Strength Design in Steel Using Programmable Calculators

JAMES C. ANDERSON

During the past ten years, a gap has developed between large digital computers and manual computing methods. This is particularly true with regard to the analyses and design of structural members. The increasing availability of large computer systems and their associated software, coupled with a dramatic reduction in cost, has placed computer analysis capability well within reach of most structural engineering offices. Member forces can be readily determined using either special frame analysis programs or general purpose programs which are available on commercial computer systems. However, once the member design forces are determined, the engineer must revert to hand calculations for determining and checking the design of the individual members. These calculations can become monotonous and time consuming for a structure of even modest size. Furthermore, such calculations tend to become inaccurate and insufficient. For example, a designer may accept a given section as adequate without trying other possibilities which may lead to a more efficient design.

Recent developments in the field of mini-computers are changing this situation. The smallest mini-computer is commonly referred to as a programmable calculator, with somewhat larger systems being referred to as personal computers. These powerful yet compact machines perform a series of computations according to instructions stored on magnetic card, tape cassette, or floppy disk. These systems have also seen a dramatic reduction in price over the past five years and are now appearing in increasing numbers in design offices. This is particularly true of the leading hand-held calculators, which currently sell for under $400.00.

Programmable calculators are a powerful tool for bridging the gap between large scale computer systems and final design calculations. All that remains is to develop the software necessary to interface with the large scale system and solve the design problem. This will free the design engineer from the more mundane calculations involved in member selection and give him time for more creative endeavors.

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The purpose of this paper is to indicate how this can be accomplished using a hand-held programmable calculator having a magnetic card reader and 224 program steps. A design procedure is presented which is based on the strength design concepts given in Part 2 of the AISC Specification (AISCS).[1] The procedure is divided into a series of subroutines and detailed flow charts are presented. This will allow practicing engineers a certain amount of flexibility in adapting the procedure to their individual calculators or mini-computers. The detailed flow charts will also facilitate changes in the design procedure due to later changes in the design specifications. Finally, several detailed design examples are presented to illustrate the implementation on a particular programmable calculator.

STRENGTH DESIGN PROCEDURE

The design procedure presented below assumes that the member forces have been determined from a force analysis using factored loads. The design of three basic structural members is considered. These include the vertical bracing member, the beam member, and the beam-column member. The design requirements are those specified in Part 2 of the AISCS. The design of these members is discussed in the following paragraphs with reference to the flow diagrams given in Appendix A.

Vertical Bracing Member—The maximum strength of an axially loaded tension member is limited to:

$$P_{cr} = 0.85\ AF_y \quad \text{[AISCS Formula (2.3-1)]} \qquad (1)$$

Furthermore, the maximum strength of an axially loaded compression member may not exceed

$$P_{cr} = 1.7\ AF_a \quad \text{[AISCS Formula (2.4-1)]} \qquad (2)$$

where A is the gross area of the member, F_y is the yield stress of the steel, and F_a is the allowable axial stress given by Formula (1.5-1) of the AISCS. Hence, the maximum axial force in the member is given by Eq. (1) for tension and by the smaller of Eqs. (1) and (2) for compression. This procedure is detailed in Chart 1 of Appendix A, which is identified by $\boxed{\text{C}} \rightarrow P_{critical}$.

Beam Member—For beam members, the required plastic section modulus, Z, can be readily obtained from:

$$Z = M_{max}/F_y \tag{3}$$

where M_{max} is the maximum bending moment at the ends of the member. This simple procedure is represented by Chart 2, which is identified as $\boxed{C} \rightarrow Z_{required}$. In order to insure that the member can develop its full plastic moment capacity, it is necessary to check the section to see that a premature local instability will not occur. This is done by checking the restrictions placed on the following geometric ratios: (a) b/t which protects against flange buckling, (b) d/t which protects against web crippling, and (c) L/r which protects against lateral buckling.

The b/t requirement is given in tabular form in Sect. 2.7 of the AISCS. This table is represented in Chart 5, identified as $\boxed{d} \rightarrow (b/t)_{max}$.

The maximum depth-to-thickness ratio of the web is given by:

$$\frac{d}{t} = \frac{412}{\sqrt{F_y}}\left(1 - 1.4\frac{P}{P_y}\right) \quad \text{when } \frac{P}{P_y} \leq 0.27$$

[AISCS Formula (2.7-1a)] (4)

or

$$\frac{d}{t} = \frac{257}{\sqrt{F_y}} \quad \text{when } \frac{P}{P_y} > 0.27$$

[AISCS Formula (2.7-1b)] (5)

This requirement is presented in Chart 4, which is identified by $\boxed{D} \rightarrow (d/t)_{max}$.

In Sect. 2.9 of the AISC Specification, the unsupported length between adjacent points which are braced laterally is limited to:

$$L_{cr} = r_y\left(\frac{1375}{F_y} + 25\right) \quad \text{when } 1 > \frac{M}{M_p} > -0.5$$

[AISCS Formula (2.9-1a)] (6)

or

$$L_{cr} = r_y\left(\frac{1375}{F_y}\right) \quad \text{when } -0.5 \geq \frac{M}{M_p} > -1.0$$

[AISCS Formula (2.9-1b)] (7)

Here M is the smaller of the moments at the ends of the unbraced segment and the sign of M/M_P is positive when the member is bent in reverse curvature. This requirement is represented by Chart 3, identified by $\boxed{c} \rightarrow L_{cr}$.

