RIGID FRAME STUDIES

Progress Report

DESIGN METHODOLOGY FOR TAPERED BEAMS

Submitted to

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by

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INTRODUCTION

In a previous report (1), differential equations for elastic, singly symmetrical, laterally unbraced, web-tapered members subjected to unequal end moments were derived using the energy approach suggested by Trahair and his research team (2,3,4). The differential equations were solved by the Rayleigh-ritz procedure to find the critical moments. Solutions were verified with a finite element method and by comparison with available closed-form solutions for prismatic members. For design purposes, modifying coefficients for prismatic beam solutions to account for taper and moment gradient were developed using a multiple linear regression technique. The developed simplified procedure was found to be accurate to $\pm$ 8% for tapered beams in single curvature, and $\pm$ 12% for double curvature.

The present report further develops the proposed design methodology. The elastic results are extended to the inelastic range using the CRC formula (5):

$$\sigma_{cr} = \sigma_y - \frac{\sigma_y^2}{4\sigma_e} \qquad (1)$$

where σ_{cr} = inelastic buckling stress, σ_y = yield stress, and σ_e = elastic buckling stress. Ultimate load is defined by either elastic or inelastic lateral-torsional buckling or by first yield when residual stresses are neglected. These are the same criteria as the allowable stress design procedures of the AISC specification (6).

To facilitate computational efficiency a small computer program was developed and is described. Manual solutions for a wide range of problems are presented for verification of the computer program.

DESIGN METHODOLOGY FOR TAPERED BEAMS

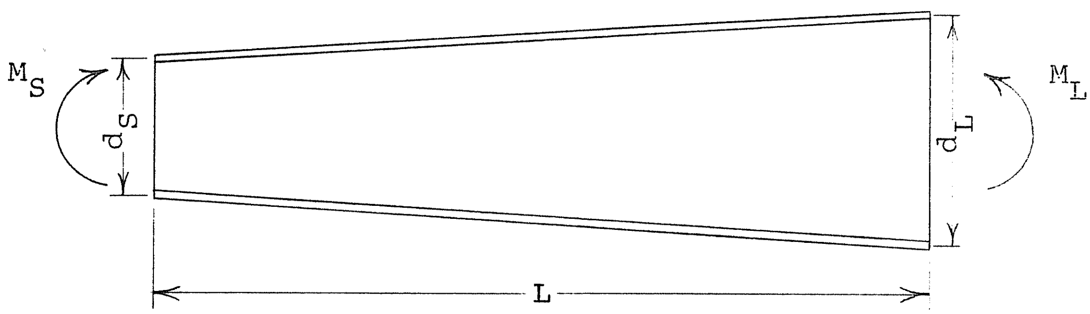
The design methodology used for web-tapered beams subjected to end moments, Figure 1a, is based on solutions for a prismatic beam with the same cross-section as at the small end of the tapered beams and subjected to equal but opposite end moments as shown in Figure 1b. This beam and loading condition is referred to as the "basic case" prismatic beam. For singly symmetrical cross-sections loaded in the plane of symmetry, the elastic critical moment for the basic case is

$$M_e = \pm \frac{\pi^2 E I_y \beta_x}{2L^2} \left\{ 1 + \sqrt{1 + \frac{4}{\beta_x^{\,2}} \left(\frac{C_w}{I_y} + \frac{GJL^2}{\pi^2 E I_y} \right)} \right\} \qquad (2a)$$

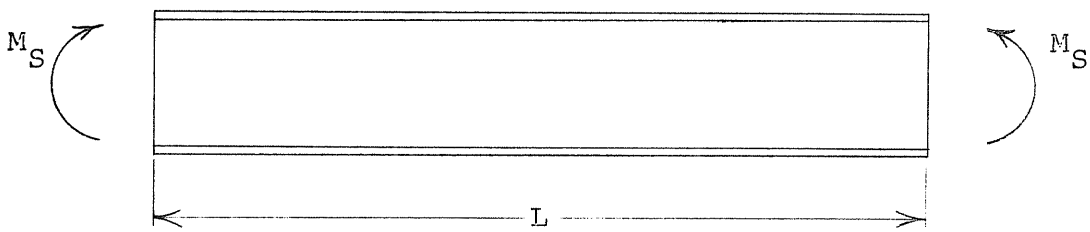
where I_y = weak axis moment of inertia, β_x = a cross-sectional property to be defined later, L = unbraced length of the tapered beam, C_w = warping moment of inertia, J = torsion constant, E = modulus of elasticity, and G = shear modulus of elasticity. The positive root is taken when the large flange is in compression; the negative root is taken when the small flange is in compression.

For doubly symmetrical sections β_x is zero and equation 2a reduces to

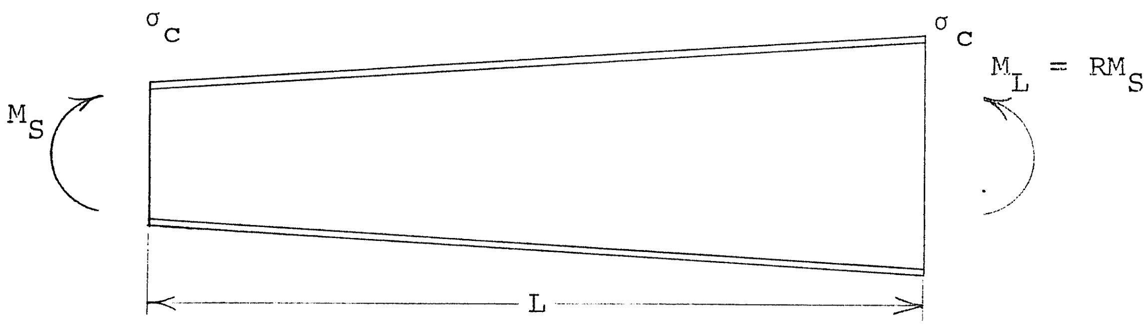
$$M_e = \pm \frac{\pi^2}{2L^2} \sqrt{E I_y \, GJ + \frac{E^2 I_y C_w \pi^2}{L^2}} \qquad (2b)$$



a) Tapered Beam Subjected to End Moments



b) Basic Case Prismatic Beam



c) Basic Case Tapered Beam

Figure 1. Beam Geometry and Loading

The tapered beam basic case loading condition is defined as equal compressive stress at the exterior flange tips when the beam is bent in single curvature as shown in Figure 1c. This loading condition produces approximately uniform compressive flange stress along the member. The relationship between end moments is

$$M_S = R\ M_L \qquad (3)$$

where M_S = moment at small end, M_L = moment at large end, and R = the ratio of the section moduli to the extreme fiber of the compression flange at the large end to that at the small end.

The critical moment for the tapered beam, basic case loading, is estimated from

$$(M_e)_S = C_a\ M_e \qquad (4)$$

where M_e is determined from equations 2a or 2b, (M_e) = the critical moment at the small end of the tapered beam and C_a = a coefficient to account for the effects of taper. The critical moment at the large end for the basic case loading is

$$(M_e)_L = R\ (M_e)_S = R\ C_a\ M_e \qquad (5)$$

For the case of varying flange stress along the member, Figure 1a, equations 4 and 5 are modified to

$$(M_e)_S = C_b\ C_a\ M_e \qquad (6a)$$

$$(M_e)_L = C_b\ R\ C_a\ M_e \qquad (6b)$$

where C_b is a modifying factor to account for stress variation due to unequal compressive flange stresses at the beam ends.

The multiplying factors C_a and C_b were determined using multiple linear regression analysis of theoretically correct values of critical moments for tapered beams (1). The best-fit equation for C_a is

$$C_a = 1.0 - 1.4580\ \alpha \sqrt{\frac{GJL^2}{EC_w} + 44.6328\ \alpha\ \frac{J}{I_y}} \tag{7}$$

Here

$$\alpha = \frac{d_L - d_S}{L}$$

where d = beam depth.

The multiplying factor C_b depends on the relative stresses in a flange at each end of the beam. Defining "r" as the stress ratio such that

$$-1.0 \leq r = \frac{\sigma_{opp}}{\sigma_{max}} \leq 1.0$$

where σ_{max} is the maximum compressive flange stress at a beam end and σ_{opp} is the stress in the <u>same</u> flange at the opposite end of the beam. A stress ratio of +1.0 is equivalent to the tapered beam basic case loading, e.g. single curvature, and a ratio of -1.0 is equivalent to double curvature bending with stresses in the reference flange equal in magnitude but opposite in sense at opposite ends. It was not possible to find a single best-fit equation for C_b over the entire stress ratio range For stress ratios greater than -0.40, C_b is given by

$$C_b = 1.0 - 0.3867(r-1.0) + 0.4739(r-1.0)^2 + 0.9074\frac{\alpha L}{d}(r-1)^2 \qquad (8)$$

Best-fit equations for the range $-0.40 > r \geq -1.0$ predicted unacceptable unconservative results. An acceptable design approach for this range is to compute C_b for $r = -0.40$ from equation 8 and C_b for $r = -1.0$ from

$$C_b = 2.7596 - 2.8152\frac{\beta_x}{d} - 0.6562\frac{\alpha L}{d} - 15.6530\,\alpha\frac{\beta_x}{d} \qquad (9a)$$

for <u>small</u> end reference and

$$C_b = 2.7684 + 1.2025\frac{\alpha L}{d} - 2.2686\frac{\beta_x}{d} - 22.0724\,\alpha\frac{\beta_x}{d} \qquad (9b)$$

for <u>large</u> end reference and use linear interpolation between the two values to determine C_b for the given r.

