' ',3F8.2)
C     COMPARE ' VSB2 ' TO ' VSM2 '
      IF(VSB2.LT.(VSM2+TS1)) GO TO 403
      V=VSM2+TS1
      GO TO 402
  403 CONTINUE
      WRITE(IPUT,8404)
 8404 FORMAT('0','BEARING FAILURE LIMITS VERTICAL SHEAR')
      V=VSB2
```

```
          GO TO 402
    401   CONTINUE
          WRITE(IPUT,8301) MB11,MB12,MB1,MB2,MB3,MB,TS1
   8301 FORMAT('-',3X,'MB11',4X,'MB12',4X,'MB1',5X,'MB2',5X
       1            ,'MB3',5X,'MB',6X,'TS1'//
       1            ' ',7F8.2)
    402   CONTINUE
          WRITE(IPUT,8401) V
   8401 FORMAT('-','ULTIMATE SHEAR RESISTANCE =',F8.2)
    102   CONTINUE
          WRITE(IPUT,8402) JOBID
   8402 FORMAT('1','  END OF OUTPUT FOR',20A4)
          STOP
          END

     $DATA
NUMBER OF TEST CASES =     1
```

APPENDIX C

CONTRIBUTION FROM MOMENT CAPACITY OF BEAM FLANGE TO MOMENT MODEL; UNIVERSITY OF TEXAS TESTS

Calculate moment capacity of the flange of the W18x60 used in Test 18-13.

W18x60 b = width of flange = 7.5 in.