The web of the member must also be checked to see that the ultimate shear capacity is not exceeded. The ultimate shear is calculated as:

$$V_u = 0.55F_ytd \quad \text{[AISCS Formula (2.5-1)]}$$

where t is the web thickness and d is the depth of the section. This value is then compared with the developed shear.

This series of small subroutines combined with the property tables in the AISC Manual produce a simple and expedient means of designing beams, once the design forces are known.

Beam-Column Members—Columns which are subjected to axial compression plus end moments are the more complicated members to design. Using manual computations, it is largely a trial and error process. Because of this, the use of a programmed design procedure is extremely helpful. The Specification requires that members subjected to combined axial load and bending moment satisfy the following two interaction formulas:

$$\frac{P}{P_{cr}} + \frac{C_mM}{\left(1 - \frac{P}{P_e}\right)M_m} \leq 1.0$$

[AISCS Formula (2.4-2)] (8)

$$\frac{P}{P_y} + \frac{M}{1.8M_p} \leq 1.0 \qquad \text{[AISCS Sect. 2.3.2]} \tag{9}$$

In all cases, the applied bending must not exceed the plastic moment capacity. If the column is part of an unbraced frame, the axial force cannot exceed

$$P = 0.75AF_y \quad \text{[AISCS Formula (2.3-2)]} \tag{10}$$

For purposes of design, it is convenient to express Eq. (9) in the following form:

$$M_{p1} = \frac{M_{max}}{C_1\left(1 - \frac{P}{F_yA}\right)} \tag{11}$$

where $C_1 = 1.18$ for bending about the strong axis and $C_1 = 1.67$ for bending about the weak axis. In this form, M_{p1} is the estimate of the required plastic moment capacity, based on Eq. (9). In a similar manner, Eq. (8) may be expressed in the form:

$$M_{p2} = \frac{C_mM_{max}}{\left(1 - \frac{P}{P_e}\right)\left(1 - \frac{P}{P_{cr}}\right)C_3} \tag{12}$$

where

$$P_e = \frac{23}{12}AF'_e \tag{13}$$

and

$$F'_e = \frac{12}{23}\frac{\pi^2E}{(KL_b/r_b)^2} \tag{14}$$

KL_b and r_b are the effective unsupported length and the radius of gyration in the plane of bending; respectively, C_m is a coefficient defined in Sect. 1.6.1, which accounts for the moment gradient across the member; P_{CR} is defined by Eq. (2), and C_3 is a coefficient which depends upon whether or not the columns are braced in the weak direction. The

Commentary to the AISC Specification states that a column is considered to be fully braced if the distance between brace points is less than or equal to the length given by Eq. (6) or (7). If the columns are braced, the coefficient C_3 has the value of unity. If the columns are not braced, the coefficient is given as:

$$C_3 = 1.07 - \frac{(L/r_y)\sqrt{F_y}}{3160} \leq 1.0 \qquad \text{[AISCS Formula (2.4-4)]} \quad (15)$$

In this form, M_{p2} represents the required plastic moment capacity based on Eq. (8). The design of the section must be based on the larger of the values determined by Eq. (11) or (12). If this maximum moment is denoted as M_{pm}, the required plastic section modulus can be determined as:

$$Z_{required} = \frac{M_{pm}}{F_y} \qquad (16)$$

This design routine is represented in Chart 6, which is identified as $\boxed{a} \rightarrow Z_{required}$. This algorithm uses one subroutine, identified as subroutine B, which calculates the moment gradient coefficient C_m.

With these simple subroutines and a table of properties of rolled sections, one can design the primary members in a steel framing system. Implementing these routines on a small programmable calculator makes the process semi-automated. However, this is still a significant step in freeing the design engineer from the more mundane calculations involved in steel design. In the following sections, several detailed design examples are presented to illustrate the application of these procedures.

DESIGN APPLICATIONS

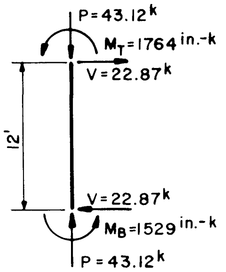
In this section, the semi-automated design of the basic steel structural members is illustrated. The members and forces are representative of multistory frames, both braced and unbraced. The design forces are determined using factored loads and an elastic force analysis performed by the ETABS[2] program. This is a very efficient program for regular multistory frames, available for a nominal service charge through the National Information Service for Earthquake Engineering. The program has a simplified output format which is easily adaptable to design computations. It also has the capability of considering both static and dynamic loads along with three-dimensional framing systems.

In this instance, the design calculations are done on an HP-67 programmable calculator, which has a magnetic card reader. The flow charts would work equally well with other programmable calculators. The series of subroutines are contained on three cards which are equivalent to approximately 672 program steps. The routines use 15 registers to retain input data and intermediate results. The various subroutines are identified by user-definable keys which are noted as A through E and a through e. The detailed operations are now illustrated by examples. The force analysis and the subsequent design analysis use the following sign convention: compressive axial loads are positive and counter-clockwise moments are positive. A summary of the input data and their storage locations is given in Table 1.