If the resulting value of M_e is greater than $0.5M_y$ for the referenced flange the CRC formula, equation 1, is to be used to estimate the inelastic critical moment, M_{cr}. A check must also be made to determine if the tension flange yields.

The equations for C_a and C_b are applicable only to cross-sections, taper ratios and unbraced lengths normally found in rigid frames of pre-engineered metal buildings and the following limitations must be followed:

$1.5 \leq \frac{\text{minimum depth}}{\text{flange width}} \leq 3.0$ | 6 in. $\leq d_L \leq$ 24 in.

$24 \geq \frac{\text{span length}}{\text{minimum depth}}$ | 3 in. $\leq$ flange width $\leq$ 12 in.

$15 \leq \frac{\text{flange width}}{\text{flange thickness}} \leq 50$ | 3/16 in. $\leq$ flange thickness $\leq$ 3/4 in.

In addition, to limit the unconservative error to less than 8% for beams in single curvature and to less than 12% for beams in double curvature, both at the 95% confidence level, in the range $-0.4 \geq r \geq 1.0$, the following limitations must be observed

$$\frac{I_{yL}}{I_{yS}} \left(\frac{\alpha L}{d} \right) \leq 1.30 \qquad\qquad \frac{I_{yL}}{I_{yS}} \leq 2.5$$

It is noted that the error of estimation is reduced as the taper decreases and as the cross-section becomes more nearly doubly symmetric.

The design procedure is summarized as follows:

(1) For a given member and end moments, calculate the extreme fiber compressive stresses at the beam ends. The flange location with the largest stress is chosen as the reference end flange

(2) Calculate the basic case critical moment for a prismatic beam with the same cross-section as the small end of the beam in question and with the flange corresponding to the reference flange in compression, using either equation 2a or 2b.

(3) Calculate the modifying factor C_a with reference to the small end section properties, using equation 7.

(4) Calculate the stress ratio, r, as the ratio of the beam end stresses in the reference flange. Note that r is greater than zero for single curvature bending and less than zero for double curvature

bending.

(5) If r is greater than -0.40, use equation 8 and the referenced end properties to calculate C_b.

(6) If r is less than -0.40, calculate C_b for r = 0.40 using equation 8 and C_b for r = -1.0 using equation 9a for small end reference and 9b for large end reference. Use linear interpolation between these two value to determine C_b for the stress ratio in question.

(7) Calculate R, the ratio of section moduli to extreme fibers of the reference flange at the beam ends.

(8) Calculate the tapered beam critical moments using equation 6a for small end reference and equation 6b for large end reference.

(9) Determine if the critical moment exceeds one-half of the first yield moment of the referenced flange. If this moment is exceeded, estimate the inelastic critical moment using equation 1.

(10) Determine the critical moment at the opposite end using equation 3.

(11) Check that the yield stress of the material is not exceeded in the tension flanges. If it is, reduce the moment to a value which causes first yield in the tension flange and use equation 3 to determine the moment at the opposite end.

(12) Apply a factor of safety, commonly 1.67, to obtain the working load moments.

A flow chart of the procedure is shown in Figure 1 in terms of stresses rather than moments. A listing of a FORTRAN IV computer program for the design procedure is found in the appendix to this report. The sign convention for moments in this program is clockwise moments are positive.

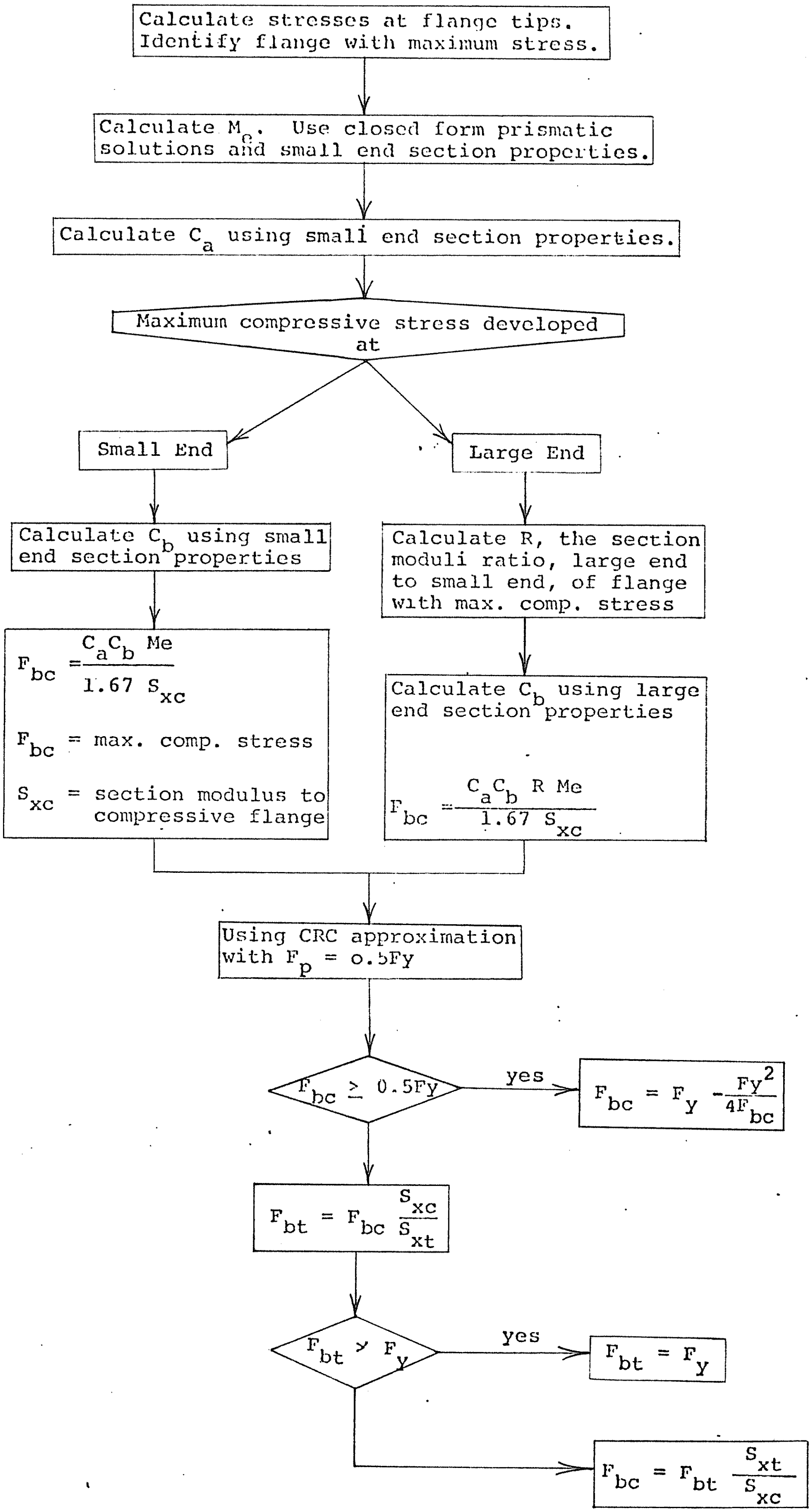
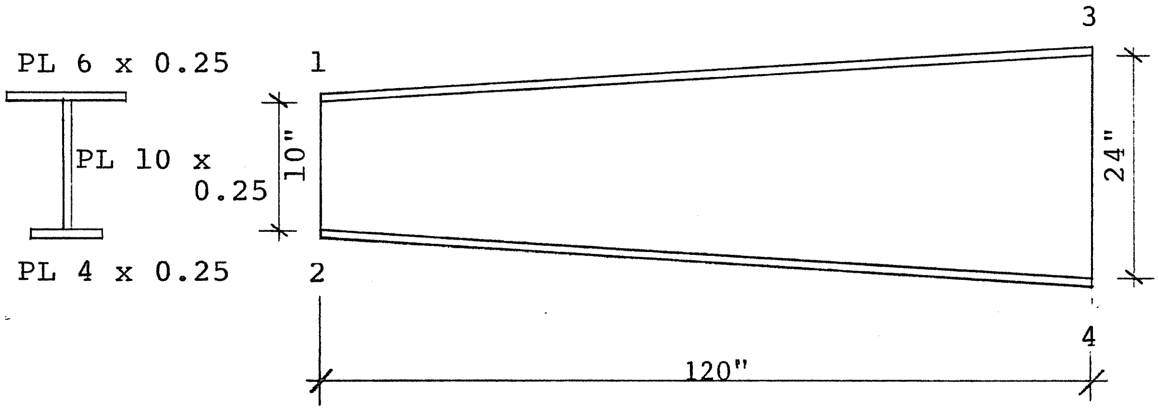


Figure 2. Flow Chart for Design Procedure

EXAMPLE CALCULATIONS

The singly symmetrical, laterally unbraced, tapered beam shown in Figure 3 was analyzed manually to determine the critical moments and stresses for the four loadings shown in Figure 4. The results from the computer program are found in the Appendix and are identical to the manual calculations.