t = thickness of flange - 0.684 in.

w = thickness of web = 0.440 in.

k = distance from outer face of flange to web toe of fillet of rolled shape = 1.188 in.

Find plastic section modulus of flange plus portion of the web included in the "k" distance, (see Figure 11). First locate the neutral axis.

$$7.5 \text{ in.}(y) = 0.44 \text{ in.}(1.188 \text{ in.} - 0.684 \text{ in.}) + 7.5 \text{ in } (0.684 \text{ in.} - y)$$

solving for "y" gives: $y = 0.357$ in.

where: y = distance from outer face of flange to neutral axis of section being considered.

Secondly, taking moment of areas about the neutral axis will give:

$$Z = (7.5 \text{ in.} \times 0.357 \text{ in.})\ 0.179 \text{ in.} + (7.5 \text{ in.} \times 0.327 \text{ in.}) \times 0.164 \text{ in} + 0.44 \text{ in.}\ (1.188 \text{ in.} - 0.684 \text{ in.})\ 0.579 \text{ in.}$$

$$Z = 1.01 \text{ in}^3$$

where Z = plastic section modulus

$$M = F_y Z$$

$$M = 38.5 \text{ ksi} \times 1.01 \text{ in}^3 = 38.9 \text{ kips in}$$

$\therefore$ Flange contribution to moment resistance = 38.9 kip in.

Web contribution to moment resistance = 882. kip in. (from "Moment" model, pp. 103)

Flange contribution is only 4.4% of the web contribution and therefore it has been neglected.

PROGRAM FOR EVALUATION OF ULTIMATE SHEAR WHEN
TOP FLANGE IS COPED

TEST NUMBER 18-13 UNCOPED FLANGE

CONNECTION DATA

DIAMETER OF BOLTS = 0.7500 INCH
THICKNESS OF WEB = 0.4390 INCH
CLIP ANGLE ECCENTRICITY = 2.5000 INCHES
YIELD STRENGTH OF WEB = 36.000 KSI
ULTIMATE TENSILE STRENGTH OF WEB = 58.000 KSI
NUMBER OF ROWS OF BOLTS = 2
GAGE = 3.0000

HORIZONTAL END DISTANCE = 1.0000 INCHES
VERTICAL END DISTANCE = 2.3125 INCHES

NUMBER OF BOLTS IN ROW 1 = 3 PITCH = 3.0000

NUMBER OF BOLTS IN ROW 2 = 2 PITCH = 6.0000

XC = 0.0000 INCH YC = 3.5110 INCH

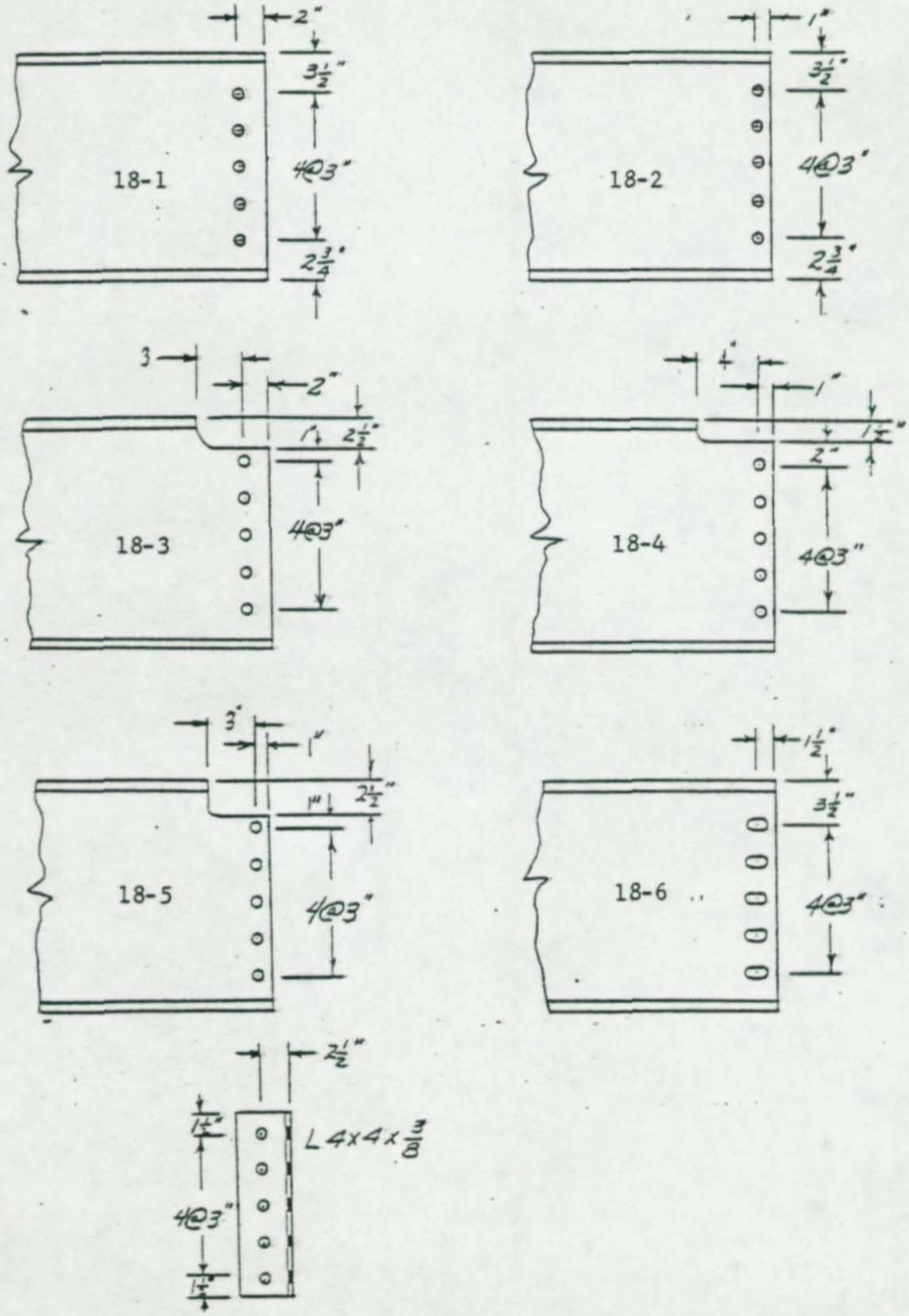
SOLUTION

MB11	MB12	MB1	MB2	MB3	MB	TS1
158.57	203.70	362.26	319.27	200.33	881.87	101.85

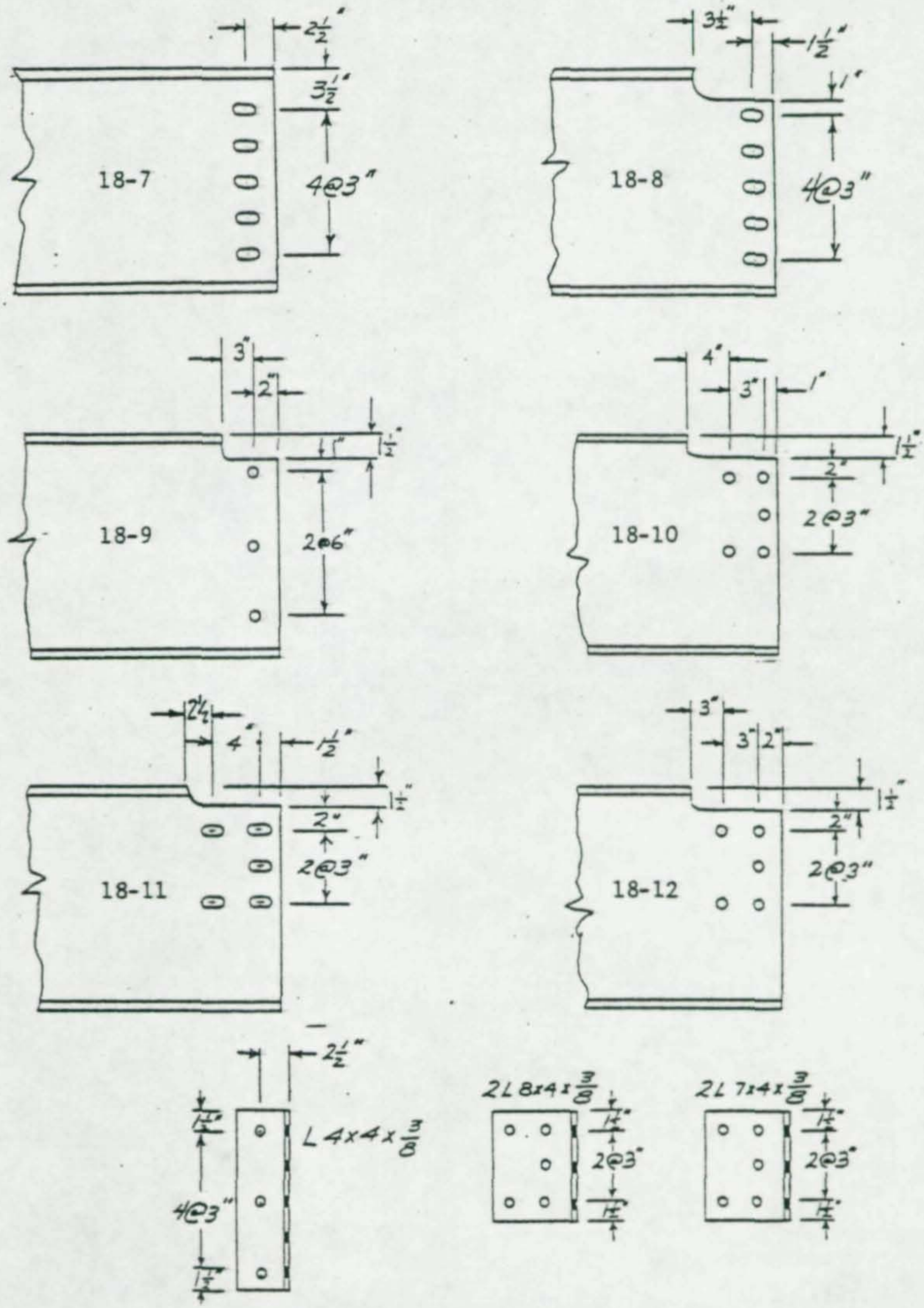