Table 1. Input Loading and Material Data

Storage Register	Input Data
A (20)	Axial force
B (21)	Moment at top (left)
C (22)	Moment at bottom (right)
D (23)	Yield stress
E (24)	Young's modulus
0	Effective length in plane of bending
1	Effective length normal to plane of bending
2	Cross-sectional area
3	Radius of gyration in plane of bending
4	Radius of gyration normal to plane of bending
5	Plastic section modulus
6	
7	Web thickness
8	Depth of section
9	Framing indicator 0 = unbraced 1 = braced

Example 1—Beam-Column Member



Design conditions:

a. Unbraced frame
b. Design forces due to dead load plus live load
c. A36 steel
d. $K = 1.0$
e. Major axis bending

Read Card 1.

Input Loading and Material Data:

A	B	C	D	E
43.12	1764.	1529.	36.00	29,000.

Try W12x40.

Input section properties:

0	1	2	3	4
144.	144.	11.8	5.13	1.94

5	6	7	8	9
57.5	0.	0.294	11.94	0.

$\boxed{c} \rightarrow L_{cr} = 122.6$ in. < 144 (unbraced in weak direction)

$\boxed{D} \rightarrow (d/t)_{max} = 58.91 > 40.6$ **o.k.**

$\boxed{d} \rightarrow (b/t)_{max} = 8.5 > 7.75$ **o.k.**

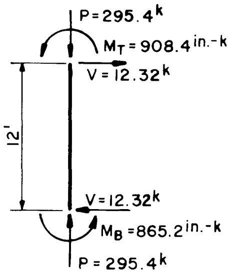
$\boxed{E} \rightarrow V_u = 69.51 > 22.87$ kips **o.k.**

Read Card 2.

$\boxed{a} \rightarrow Z_{req'd} = 52.33 < 57.5$ in.^{3} **o.k.**

Use W12x40.

Example 2—Beam-Column Member



Design conditions:

a. Unbraced frame
b. Design loads due to dead load + live load + wind
c. A36 steel
d. $K = 1.0$
e. Minor axis bending

Read Card 1.

Input loading and material data:

A	B	C	D	E
295.4	908.4	865.2	36.	29000.

Try W14x68.

Input section properties:

0	1	2	3	4
144.	144.	20.0	2.46	6.02

5	6	7	8	9
36.8	0.	0.418	14.06	0.

$\boxed{c} \rightarrow L_{cr} = 380.43$ in.

$\boxed{D} \rightarrow (d/t)_{max} = 42.83 > 33.6$ **o.k.**

$\boxed{d} \rightarrow (b/t)_{max} = 8.50 > 6.99$ **o.k.**

$\boxed{E} \rightarrow V_u = 116.37 > 12.32$ kips **o.k.**

Read Card 2.

$\boxed{a} \rightarrow Z_{req'd} = 51.54 > 36.8$ in.^{3} **n.g.**

Try W14x78.

Read Card 1.

Input section properties:

0	1	2	3	4
144.	144.	22.9	3.00	6.09

5	6	7	8	9
52.4	0.	0.428	14.06	0.

$\boxed{c} \rightarrow L_{cr} = 384.85$ in.

$\boxed{D} \rightarrow (d/t)_{max} = 42.83 > 32.9$ **o.k.**

$\boxed{d} \rightarrow (b/t)_{max} = 8.50 > 8.36$ **o.k.**

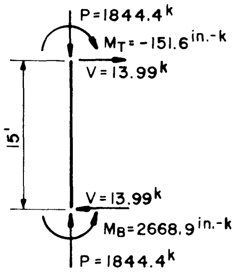
$\boxed{E} \rightarrow V_u = 119.15 > 12.32$ kips **o.k.**

Read Card 2.

$\boxed{a} \rightarrow Z_{req'd} = 40.54 < 52.4$ in.^{3} **o.k.**

Use W14x78.

Example 3— Beam-Column Member



Design conditions:

a. Braced frame
b. Design forces due to dead load + live load + earthquake
c. A36 steel
d. $K = 1.0$
e. Major axis bending

Read Card 1.

Input loading and material data:

A	B	C	D	E
+1844.4	−151.6	2668.9	36.	29,000.

Try W14x111.

Input section properties:

0	1	2	3	4
180.	180.	32.7	6.23	3.73

5	6	7	8	9
196.	0.	0.540	14.37	1.

$\boxed{c} \rightarrow L_{cr} = 235.72$ in.
$\boxed{D} \rightarrow (d/t)_{max} = 42.83 > 26.6$ **o.k.**
$\boxed{d} \rightarrow (b/t)_{max} = 8.50 > 8.37$ **o.k.**
$\boxed{E} \rightarrow V_u = 153.64 > 13.99$ kips **o.k.**

Read Card 2.
$\boxed{a} \rightarrow$ Blinking display = −0.67 (implies that $P > P_{cr}$ and $(1 - P/P_{cr}) < 0$ **n.g.**)

Try W14x211.