In the calculations that follow, flange tips are referenced as shown in Figures 3 and 4. For each loading case the stresses at the four flange extremities are first calculated, and the reference end and reference flange determined. Next the prismatic elastic buckling moment is found and the modifying factors to account for taper and stress gradient determined. The elastic buckling stress for the critical location is then calculated and compared to the assumed proportional limit (0.5 F_y). If the elastic buckling stress exceeds the proportional limit, the inelastic buckling stress is estimated using the CRC formula. Once the critical stress is found, the stress in the tension flange is computed and compared to the yield stress. If the yield stress is exceeded, the corresponding moments at each end are computed. Finally, stresses caused by the final moments are calculated.

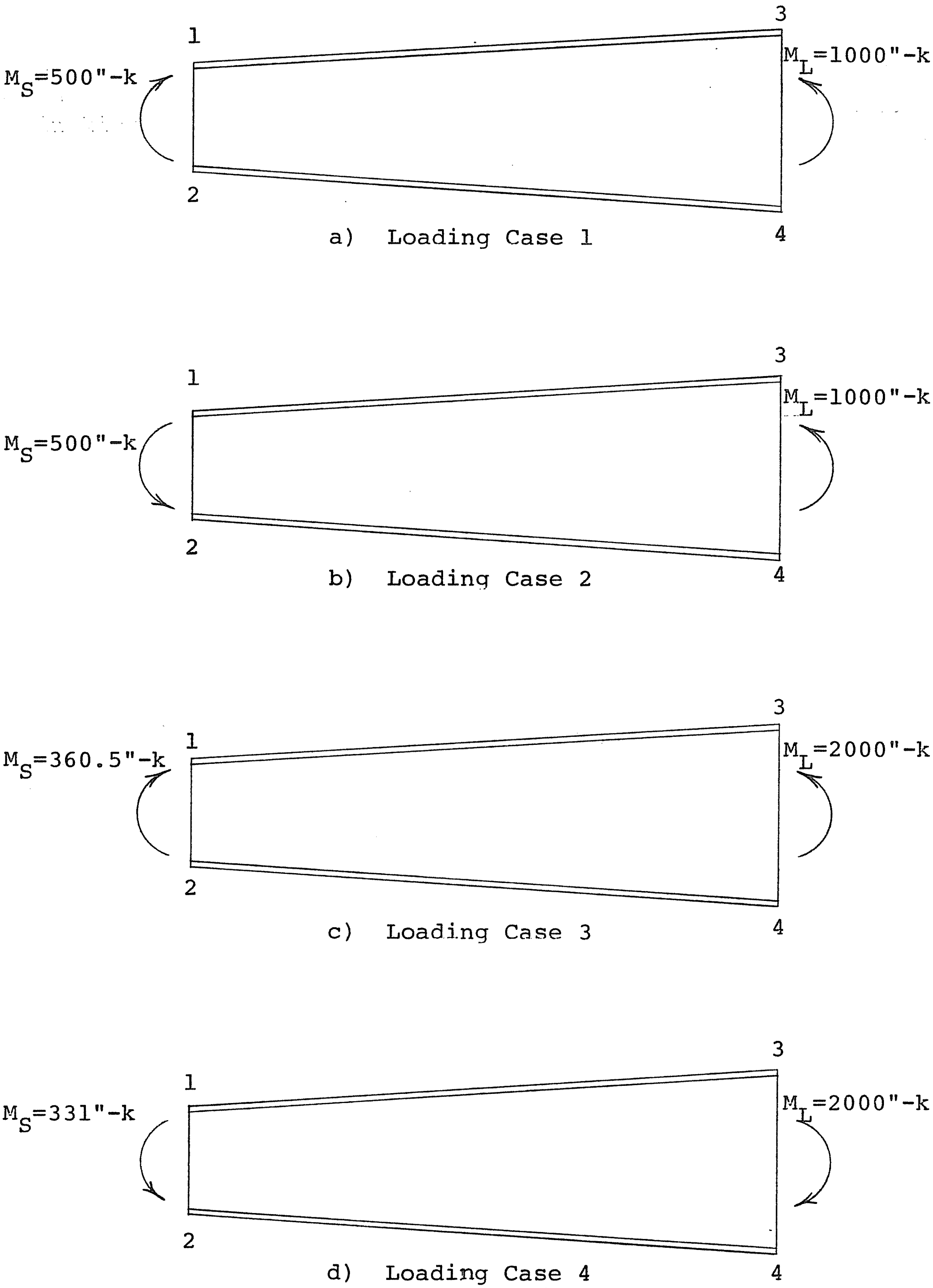


$\alpha = \frac{24 - 10}{120} = 0.1167$ E = 30,000 ksi

F_y = 50.0 ksi G = 11,200 ksi

Property	Section Property at Small End	Section Property at Large End
B_2	6.0 in.	6.0 in.
T_2	0.25 in.	0.25 in.
B_3	10.0 in.	24.0 in.
T_3	0.125 in.	0.125 in.
B_4	4.0 in.	4.0 in.
T_4	0.25 in.	0.25 in.
$\bar{y}$	4.33 in.	10.91 in.
A	3.75 $in.^2$	5.5 $in.^2$
I_x	71.26 $in.^6$	497.5 $in.^4$
I_y	5.83 $in.^4$	5.83 $in.^4$
C_w	102.85 $in.^6$	592.46 $in.^6$
J	0.0586 $in.^6$	0.0677 $in.^4$
β_x	5.06 in.	12.33 in.
S_x	16.44 $in.^3$	45.6 $in.^3$
S_x	12.57 $in.^3$	38.0 $in.^3$

Figure 3. Beam Dimensions and Properties for Example Calculations



a) Loading Case 1

b) Loading Case 2

c) Loading Case 3

d) Loading Case 4

Figure 4. Loading Cases for Example Calculations

LOADING CASE 1

Stresses

$F_1 = M_S/S_1 = 500/16.44 = 30.40$ ksi C

$F_2 = M_S/S_2 = 500/12.57 = 39.78$ ksi T

$F_3 = M_L/S_3 = 1000/45.60 = 21.93$ ksi C

$F_4 = M_L/S_4 = 1000/38.00 = 26.31$ ksi T

Note maximum compressive stress is at 1, therefore, reference end is small end and reference flange is top flange, hence

r = stress ratio for top flange

$= F_3/F_1 = 21.93/30.4 = 0.721$

R = moduli ratio for top flange

$= S_3/S_1 = 45.6/16/44 = 2.77$

M_e

Basic case moment from equation 2, positive signs.

$$M_e = \frac{\pi^2(30000)(5.06)}{2(120)^2}\{1 + \sqrt{1 + (\frac{4}{5.06})^2 (\frac{102.86}{5.83} + \frac{11200(0.0586)(120)^2}{\pi^2(30000)(5.83)})}\}$$

$$= 303.28\{1 + \sqrt{1 + 3.61}\}$$

$$= 954.48 \text{ in-kip}$$

C_a

Using small end properties

$$C_a = 1 - 1.458(0.1167)\sqrt{\frac{(11200)(0.0586)(120)^2}{(30.000)(102.86)} + \frac{44.63(0.1167)(0.0586)}{5.83}}$$

$$= 0.754$$

$\underline{C_b}$

Since r > -0.40

$$C_b = 1-0.3867(0.721 -1)+0.4739(0.721-1)^2 + \frac{0.9074 (0.1167)(120)(0.721-1)^2}{10.0}$$

$$= 1.24$$

$\underline{F_{bc}}$

Since node 1 is critical

$$F_{bc} = C_a C_b M_e/S_1 = 0.754(1.24)(954.48)/16.44$$

$$= 54.3 \text{ ksi} > F_y/2 = 25 \text{ ksi}$$

Inelastic buckling, CRC Formula

$$F_{bc} = 50.0 - \frac{50.0^2}{4(54.3)} = 38.5 \text{ ksi C}$$

$$F_{bt} = F_{bc} S_1/S_2 = (38.5)(16.44)/12.57$$

$$= 50.4 \text{ ksi T} > F_y = 50.0 \text{ ksi}$$

Tension controls, thus

$$F_{bt} = 50.0 \text{ ksi T}$$

$$F_{bc} = F_y S_2/S_1 = 50.0(12.57)/16.44 = 38.2 \text{ ksi C}$$

<u>Critical Final Moments</u>

$M_S = F_{bc} S_1 = 38.2(16.44) =$ <u>62.85 in-kips > 500 in</u>-kips

$M_L = M_S rR = 628.5(0.721)(2.77)$

= <u>1255 in-kips</u> >1000 in-kips

Beam is satisfactory for loading

Critical Stresses

$F_1 = 38.2$ ksi C

$F_2 = 50.0$ ksi T

$F_3 = 1255/45.6 = 27.5$ ksi C

$F_4 = 1255/38.0 = 33.0$ ksi T $< F_y = 50.0$ ksi

LOADING CASE 2

Stresses

$F_1 = M_S/S_1 = 500/16.44 = 30.40$ ksi T

$F_2 = M_S/S_2 = 500/12.57 = 39.78$ ksi C

$F_3 = M_L/S_3 = 1000/45.60 = 21.93$ ksi C

$F_4 = M_L/S_4 = 1000/38.00 = 26.31$ ksi T

Note maximum compressive stress is at 2. Therefore, reference end is small end and reference flange is bottom flange, hence

r = stress ratio for bottom flange

$= F_4/F_2 = +26.31/-39.78 = -0.66$

R = moduli ratio for bottom flange

$= S_4/S_2 = 38.00/12.57 = 3.02$

M_e

Basic case moment from equation 2, negative signs

$M_e = 303.28\ (1-\sqrt{1+3.61})$

$= 347.89$ in-kips

C_a

$C_a = 0.754$, same as for loading case 1.