ULTIMATE SHEAR RESISTANCE = 160.34

APPENDIX D

DESCRIPTION OF CONNECTIONS TESTED
AT THE UNIVERSITY OF TEXAS

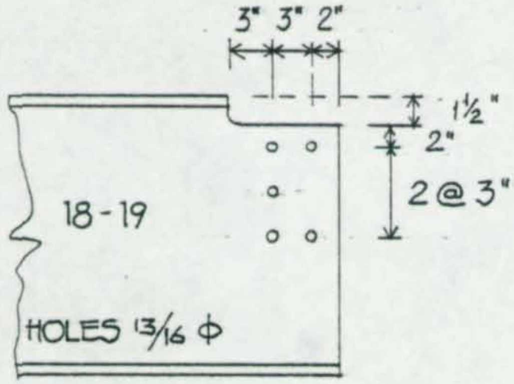


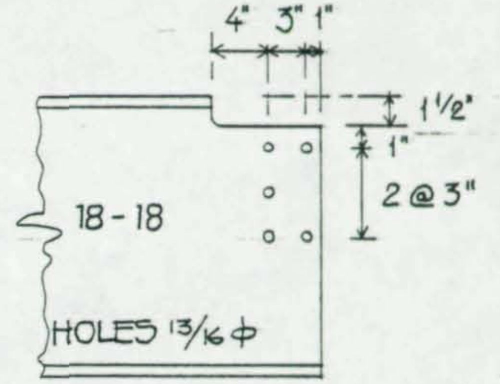
Test Connection Details

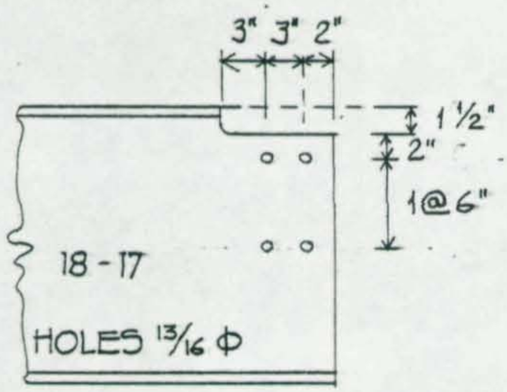

18-7
18-8
18-9
18-10
18-11
18-12
4@3"
2@6"
2@3"
L 4x4x3/8
2L 8x4x3/8
2L 7x4x3/8

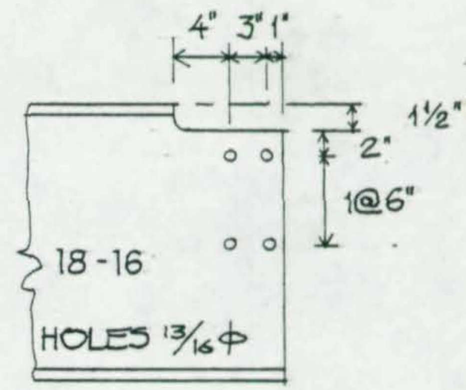
(Continued)

ALL BEAMS
W 18 × 60 × 10'-0" ± 2"

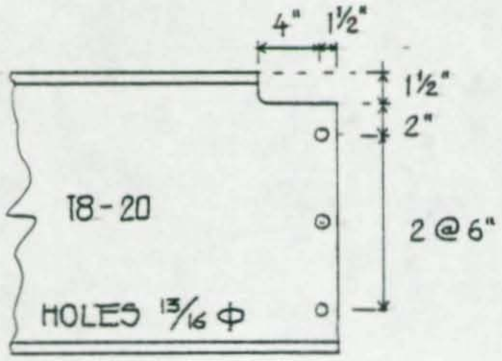


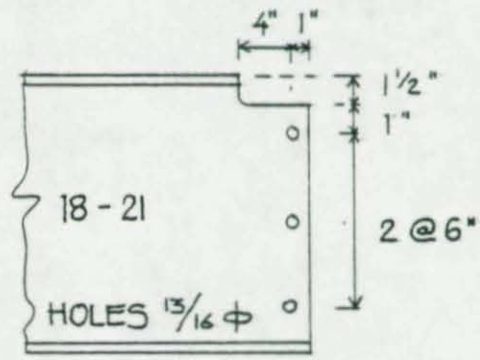


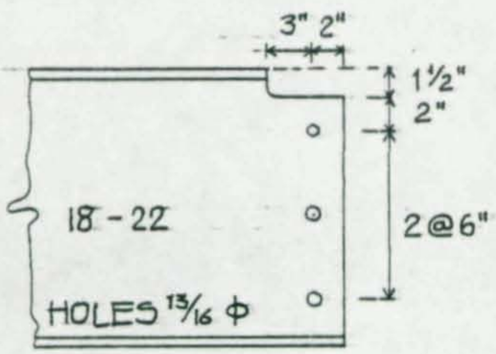


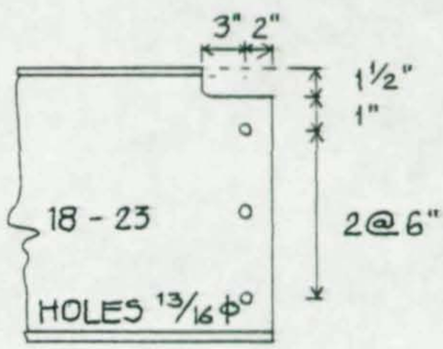


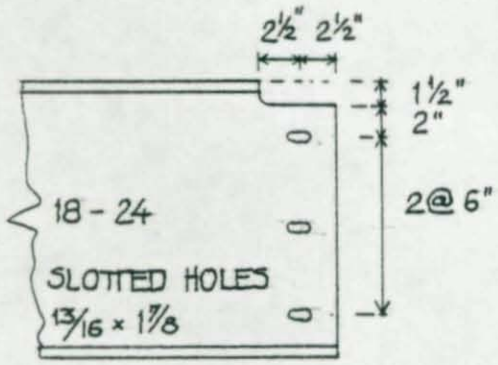
W 18 × 60 × 10'-0" ± 2"

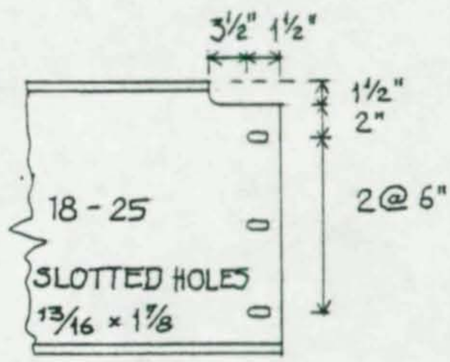

4" 1½"
1½"
2"
18-20
2 @ 6"
HOLES 13/16 Φ


4" 1"
1½"
1"
18 - 21
2 @ 6"
HOLES 13/16 Φ


3" 2"
1½"
2"
18 - 22
2@6"
HOLES 13/16 Φ


3" 2"
1½"
1"
18 - 23
2@ 6"
HOLES 13/16 Φ


2½" 2½"
1½"
2"
18 - 24
2@ 6"
SLOTTED HOLES
13/16 × 1⅞


3½" 1½"
1½"
2"
18 - 25
2@ 6"
SLOTTED HOLES
13/16 × 1⅞