Read Card 1.

Input section properties:

0	1	2	3	4
180.	180.	62.1	6.56	4.07

5	6	7	8	9
391.	0.	0.980	15.75	1.

$\boxed{c} \rightarrow L_{cr} = 257.20$ in.
$\boxed{D} \rightarrow (d/t)_{max} = 42.83 > 16.1$ **o.k.**
$\boxed{d} \rightarrow (b/t)_{max} = 8.50 > 5.05$ **o.k.**
$\boxed{E} \rightarrow V_u = 305.61 > 13.99$ kips **o.k.**

Read Card 2.
$\boxed{a} \rightarrow Z_{req'd} = 686.62 > 391$ in.^{3} **n.g.**

Try W14x228.

Read Card 1.

Input section properties:

0	1	2	3	4
180.	180.	67.1	6.62	4.10

5	6	7	8	9
427.	0.	1.045	16.00	1.

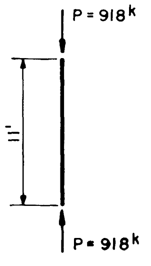
$\boxed{c} \rightarrow L_{cr} = 259.10$
$\boxed{D} \rightarrow (d/t)_{max} = 42.83 > 15.3$ **o.k.**
$\boxed{d} \rightarrow (b/t)_{max} = 8.50 > 4.70$ **o.k.**
$\boxed{E} \rightarrow V_u = 331.06 > 13.99$ **o.k.**

Read Card 2.

$\boxed{A} \rightarrow Z_{req'd} = 346.93 < 427$ in.^{3} **o.k.**

Use **W14x228.**

Example 4—Bracing Member



Design conditions:
a. $K_y = 1.0$; $K_x = 2.0$
b. Design forces due to dead load + live load
c. A36 steel

$KL_x = 2 \times 11 \times 12 = 264$ in.
$KL_y = 1 \times 11 \times 12 = 132$ in.

Read Card 3.

Input Loading and Material Data:

A	B	C	D	E
918.	—	—	36.0	29,000.

Try W12x106.

Input section properties:

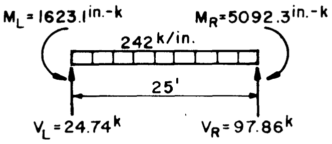
0	1	2	3	4
264.	132.	31.2	5.46	311.

5	6	7	8	9
—	—	—	—	—

$\boxed{C} \rightarrow P_{cr} = 954.72 > 918$ kips **o.k.**

Use W12x106.

Example 5—Beam Member



Design Conditions:
a. Unbraced frame
b. Design forces due to dead load + live load + wind
c. A36 steel

Read Card 1.

Input loading and material data:

A	B	C	D	E
—	1623.1	−5092.3	36	29,000.

$\boxed{C} \rightarrow Z_{req'd} = 141.45$ in.^{3}

Try W24x61 ($Z = 152$ in.^{3})

Input section properties:

0	1	2	3	4
300.	300.	18.0	9.25	1.38

5	6	7	8	9
152.	0.	.419	23.72	0.

$\boxed{c} \rightarrow L_{cr} = 87.21 \text{ in.} = 7.27 \text{ ft} \therefore$ brace @ $\frac{1}{4}$ points
$\boxed{D} \rightarrow (d/t)_{max} = 68.67 > 56.6$ **o.k.**
$\boxed{d} \rightarrow (b/t)_{max} = 8.5 > 5.94$ **o.k.**
$\boxed{E} \rightarrow V_u = 196.79 > 47.86$ kips **o.k.**

Use W24x61.

CONCLUSIONS

This paper has illustrated the use of a hand-held programmable calculator for the strength design of steel frames. Although the process is only semi-automatic, it still saves the designer from having to perform the rather mundane calculations involved with member selection and checking. Its main attraction is the low cost and ready availability of the calculators, plus the fact that the calculation can be done at the designer's desk without extensive data preparation. The procedures assume that the design forces are known. Therefore, a force analysis will have to be done on a larger computer system. It should also be noted that the use of this type of design procedure allows complete interaction of the engineer with this design process. Therefore, engineering judgment decisions can be made and incorporated in the design as it progresses.

The computer industry is currently witnessing the rapid introduction of new products in the small computer area. These include programmable calculators, microprocessors, personal computers, and mini-computers. This is tending to reduce the cost such that more and more of this equipment will be finding its way into design offices. Small computers will have a tremendous effect on the design process. What is required are some standardized design procedures which can be readily adapted to a particular computer and bridge the gap between large computer systems and design computations.

The design procedures developed in this study will be available on three magnetic cards through the Hewlett Packard User's Library in Corvallis, Oregon.