C_b

Since $r < -0.40$

C_{b1} for $r = -0.40$

$= 1-0.3867(-0.4-1.0) + 0.4739(-0.4-1.0)^2$

$+ \dfrac{0.9074(0.1167)(120)(-0.6-1.0)^2}{10}$

$= +4.96$

C_{b2} for r = -1.0

$$= 2.7596 - \frac{2.8152(5.06)}{10} - \frac{0.565(0.1167)(120)}{10}$$

$$- \frac{15.653(0.1167)(5.06)}{10}$$

$= -0.508$

Using linear interpolation,

$$C_b = -0.508 + \frac{4.96 + 0.0508)(1.-0.66)}{0.6}$$

$= +2.59$

<u>F_{bc}</u>

Since node 2 is critical

$F_{bc} = C_a C_b \, M_e/S_2 = 0.754(2.59)(347.89)/12.57$

$= 54.0$ ksi $> F_y/2 = 25$ ksi

Inelastic buckling, CRC Formula

$$F_{bc} = 50.0 - \frac{50.0^2}{4(54.0)} = 38.4 \text{ ksi C}$$

$F_{bt} = F_{bc} \, S_2/S_1 = 38.4\ (12.57)/16.44$

$= 29.36$ ksi T $< F_y = 50.0$ ksi

<u>Critical Moments</u>

$M_S = F_{bc} \, S_2 = 38.4(12.57) =$ <u>482.7 in-kips</u> <500 in-kips

$M_L = M_S \, rR = 482.7(0.66)(3.02)$

= <u>962.1 in-kips</u> <1000 in-kips

Beam is not satisfactory for loading.

<u>Critical Stresses</u>

$F_1 = 29.36$ ksi T

$F_2 = 38.4$ ksi C

$F_3 = 962.1/45.6 = 21.1$ ksi C

$F_4 = 962.1/38.0 = 25.3$ ksi T

LOADING CASE 3

Stresses

$F_1 = M_S/S_1 = 360.5/16.44 = 21.91$ ksi C

$F_2 = M_S/S_2 = 500.0/12.57 = 28.68$ ksi T

$F_3 = M_L/S_3 = 2000.0/45.6 = 43.86$ ksi C

$F_4 = M_L/S_4 = 2000.0/38.0 = 52.63$ ksi T

Note maximum compressive stress is at 3. Therefore reference end is large and reference flange is top flange, hence

r = stress ratio for top flange

$= F_1/F_3 = -21.82/-43.86 = +0.50$

R = moduli ratio for top flange

$= S_3/S_1 = 45.6/16.44 = 2.77$

M_e

Basic case moment from equation 2, positive signs

$M_e = 954.48$ in-kips, same as for loading case 1.

C_a $C_a = 0.754$. Same as for loading case 1.

C_b Since $r > -0.40$

$$C_b = 1 - 0.386(0.5-1.0) + 0.4739(0.5-1.0)^2 + \frac{0.9074(0.1167)(120)(0.5-1.0)^2}{24}$$

$= +1.44$

F_{bc}

Since node 3 is critical

$$F_{bc} = C_a C_b M_e R/S_3$$

$= 0.754(1.44)(954.48)(2.77)/45.6$

$= 62.95 \text{ ksi} > \frac{F_y}{2} = 25 \text{ ksi}$

Inelastic buckling, CRC Formula

$$F_{bc} = 50.0 - \frac{50.0^2}{4(62.95)} = 40.1 \text{ ksi C}$$

$$F_{bt} = F_{bc}\frac{S_3}{S_4} = 40.1\ (45.6)/38.0$$

$$= 48.12 \text{ ksi T} < F_y = 50 \text{ ksi}$$

<u>Critical Moments</u>

$$M_L = F_{bc}\ S_3 = 40.1\ (45.6) = \underline{1828.6 \text{ in-kips}} < 2000 \text{ in-kips}$$

$$M_S = M_1\ r/_R = 1828.6(0.5)/2.77$$

$$= \underline{330 \text{ in-kips}} < 360.5 \text{ in-kips}$$

Beam is not satisfactory for loading

<u>Critical Stresses</u>

$F_1 = 330/16.44 = 20.1$ ksi C

$F_2 = 330/12.57 = 26.2$ ksi T

$F_3 = 40.1$ ksi C

$F_4 = 48.12$ ksi T

LOADING CASE 4

Stresses

$F_1 = M_s/S_1 = 331/16.44 = 20.13$ ksi T

$F_2 = M_s/S_2 = 331/12.57 = 26.33$ ksi C

$F_3 = M_2/S_3 = 2000/45.6 = 43.86$ ksi T

$F_4 = M_2/S_4 = 2000/38.0 = 52.63$ ksi C

Note maximum compressive stress is at 4. Therefore, reference end is large end and reference flange is bottom flange, hence

r = stress ratio for bottom flange

$= F_2/F_4 = -26.33-52.63 = +0.50$

R = moduli ratio for bottom flange

$= S_4/S_2 = 38.0/12.57 = 3.02$

M_e

Basic case moment from equation 2, negative signs

$M_e = 347.89$ in-kips, same as for loading case 2

C_a

$C_a = 0.754$, same as for loading case 1 and 2.

C_b

Since r >-0.40

$C_b = 1-0.3867(0.5-1.0)+0.4739(0.5-1.0)^2$

$+ 0.9074(0.1167)(120)(0.5-1.0)^2/24$

$= +1.44$

F_{bc}

Since node 4 is critical,

$F_{bc} = C_aC_bM_eR/S_4 = 0.754(1.44)(347.84)(3.02)/38$

$= 30 \text{ ksi} > \frac{F_y}{2} = 25 \text{ ksi}$

Inelastic buckling, CRC Formula

$$F_{bc} = 50.0 - \frac{50.0^2}{4(30)} = 29.2 \text{ ksi C}$$

$$F_{bt} = S_4/S_3 = 29.2\ (38.0)/45.6$$

$$= 24.3 \text{ ksi T} < F_y = 50 \text{ ksi}$$

Critical Moments

$M_L = F_{bc}\ S_4 = 29.2\ (38.0) =$ 1109.6 in-kips < 2000 in-kips

$$M_S = M_L\ r/R = 1109.6(0.5)/3.02$$

= 183.7 in-kips < 331 in-kips

Beam is not satisfactory for loading

Critical Stresses

$$F_1 = 183.7/16.44 = 11.2 \text{ ksi T}$$

$$F_2 = 183.7/12.57 = 14.6 \text{ ksi C}$$

$$F_3 = 24.3 \text{ ksi T}$$

$$F_4 = 29.2 \text{ ksi C}$$

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APPENDIX

COMPUTER PROGRAM AND EXAMPLE OUTPUT

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C
C***********************************************************************
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C**                                                                  **
C**                              LTBTB                               **
C**                                                                  **
C**                   EXAMPLE DESIGN METHODOLOGY                     **
C**                                                                  **
C**                        FOR TAPERED BEAMS                         **
C**                                                                  **
C**                              B2                                  **
C**                         ============                             **
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C**                                                                  **
C**                        INPUT INSTRUCTIONS                        **
C**                                                                  **
C**           CARD ONE                                               **
C**                                                                  **
C**                FULL EIGHTY COLUMNS AVAILABLE FOR LOAD AND GEOMETRY **
C**                IDENTIFICATION. INFORMATION NOT USED IN PROGRAM.  **
C**                                                                  **
C**           CARD TWO                                               **
C**                                                                  **
C**                IN 8F10 FORMAT, ENTER BEAM CROSS-SECTION DATA.    **
C**                                                                  **
C**                B1  TOP FLANGE LIP LENGTH                         **
C**                T1  TOP FLANGE LIP THICKNESS                      **
C**                B2  TOP FLANGE LENGTH                             **
C**                T2  TOP FLANGE THICKNESS                          **
C**                B3  WEB LENGTH                                    **
C**                T3  WEB THICKNESS                                 **
C**                B4  BOTTOM FLANGE LENGTH                          **
C**                T4  BOTTOM FLANGE THICKNESS                       **
C**                                                                  **
C**           CARD THREE                                             **
C**                                                                  **
C**                IN 3F10 FORMAT,ENTER BEAM GEOMETRY AND YIELD STRENGTH **
C**                                                                  **
C**                ALPHA   TAPER                                     **
C**                XL      LENGTH                                    **
C**                FY      YIELD STRENGTH OF STEEL                   **
C**                                                                  **
C**           CARD FOUR                                              **
C**                                                                  **
C**                IN 2F10 FORMAT, ENTER END MOMENTS                 **
```

```
               C**                                                              **
               C**                 XMS  SMALL END MOMENT                        **
               C**                 XML  LARGE END MOMENT                        **
               C**                 CLOCKWISE MOMENT POSITIVE
               C**
               C**            LAST CARD                                         **
               C**                                                              **
               C**                 "END OF DATA"   STARTING IN COLUMN 2.        **
               C**                                                              **
               C*****************************************************************
               C*****************************************************************
               C
               C
               C
               C         DATA INPUT
               C
0001                 DIMENSION S(6),F(4)
0002                 REAL MCRS,MCRL
0003             999 READ(5,2000,END=55)
0004            2000 FORMAT(1X,79H
                    1
0005                 WRITE(6,2001)
0006            2001 FORMAT(1H1)
0007                 WRITE(6,2000)
0008                 READ(5,101,END=55)B1,T1,B2,T2,B3,T3,B4,T4
0009             101 FORMAT(8F10.4)
0010              25 READ(5,125)ALPHA,XL,FY
0011             125 FORMAT(3F10.4)
               C
               C     INPUT OF MOMENT DATA
               C
0012                 READ(5,100)XMS,XML
0013             100 FORMAT(2F10.3)
               C
0014                 E=30000.
0015                 G=11200.
0016                 PI=3.1415927
               C
               C        SECTION PROPERTY CALCULATIONS
               C
0017                 BR=B3+ALPHA*XL
0018                 AREA=B1*T1+B2*T2+B3*T3+B4*T4+B1*T1
0019                 AREAP=AREA+ALPHA*XL*T3
0020                 YBAL=(2.*B1*T1*(B3-B1/2.)+B2*T2*B3+B3*B3*T3/2.)/AREA
0021                 YBAR=(2.*B1*T1*(BR-B1/2.)+B2*T2*BR+BR*BR*T3/2.)/AREAR
0022                 YTOL=B3-YBAL
0023                 YTOR=BR-YBAR
0024                 XIL=(T1*B1**3*2.+B2*T2**3+T3*B3**3+B4*T4**3)/12.+2.*T1*B1*(YTOL-B1
                    1/2.)**2+B2*T2*YTOL*YTOL+B3*T3*(B3/2.-YTOL)**2+B4*T4*YBAL*YBAL
0025                 XIR=(T1*B1**3*2.+B2*T2**3+T3*BR**3+B4*T4**3)/12.+2.*T1*B1*(YTOR-B1
                    1/2.)**2+B2*T2*YTOR*YTOR+BR*T3*(BR/2.-YTOR)**2+B4*T4*YBAR*YBAR
0026                 TJL=(2.*B1*T1**3+B2*T2**3+B3*T3**3+B4*T4**3)/3.
0027                 TJR=(2.*B1*T1**3+B2*T2**3+BR*T3**3+B4*T4**3)/3.
0028                 YTL=2.*B1*T1*(B2/2.)**2+T2*B2**3/12.
0029                 YBL=T4*B4**3/12.
0030                 YI=YTL+YBL
0031                 HL=(3.*B1*B1*B2*B2*T1-T4*B3*B4**3)/12./YI
0032                 HR=(3.*B1*B1*B2*B2*T1-T4*BR*B4**3)/12./YI
```

```
0033            YOL=B3+HL-YBAL
0034            YOR=BR+HR-YBAR
0035            E0=B2*B2*B1*B1*T1/4./YTL
0036            WIL=YTL*YBL*(EX+B3)**2/YI
0037            WIR=YTL*YBL*(EX+BR)**2/YI
0038            ROL=(XIL+YI)/AREA+YOL**2
0039            ROR=(XIR+YI)/AREAR+YOR**2
0040            SL=XIL/YTOL
0041            SR=XIR/YTOR
0042            SBL=XIL/YBAL
0043            SBR=XIR/YBAR
0044            S(1)=SL
0045            S(2)=SBL
0046            S(3)=SR
0047            S(4)=SBR
0048            GAM=YTL*YBL/YI
0049            BXL=(YBAL*(T4*B4**3/12.+B4*T4*YBAL*YBAL)-YTOL*(T2*B2**3/12.+B2*T2*
               1YTOL*YTOL)-B2*B2*B1*T1*(YTCL/2.-B1/4.)-2.*B1*T1*(YTOL**3-1.5*B1*YT
               2OL*YTOL+B1*B1*YTOL-B1*B1*B1/4.)+YBAL**4*T3/4.-YTOL**4*T3/4.)/XIL+2
               3.*YOL
0050            BXR=(YBAR*(T4*B4**3/12.+B4*T4*YBAR*YBAR)-YTOR*(T2*B2**3/12.+B2*T2*
               1YTOR*YTOR)-B2*B2*B1*T1*(YTOR/2.-B1/4.)-2.*B1*T1*(YTOR**3-1.5*B1*YT
               2OR*YTOR+B1*B1*YTOR-B1*B1*B1/4.)+YBAR**4*T3/4.-YTOR**4*T3/4.)/XIR+2
               3.*YOR
      C
      C         CHECKING SECTION LIMITATIONS
      C
0051            DWT=B3/B2
0052            DWB=B3/B4
0053            XLB3=XL/B3
0054            B2T2=B2/T2
0055            B4T4=B4/T4
0056            R1=(YTL/YBL)*(ALPHA*XL/B3)
0057            R2=YTL/YBL
0058            IF(DWT.LT.1.5.OR.DWB.LT.1.5.OR.DWT.GT.3.0.OR.DWB.GT.3.0.OR.XLB3.GT
               1.24.0.OR.B2T2.LT.15.0.OR.B4T4.LT.15.0.OR.B2T2.GT.50.0.OR.B4T4.GT.5
               20.0.OR.B3.LT.6.0.OR.B3.GT.24.0.OR.B2.LT.3.0.OR.B2.GT.12.0.OR.B4.LT
               4.3.0.OR.B4.GT.12.0.OR.T2.LT.0.1875.OR.T2.GT.0.75.OR.T4.LT.0.1875.O
               5R.T4.GT.0.75.OR.R1.GT.1.30.OR.R2.GT.2.50)GO TO 750
0059            GO TO 171
      C
      C            OUTPUT OF SECTION DATA
      C
0060        750 WRITE(6,114)B1,T1,B2,T2,B3,T3,B4,T4,XL,ALPHA,FY
0061        114 FORMAT(1H0,//,' SECTION DOES NOT MEET SPECIFIED LIMITATIONS',///,'
               1 DIMENSIONS OF CROSS SECTION',//,3X,'B1  =', F9.4,' IN',3X,'T1  =
               2', F7.4,' IN',//,3X,'B2  =', F9.4,' IN',3X,'T2  =', F7.4,' IN',//,
               33X,'B3  =', F9.4,' IN',3X,'T3  =', F7.4,' IN',//,3X,'B4  =', F9.4,
               4' IN',3X,'T4  =', F7.4,' IN',//,3X,'LENGTH =',F10.2,' IN',//,3X,'A
               5LPHA =',F8.4,///,2X,'FY =', F7.2,' KSI')
0062            GO TO 170
0063        171 WRITE(6,113)B1,T1,B2,T2,B3,T3,B4,T4,XL,ALPHA,FY
0064        113 FORMAT(1H0,/' DIMENSIONS OF CROSS SECTION'//
               13X,'B1  =', F9.4,' IN',3X,'T1  =', F7.4,' IN'//
               13X,'B2  =', F9.4,' IN',3X,'T2  =', F7.4,' IN'//
               13X,'B3  =', F9.4,' IN',3X,'T3  =', F7.4,' IN'//
               13X,'B4  =', F9.4,' IN',3X,'T4  =', F7.4,' IN'//
               13X,'LENGTH=',F10.2,' IN.',//,3X,'ALPHA = ',F8.4,///,2X,'FY =', F7.
```