REFERENCES

1. *American Institute of Steel Construction* Manual of Steel Construction *Seventh Edition, New York, N.Y.*
2. *Wilson, E. L., J. P. Hollings, and H. H. Dovey* Three-Dimensional Analysis of Building Systems (Extended Version) *Earthquake Engineering Research Center, EERC 75-13, Berkeley, California, 1975.*

APPENDIX A

(*See following pages.*)

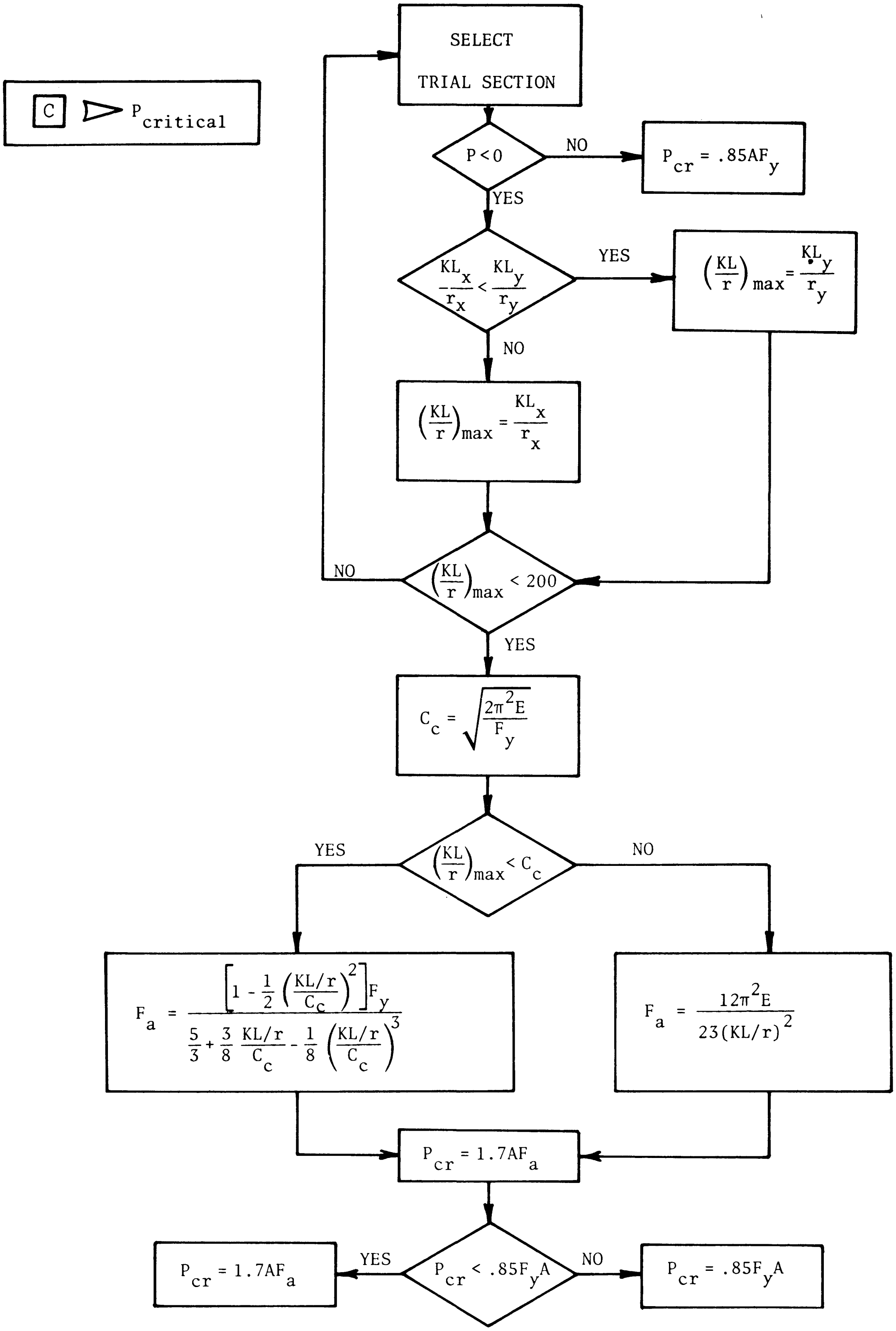

C
P critical
SELECT TRIAL SECTION
$P < 0$
NO
$P_{cr} = .85AF_y$
YES
$\frac{KL_x}{r_x} < \frac{KL_y}{r_y}$
YES
$\left(\frac{KL}{r}\right)_{max} = \frac{KL_y}{r_y}$
NO
$\left(\frac{KL}{r}\right)_{max} = \frac{KL_x}{r_x}$
NO
$\left(\frac{KL}{r}\right)_{max} < 200$
YES
$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$
YES
$\left(\frac{KL}{r}\right)_{max} < C_c$
NO
$F_a = \frac{\left[1 - \frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]F_y}{\frac{5}{3} + \frac{3}{8}\frac{KL/r}{C_c} - \frac{1}{8}\left(\frac{KL/r}{C_c}\right)^3}$
$F_a = \frac{12\pi^2 E}{23(KL/r)^2}$
$P_{cr} = 1.7AF_a$
$P_{cr} = 1.7AF_a$
YES
$P_{cr} < .85F_yA$
NO
$P_{cr} = .85F_yA$

Chart 1

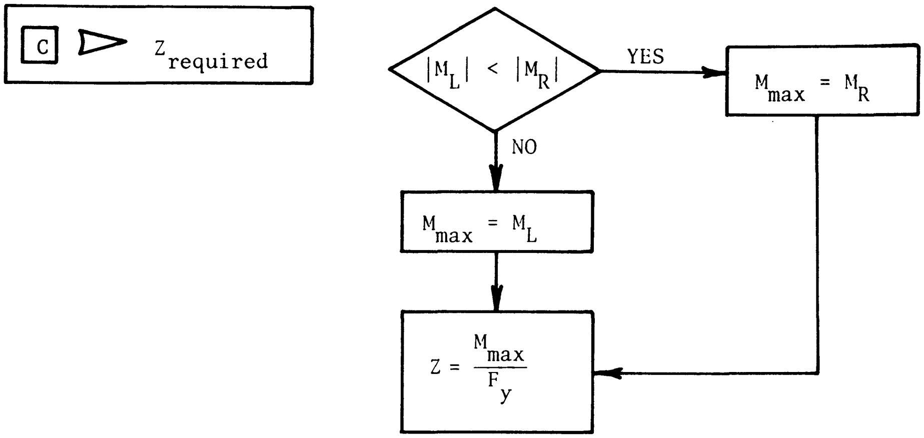


Chart 2

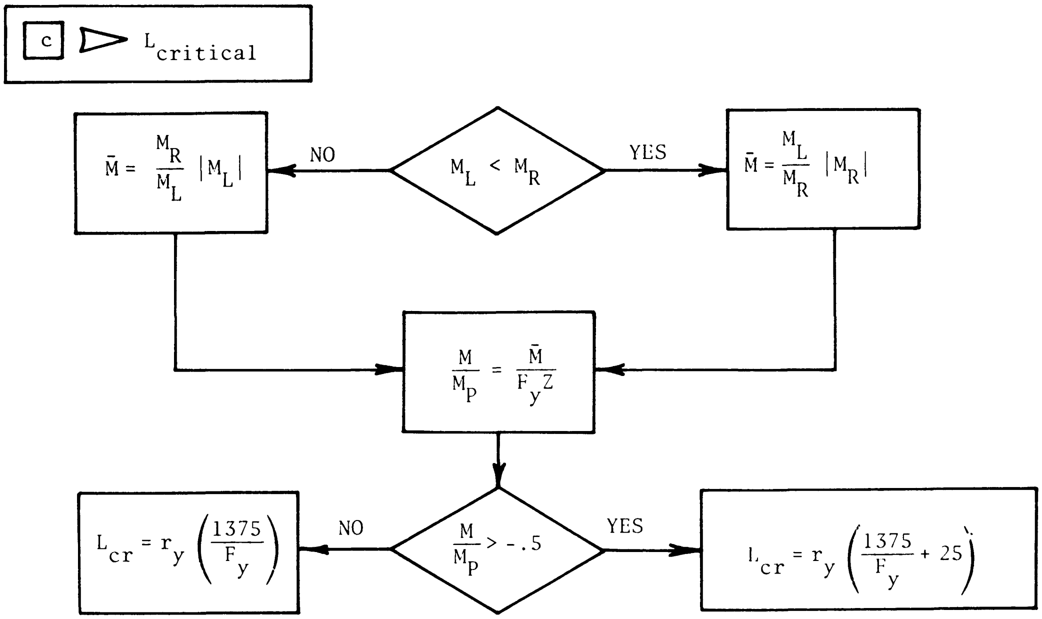


Chart 3