```
                  12,' KSI')
0065          170 WRITE(6,111)ROL,ROR,YOL,YOR,YTOL,YTOR
0066          111 FORMAT(         ///,10X,'LEFT END SECTION PROPERTIES',20X,'RIGH
                 2T END SECTION PROPERTIES',//,7X,'RO    =',F11.4,' IN.',22X,'RO    =
                 2',F10.4,' IN.',//,7X,'YO    = ',F10.4,' IN.',22X,'YO    = ',F10.4,'
                 3IN.',//,7X,'YBAR = ',F10.4,' IN. FROM TOP',13X,'YBAR = ',F10.4,' I
                 1N. FROM TOP')
0067              WRITE(6,103)YBAL,YBAR,AREA,AREAR,XIL,XIR
0068          103 FORMAT(/,7X,'YBAR = ',F10.4,' IN.FROM BOTTOM',11X,'YBAR = ',F10.4,
                 1' IN. FROM BOTTOM',//,7X,'AREA = ',F10.4,' IN**2',20X,'AREA = ',F1
                 20.4,' IN**2',//,7X,'IX    = ',F10.4,' IN**4',20X,'IX    = ',F10.4,'
                 3IN**4')
0069              WRITE(6,73)YI,YI,WIL,WIR,TJL,TJR,BXL,BXR,S(1),S(3),S(2),S(4)
0070           73 FORMAT( /,7X,'IY    = ',F10.4,' IN**4',20X,'IY    = ',F10.4,' IN**4'
                 1,//,7X,'CW    = ',F10.4,' IN**6',20X,'CW    = ',F10.4,' IN**6',//,7X
                 2,'J     = ',F10.4,' IN**4',20X,'J     = ',F10.4,' IN**4',//,7X,'BX
                 3 = ',F10.4,' IN.',22X,'BX    = ',F10.4,' IN.',//,7X,'SEC. MOD. =',F
                 410.4,' IN**3 TOP',12X,'SEC. MOD. =',F10.4,' IN**3 TOP',//,7X,'SEC.
                 5 MOD. =',F10.4,' IN**3 BOTTOM',9X,'SEC. MOD. =',F10.4,' IN**3 BOTT
                 6OM')
0071              WRITE(6,75)XMS,XML
0072           75 FORMAT(//,7X,'MOMENT =',F10.2,' K-IN.',19X,'MOMENT =',F10.2,' K-IN
                 1.')
            C
            C
            C     CALCULATION OF STRESSES AT FLANGE TIPS
            C
0073              F(1)=-XMS/S(1)
0074              F(2)=XMS/S(2)
0075              F(3)=XML/S(3)
0076              F(4)=-XML/S(4)
0077              WRITE(6,2001)
0078              WRITE(6,2000)
0079              WRITE(6,112)F(1),F(2),F(3),F(4)
0080          112 FORMAT(///,1X,'STRESS AND MOMENT RESULTS',//,3X,'STRESS AT LEFT TO
                 1P   =',F10.2,' KSI',10X,'STRESS AT LEFT BOTTOM   =',F9.2,' KSI',//,3
                 2X,'STRESS AT RIGHT TOP =',F10.2,' KSI',10X,'STRESS AT RIGHT BOTTOM
                 3 =', F9.2,' KSI')
0081          105 I=1
0082              J=1
0083          180 FMIN=F(J)
0084          106 I=I+1
0085          107 IF(I.LE.4)GO TO 108
0086              GO TO 109
0087          108 IF(FMIN.LT.F(I))GO TO 106
0088              J=I
0089              GO TO 180
0090          109 WRITE(6,110)J,FMIN
0091          110 FORMAT (//,3X,'NODE WITH MAX. COMPR. STRESS =',I3,//,3X,'MAX. COMP.
                 1 STRESS =',F10.2,' KSI')
            C
            C     CALCULATION OF CRITICAL MOMENTS FOR BASIC CASE
            C
0092              IF(J.EQ.2.OR.J.EQ.4)GO TO 500
0093              CRM=(PI**2*E*YI*BXL/(2.*XL**2))*(1.+(1.+(4./BXL**2)*(WIL/YI+G*TJL*
                 1XL**2/(PI**2*E*YI)))**0.5)
0094              GO TO 510
0095          500 CRM=(PI**2*E*YI*BXL/(2.*XL**2))*(1.-(1.+(4./BXL**2)*(WIL/YI+G*TJL*
```

```
                 1XL**2/(PI**2*E*YI)))**0.5)
0096          510 WRITE(6,120) CRM
0097          120 FORMAT(//,3X,'CRITICAL MOMENT BASIC CASE =',F10.2,' K-IN.')
            C
            C         CALCULATION OF CA
            C
0098              CA=1.0-1.4580*ALPHA*(G*TJL*XL**2/(E*WIL))**0.5+44.6328*ALPHA*TJL/Y
                 1I
0099              IF(J.EQ.1.OR.J.EQ.3)GO TO 520
0100              R=S(4)/S(2)
0101              GO TO 530
0102          520 R=S(3)/S(1)
            C
            C
            C         CALCULATION OF CB
            C
0103          530 IF(J.EQ.1.OR.J.EQ.2)GO TO 540
0104              IF(J.EQ.3)GO TO 550
0105              RF=F(2)/F(4)
0106              GO TO 560
0107          550 RF=F(1)/F(3)
0108              GO TO 560
0109          540 RF=F(4)/F(2)
0110              IF(J.EQ.1)RF=F(3)/F(1)
            C
0111          560 WRITE(6,130) R,RF
0112          130 FORMAT(//,3X,'MODULAR RATIO REF. FLANGE =',F10.2,//,3X,'STRESS RAT
                 1IO REF. FLANGE  =',F10.4)
            C
0113          565 IF(RF+0.4)570,580,580
            C
            C         CALCULATION OF CB IF RF IS GREATER THAN -0.4
            C
0114          580 CB=1.-0.3867*(RF-1.)+0.4739*(RF-1.)**2+0.9074*ALPHA*
                 1XL*(RF-1.)**2/B3
0115              IF(J.EQ.3.OR.J.EQ.4)GO TO 590
0116              GO TO 630
0117          590 CB=1.-0.3867*(RF-1.)+0.4739*(RF-1.)**2+0.9074*ALPHA*
                 1XL*(RF-1.)**2/BR
0118              GO TO 630
            C
            C         CALCULATION OF CB IF RF IS LESS THAN -0.4
            C
0119          570 CB1=1.-0.3867*(-.4-1.)+0.4739*(-.4-1.)**2+0.9074*ALPHA*XL*
                 1(-.4-1.)**2/B3
0120              IF(J.EQ.3.OR.J.EQ.4)GO TO 600
0121              GO TO 615
0122          600 CB1=1.-0.3867*(-.4-1.)+0.4739*(-0.4-1.)**2+0.9074*ALPHA*XL*
                 1(-0.4-1.)**2/BR
0123              IF(J.EQ.3.OR.J.EQ.4)GO TO 610
0124          615 CB2=2.7596-2.8152*BXL/B3-0.6562*ALPHA*XL/B3-15.653*ALPHA*BXL/B3
0125              GO TO 620
0126          610 CB2=2.7684+1.2025*ALPHA*XL/BR-2.2686*BXR/BR-22.0274*ALPHA*BXR/BR
0127          620 CB=CB2+(CB1-CB2)*(1.+RF)/0.6
0128          630 WRITE(6,140)CA,CB
0129          140 FORMAT(//,3X,'CA =',F10.4,15X,'CB =',F10.4)
            C
            C         CALCULATION OF MAX. COMPRESSIVE STRESS AT REFERENCE END
```

```
C
      IF(J.EQ.3.OR.J.EQ.4)GO TO 700
      IF(F(1).LT.0.0) GO TO 900
      S(5)=S(2)
      S(6)=S(1)
      GO TO 1000
  900 S(5)=S(1)
      S(6)=S(2)
      GO TO 1000
  700 IF(F(3).LT.0.0) GO TO 950
      S(5)=S(4)
      S(6)=S(3)
      GO TO 1200
  950 S(5)=S(3)
      S(6)=S(4)
C
C     MAX. COMPRESSIVE STRESS LARGE END REFERENCE
C
 1200 FBC=CA*CB*P*CRM/S(5)
      GO TO 1250
C
C     MAX. COMPRESSIVE STRESS SMALL END REFERENCE
C
 1000 FBC=CA*CB*CRM/S(5)
C
C     MODIFICATION OF FBC USING CRC FORMULA,IF NECESSARY
C
 1250 FALL=0.5*FY
      FBC=ABS(FBC)
      IF(FBC.LT.FALL)GO TO 1400
      FBC=FY*(1.-FY/(4.*FBC))
C
C     CHECK TO SEE IF YIELDING GOVERNS
C
 1400 FBT=FBC*S(5)/S(6)
      IF(FBT.LT.FY)GO TO 1450
      FBT=FY
C
C     MODIFICATION OF FBC IF YIELDING GOVERNS
C
      FBC=FBT*S(6)/S(5)
      GO TO 800
 1450 WRITE(6,1410)FBC
 1410 FORMAT(//,3X,'GOVERNING STRESS AT FAILURE  =',F7.2,' KSI C')
      WRITE(6,1411)J
      GO TO 812
  800 IF(J.EQ.1.OR.J.EQ.3)GO TO 811
      K=J-1
      GO TO 830
  811 K=J+1
  830 WRITE(6,810)FBT
  810 FORMAT(//,3X,'GOVERNING STRESS AT FAILURE  =',F7.2,' KSI T')
      WRITE(6,1411)K
 1411 FORMAT(//,3X,'LOCATION OF GOVERNING STRESS =',I3)
  812 IF(J.EQ.3.OR.J.EQ.4)GO TO 200
C
C     CALCULATION OF STRESSES AND MOMENTS AT FAILURE,SMALL END REFERENCE
C
```

```
0169            MCRS=FBC*S(5)
0170            F1=MCRS/S(1)
0171            F2=MCRS/S(2)
0172            XCRL=MCFS*RF*R
0173            MCRL=ABS(XCRL)
0174            F3=MCRL/S(3)
0175            F4=MCRL/S(4)
0176            IF(J.EQ.2)GO TO 201
0177            IF(FF.LT.0.0)GO TO 301
0178            WRITE(6,400)F1,F3,F2,F4
0179            GO TO 815
0180        301 WRITE(6,401)F1,F3,F2,F4
0181            GO TO 815
0182        201 IF(RF.LT.0.0)GO TO 302
0183            WRITE(6,403)F1,F3,F2,F4
0184            GO TO 815
0185        302 WRITE(6,404)F1,F3,F2,F4
0186            GO TO 815
           C
           C    CALCULATION OF STRESSES AND MOMENTS AT FAILURE,LARGE END REFERENCE
           C
0187        200 MCRL=FBC*S(5)
0188            F3=MCRL/S(3)
0189            F4=MCRL/S(4)
0190            XCRS=MCFL*PF/R
0191            MCRS=ABS(XCRS)
0192            F1=MCRS/S(1)
0193            F2=MCRS/S(2)
0194            IF(J.EQ.4)GO TO 304
0195            IF(PF.LT.0.0)GO TO 303
0196            WRITE(6,400)F1,F3,F2,F4
0197        400 FORMAT(//,3X,'STRESSES AT FAILURE',//,7X,'LOCATION 1   ',F10.2,' KS
               1I C',20X,'LOCATION 3   ',F10.2,' KSI C',//,7X,'LOCATION 2   ',F10.2,
               2' KSI T',20X,'LOCATION 4   ',F10.2,' KSI T')
0198            GO TO 815
0199        303 WRITE(6,404)F1,F3,F2,F4
0200        404 FORMAT(//,3X,'STRESSES AT FAILURE',//,7X,'LOCATION 1   ',F10.2,' KS
               1I T',20X,'LOCATION 3   ',F10.2,' KSI C',//,7X,'LOCATION 2   ',F10.2,
               2' KSI C',20X,'LOCATION 4   ',F10.2,' KSI T')
0201            GO TO 815
0202        304 IF(RF.LT.0.0)GO TO 305
0203            WRITE(6,403)F1,F3,F2,F4
0204        403 FORMAT(//,3X,'STRESSES AT FAILUPE',//,7X,'LOCATION 1   ',F10.2,' KS
               1I T',20X,'LOCATION 3   ',F10.2,' KSI T',//,7X,'LOCATION 2   ',F10.2,
               2' KSI C',20X,'LOCATION 4   ',F10.2,' KSI C')
0205            GO TO 815
0206        305 WRITE(6,401)F1,F3,F2,F4
0207        401 FORMAT(//,3X,'STRESSES AT FAILURE',//,7X,'LOCATION 1   ',F10.2,' KS
               1I C',20X,'LOCATION 3   ',F10.2,' KSI T',//,7X,'LOCATION 2   ',F10.2,
               2' KSI T',20X,'LOCATION 4   ',F10.2,' KSI C')
0208        815 WRITE(6,402)MCRS,MCRL
0209        402 FORMAT(//,3X,'CRITICAL MOMENTS AT FAILURE',//,7X,'SMALL END   ',F10
               1.2,' IN-K',20X,'LARGE END   ',F10.2,' IN-K')
0210            GO TO 999
0211         55 STOP
0212            END
```

```
*OPTIONS IN EFFECT*  NOTERM,ID,EBCDIC,SOURCE,NOLIST,NODECK,LOAD,NOMAP,NOTEST
*OPTIONS IN EFFECT*  NAME = MAIN    , LINECNT =      60
*STATISTICS*    SOUPCE STATEMENTS =      212,PRCGRAM SIZE =     9784
*STATISTICS*  NO DIAGNOSTICS GENERATED
```

```
F64-LEVEL LINKAGE EDITOR OPTIONS SPECIFIED LIST,MAP,LET
         DEFAULT OPTION(S) USED -   SIZE=(196608,38912)

                                              MODULE MAP

 CONTROL SECTION                        ENTRY

   NAME     ORIGIN  LENGTH               NAME   LOCATION     NAME   LOCATION     NAME   LOCATION     NAME   LOCATION
 MAIN           00    2638
 IHOFRXPR*    2638     4A4
                                        FRXPR#      2638
 IHOECOMH*    2AE0     E30
                                        IBCOM#      2B14     FDIOCS#     2BD0    INTSWTCH     3858
 FIOAP#  *    3910     624
 IHOCOMH2*    3F38     9A5
                                        SEQDASD     430A
 IHOFCVTH*    48E0     CEA
                                        ADCON#      48E0     FCVAOUTP    494C    FCVLOUTP     4964    FCVZOUTP     497C
                                        FCVIOUTP    4994     FCVEOUTP    49CC    FCVCOUTP     49CC    ADCONI#      50D0
                                        ADCONO#     50EC     INT6SWCH    55C9
 IHOEFNTH*    55D0     800
                                        ARITH#      55D0     ADJSWTCH    5B64
 IHOEFICS*    5DD0    118C
                                        FIOCS#      5DD0     FIOCSBEP    5DD6
 IHOFIOS2*    6F60     642
 IHOSEXP *    75A8     258
                                        IH$$EXP     75A8     EXP         75A8
 IHOSLGN *    7800     244
                                        IH$ALOG     7800     ALOG        7800    LOG          7800
 IHOERRM *    7A48     624
                                        ERRMON      7A48     IHOERRE     7A60
 IHOUOPT *    8070     338
 IHOFCONI*    83A8     416
                                        FQCONI#     83A8
 IHOFCONO*    87C0     8B8
                                        FQCONO#     87C0
 IHOETRCH*    9078     2AE
                                        IHOTRCH     9078     ERRTRA      9080
 IHOUATBL*    9328     638
 IHOFTEN *    9960     220
                                        FTEN#       9960

ENTRY ADDRESS        00

 TOTAL LENGTH      9B80
****FORTGCLG  DOES NOT EXIST BUT HAS BEEN ADDED TO DATA SET
AUTHORIZATION CODE IS          0.
```

EXAMPLE PROBLEM CASE 1

SECTION DOES NOT MEET SPECIFIED LIMITATIONS

DIMENSIONS OF CROSS SECTION

```
B1 =  0.0     IN   T1 = 0.0     IN
B2 =  6.0000  IN   T2 = 0.2500  IN
B3 = 10.0000  IN   T3 = 0.1250  IN
B4 =  4.0000  IN   T4 = 0.2500  IN
LENGTH =   120.00 IN
ALPHA =  0.1167

FY =  50.00 KSI
```

LEFT END SECTION PROPERTIES		RIGHT END SECTION PROPERTIES	
RO =	24.7518 IN.	RO =	120.9223 IN.
YO =	2.0476 IN.	YO =	5.4234 IN.
YBAR =	4.3333 IN. FROM TOP	YBAR =	10.9091 IN. FROM TOP
YBAR =	5.6667 IN.FROM BOTTOM	YBAR =	13.0909 IN. FROM BOTTOM
AREA =	3.7500 IN**2	AREA =	5.5000 IN**2
IX =	71.2630 IN**4	IX =	497.4683 IN**4
IY =	5.8333 IN**4	IY =	5.8333 IN**4
CW =	102.8571 IN**6	CW =	592.4583 IN**6
J =	0.0586 IN**4	J =	0.0677 IN**4
BX =	5.0658 IN.	BX =	12.3334 IN.
SEC. MOD. =	16.4453 IN**3 TOP	SEC. MOD. =	45.6012 IN**3 TOP
SEC. MOD. =	12.5758 IN**3 BOTTOM	SEC. MOD. =	38.0010 IN**3 BOTTOM
MOMENT =	500.00 K-IN.	MOMENT =	-1000.00 K-IN.

```
EXAMPLE PROBLEM CASE 1

STRESS AND MOMENT RESULTS
  STRESS AT LEFT TOP  =    -30.40 KSI         STRESS AT LEFT BOTTOM  =    39.76 KSI
  STRESS AT RIGHT TOP =    -21.93 KSI         STRESS AT RIGHT BOTTOM =    26.32 KSI

  NODE WITH MAX. COMPR. STRESS =  1
  MAX. COMP. STRESS =    -30.40 KSI

  CRITICAL MOMENT BASIC CASE =    955.48 K-IN.

  MODULAR RATIO REF. FLANGE =      2.77
  STRESS RATIO REF. FLANGE  =    0.7213

  CA =    0.7546              CB =    1.2433

  GOVERNING STRESS AT FAILURE  =  50.00 KSI T

  LOCATION OF GOVERNING STRESS =  2

  STRESSES AT FAILURE
     LOCATION 1       38.24 KSI C                LOCATION 3       27.58 KSI C
     LOCATION 2       50.00 KSI T                LOCATION 4       33.09 KSI T

  CRITICAL MOMENTS AT FAILURE
     SMALL END      628.79 IN-K                LARGE END     1257.58 IN-K
```

```
EXAMPLE PROBLEM CASE 2.