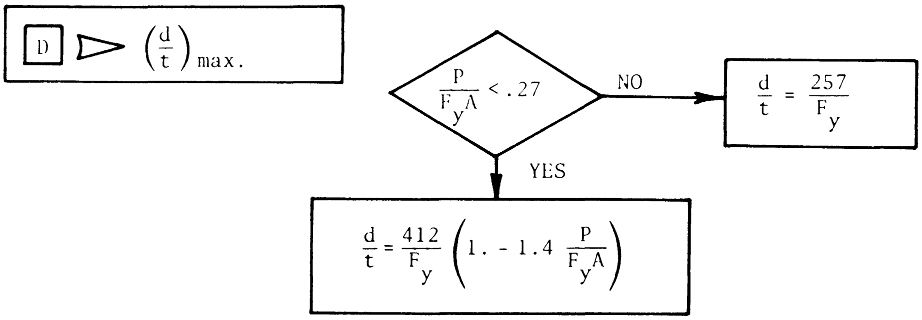


Chart 4

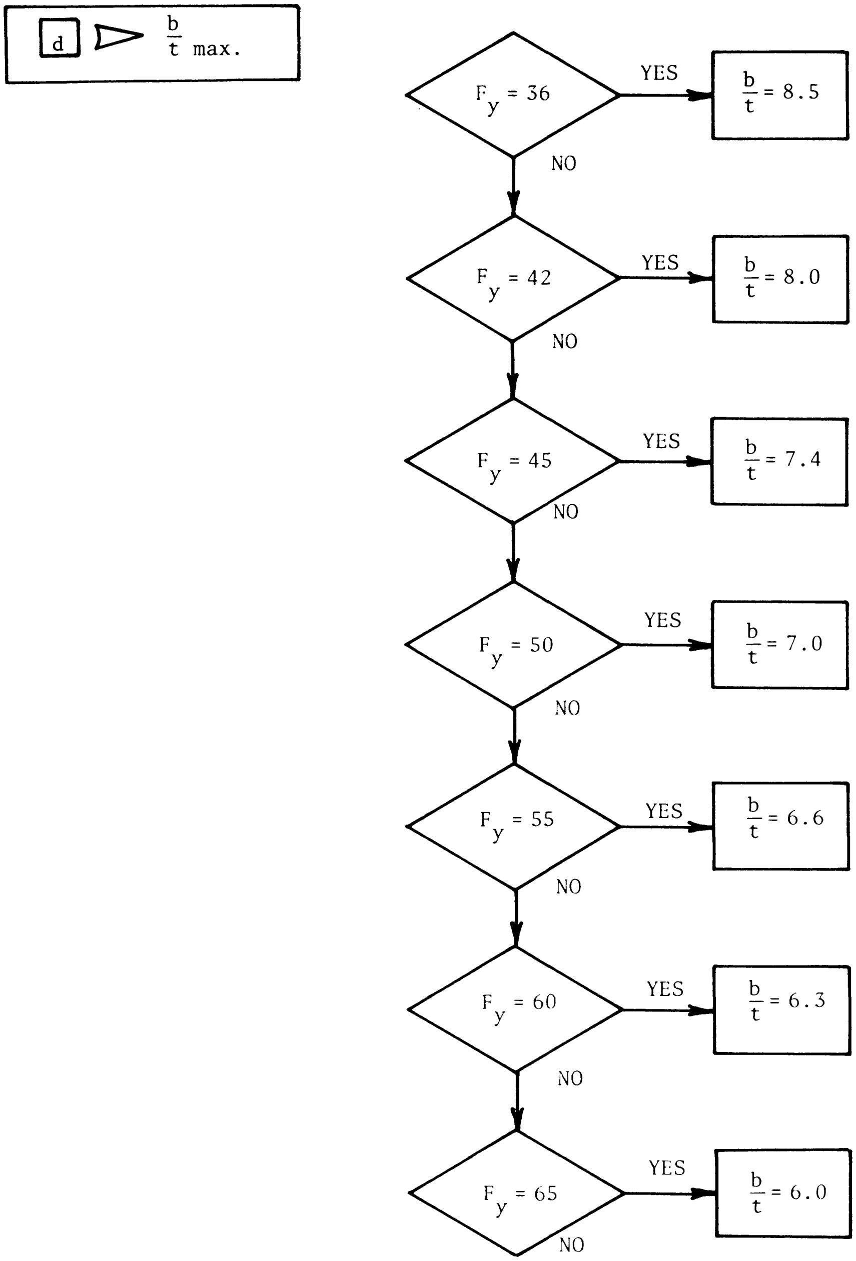
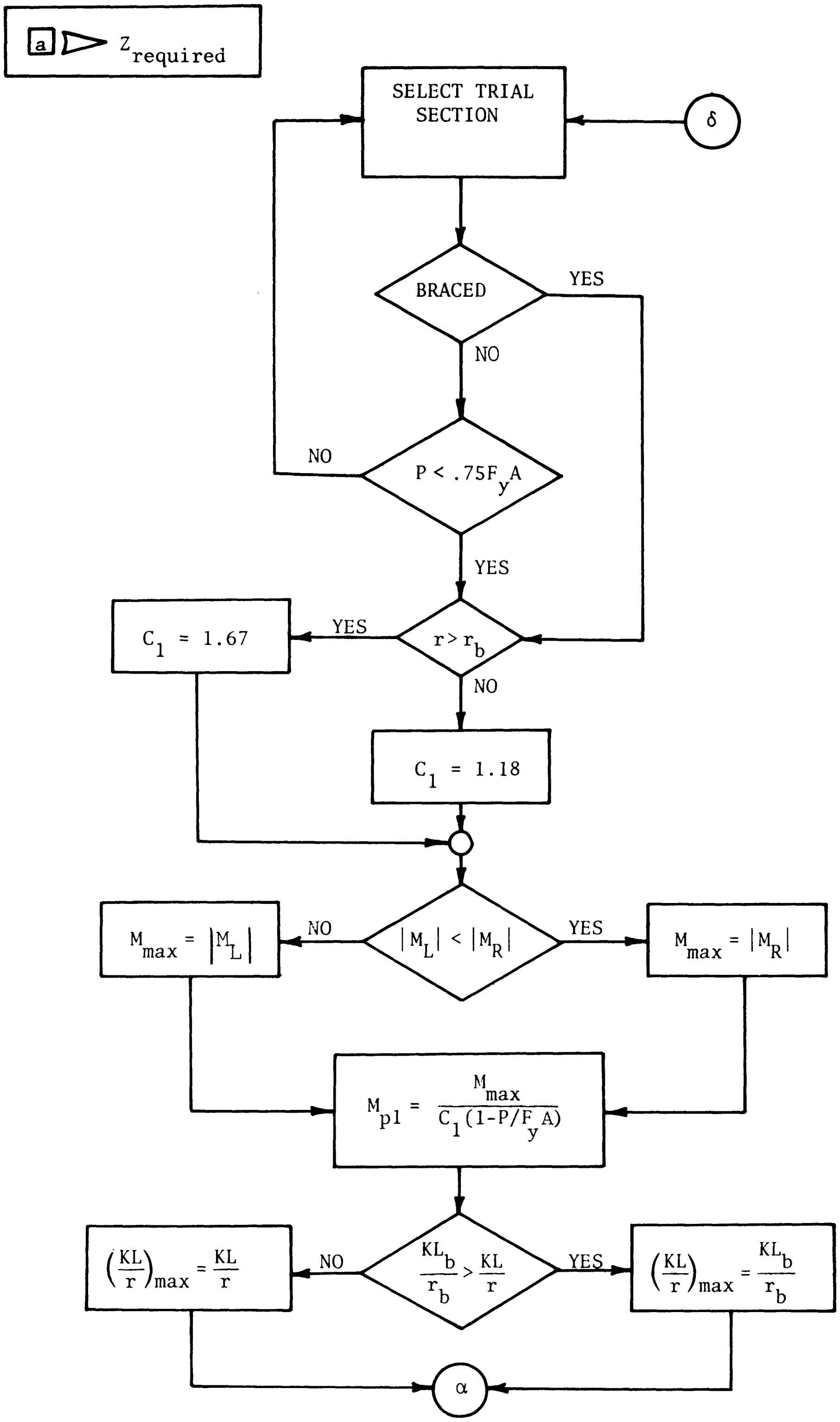