SECTION DOES NOT MEET SPECIFIED LIMITATIONS

 DIMENSIONS OF CROSS SECTION

  B1  =   0.0      IN   T1  = 0.0     IN
  B2  =   6.0000 IN   T2  = 0.2500 IN
  B3  =  10.0000 IN   T3  = 0.1250 IN
  B4  =   4.0000 IN   T4  = 0.2500 IN
  LENGTH =    120.00 IN
  ALPHA =  0.1167

 FY =  50.00 KSI

     LEFT END SECTION PROPERTIES                    RIGHT END SECTION PROPERTIES

   RO    =     24.7518 IN.                       RO    =   120.9223 IN.
   YO    =      2.0476 IN.                       YO    =     5.4234 IN.
   YBAR =      4.3333 IN. FROM TOP              YBAR =    10.9091 IN. FROM TOP
   YBAR =      5.6667 IN.FROM BOTTOM            YBAR =    13.0909 IN. FROM BOTTOM
   AREA =      3.7500 IN**2                     AREA =     5.5000 IN**2
   IX    =     71.2630 IN**4                     IX    =   497.4683 IN**4
   IY    =      5.8333 IN**4                     IY    =     5.8333 IN**4
   CW    =    102.2571 IN**6                     CW    =   592.4583 IN**6
   J     =      0.0586 IN**4                     J     =     0.0677 IN**4
   BX    =      5.0658 IN.                       BX    =    12.3334 IN.
   SEC. MOD. =    16.4453 IN**3 TOP             SEC. MOD. =   45.6012 IN**3 TOP
   SEC. MOD. =    12.5758 IN**3 BOTTOM          SEC. MOD. =   38.0010 IN**3 BOTTOM

   MOMENT =   -500.00 K-IN.                      MOMENT =  -1000.00 K-IN.
```

EXAMPLE PROBLEM CASE 2

```
STRESS AND MOMENT RESULTS
  STRESS AT LEFT TOP  =     30.40 KSI          STRESS AT LEFT BOTTOM  =   -39.76 KSI
  STRESS AT RIGHT TOP =    -21.93 KSI          STRESS AT RIGHT BOTTOM =    26.32 KSI

  NODE WITH MAX. COMPR. STRESS =  2

  MAX. COMP. STRESS =    -39.75 KSI

  CRITICAL MOMENT BASIC CASE =   -347.87 K-IN.

  MODULAR RATIO REF. FLANGE =       3.02
  STRESS RATIO REF. FLANGE  =   -0.6619

  CA =    0.7546                CB =    2.5726

  GOVERNING STRESS AT FAILURE  =  38.36 KSI C

  LOCATION OF GOVERNING STRESS =  2

  STRESSES AT FAILURE
     LOCATION 1       29.34 KSI T                LOCATION 3       21.16 KSI C
     LOCATION 2       38.36 KSI C                LOCATION 4       25.39 KSI T

  CRITICAL MOMENTS AT FAILURE
     SMALL END      482.43 IN-K                LARGE END      964.85 IN-K
```

```
EXAMPLE PROBLEM CASE 3

SECTICN DOES NOT MEET SPECIFIED LIMITATIONS

DIMENSICNS OF CROSS SECTION

B1 =   0.0     IN    T1 = 0.0     IN
B2 =   6.0000 IN    T2 = 0.2500 IN
B3 =  10.0000 IN    T3 = 0.1250 IN
B4 =   4.0000 IN    T4 = 0.2500 IN
LENGTH =    120.00 IN
ALPHA =  0.1167

FY =  50.00 KSI
```

```
LEFT END SECTION PROPERTIES

RO    =     24.7518 IN.
YO    =      2.0476 IN.
YBAR  =      4.3333 IN. FROM TOP
YBAR  =      5.6667 IN.FPOM BOTTOM
AREA  =      3.7500 IN**2
IX    =     71.2630 IN**4
IY    =      5.8333 IN**4
CW    =    102.8571 IN**6
J     =      0.0586 IN**4
BX    =      5.0658 IN.
SEC. MOD. =    16.4453 IN**3 TOP
SEC. MOD. =    12.5758 IN**3 BOTTOM

MOMENT =    360.50 K-IN.
```

```
RIGHT END SECTION PROPERTIES

RO    =    120.9223 IN.
YO    =      5.4234 IN.
YBAR  =     10.9091 IN. FROM TOP
YBAR  =     13.0909 IN. FROM BOTTOM
AREA  =      5.5000 IN**2
IX    =    497.4683 IN**4
IY    =      5,8333 IN**4
CW    =    592.4583 IN**6
J     =      0.0677 IN**4
BX    =     12.3334 IN.
SEC. MOD. =    45.6012 IN**3 TOP
SEC. MOD. =    38.0010 IN**3 BOTTOM

MOMENT =  -2000.00 K-IN.
```

```
EXAMPLE PROBLEM CASE 3'

STRESS AND MOMENT RESULTS
  STRESS AT LEFT TOP  =    -21.92 KSI          STRESS AT LEFT BOTTOM  =    28.67 KSI
  STRESS AT RIGHT TOP =    -43.86 KSI          STRESS AT RIGHT BOTTOM =    52.63 KSI

  NODE WITH MAX. COMPR. STRESS =  3
  MAX. COMP. STRESS =    -43.86 KSI

  CRITICAL MOMENT BASIC CASE =    955.48 K-IN.

  MODULAR RATIO REF. FLANGE =      2.77
  STRESS RATIO REF. FLANGE  =    0.4998

  CA =    0.7546              CB =    1.4444

  GOVERNING STRESS AT FAILURE  =  40.13 KSI C

  LOCATION OF GOVERNING STRESS =  3

  STRESSES AT FAILURE
     LOCATION 1       20.06 KSI C                 LOCATION 3     40.13 KSI C
     LOCATION 2       26.23 KSI T                 LOCATION 4     48.16 KSI T

  CRITICAL MOMENTS AT FAILURE
     SMALL END      329.86 IN-K                LARGE END    1830.02 IN-K
```

EXAMPLE PROBLEM CASE 4

SECTION DOES NOT MEET SPECIFIED LIMITATIONS

DIMENSIONS OF CROSS SECTION

```
B1  =    0.0     IN   T1  = 0.0     IN
B2  =    6.0000  IN   T2  = 0.2500  IN
B3  =   10.0000  IN   T3  = 0.1250  IN
B4  =    4.0000  IN   T4  = 0.2500  IN
LENGTH =   120.00 IN
ALPHA =  0.1167

FY =  50.00 KSI
```

LEFT END SECTION PROPERTIES		RIGHT END SECTION PROPERTIES	
RO =	24.7518 IN.	RO =	120.9223 IN.
YO =	2.0476 IN.	YO =	5.4234 IN.
YBAR =	4.3333 IN. FROM TOP	YBAR =	10.9091 IN. FROM TOP
YBAR =	5.6667 IN.FROM BOTTOM	YBAR =	13.0909 IN. FROM BOTTOM
AREA =	3.7500 IN**2	AREA =	5.5000 IN**2
IX =	71.2630 IN**4	IX =	497.4683 IN**4
IY =	5.8333 IN**4	IY =	5.8333 IN**4
CW =	102.8571 IN**6	CW =	592.4583 IN**6
J =	0.0586 IN**4	J =	0.0677 IN**4
BX =	5.0658 IN.	BX =	12.3334 IN.
SEC. MOD. =	16.4453 IN**3 TOP	SEC. MOD. =	45.6012 IN**3 TOP
SEC. MOD. =	12.5758 IN**3 BOTTOM	SEC. MOD. =	38.0010 IN**3 BOTTOM
MOMENT =	-331.00 K-IN.	MOMENT =	2000.00 K-IN.

```
EXAMPLE PROBLEM CASE 4

STRESS AND MOMENT RESULTS

  STRESS AT LEFT TOP  =     20.13 KSI          STRESS AT LEFT BOTTOM  =   -26.32 KSI
  STRESS AT RIGHT TOP =     43.86 KSI          STRESS AT RIGHT BCTTOM =   -52.63 KSI

  NODE WITH MAX. COMPR. STRESS =  4
  MAX. COMP. STPESS =    -52.63 KSI

  CRITICAL MOMENT BASIC CASE =   -347.87 K-IN.

  MODULAR RATIO REF. FLANGE =       3.02
  STRESS RATIO REF. FLANGE  =     0.5001

  CA =    0.7546                 CB =    1.4440

  GOVERNING STRESS AT FAILURE  =  29.27 KSI C

  LOCATION OF GOVERNING STRESS =  4

  STRESSES AT FAILURE
      LOCATION 1       11.19 KSI T                     LOCATION 3       24.39 KSI T
      LOCATION 2       14.64 KSI C                     LOCATION 4       29.27 KSI C

  CRITICAL MOMENTS AT FAILURE
      SMALL END      184.05 IN-K                     LARGE END     1112.11 IN-K
```