d
b/t max.
F_y = 36
YES
b/t = 8.5
NO
F_y = 42
YES
b/t = 8.0
NO
F_y = 45
YES
b/t = 7.4
NO
F_y = 50
YES
b/t = 7.0
NO
F_y = 55
YES
b/t = 6.6
NO
F_y = 60
YES
b/t = 6.3
NO
F_y = 65
YES
b/t = 6.0
NO

Chart 5



a
$Z_{required}$
SELECT TRIAL SECTION
δ
BRACED
YES
NO
NO
$P < .75F_yA$
YES
YES
$C_1 = 1.67$
$r > r_b$
NO
$C_1 = 1.18$
NO
$M_{max} = |M_L|$
$|M_L| < |M_R|$
YES
$M_{max} = |M_R|$
$M_{pl} = \frac{M_{max}}{C_1(1-P/F_yA)}$
$\left(\frac{KL}{r}\right)_{max} = \frac{KL}{r}$
NO
$\frac{KL_b}{r_b} > \frac{KL}{r}$
YES
$\left(\frac{KL}{r}\right)_{max} = \frac{KL_b}{r_b}$
α

Chart 6

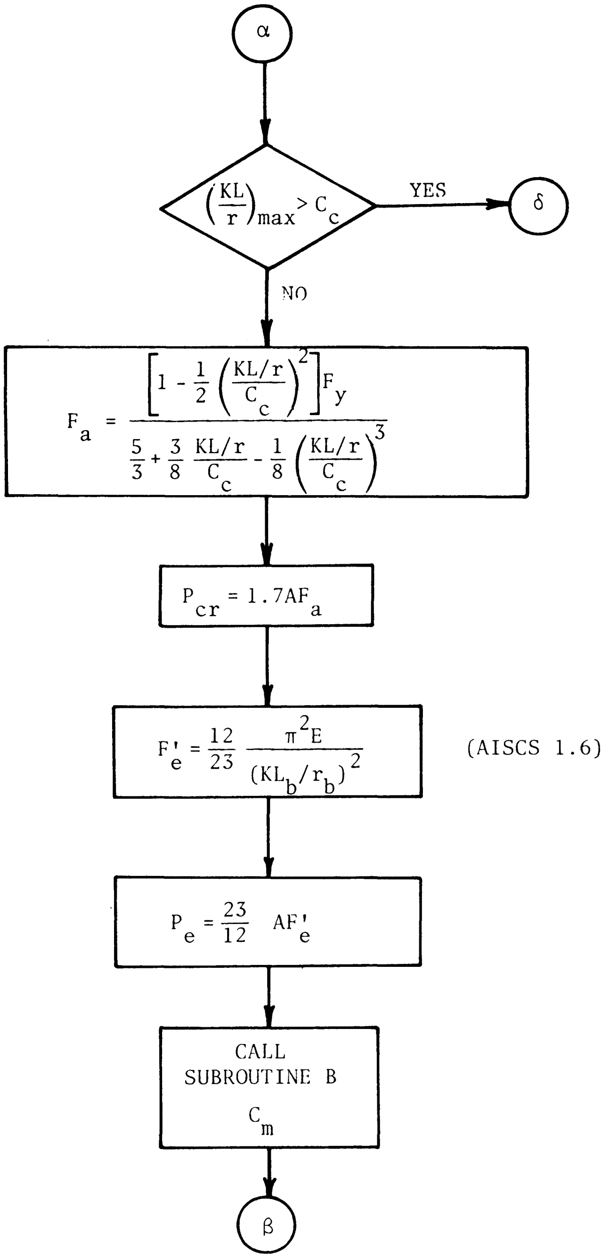


Chart 6 (cont'd)

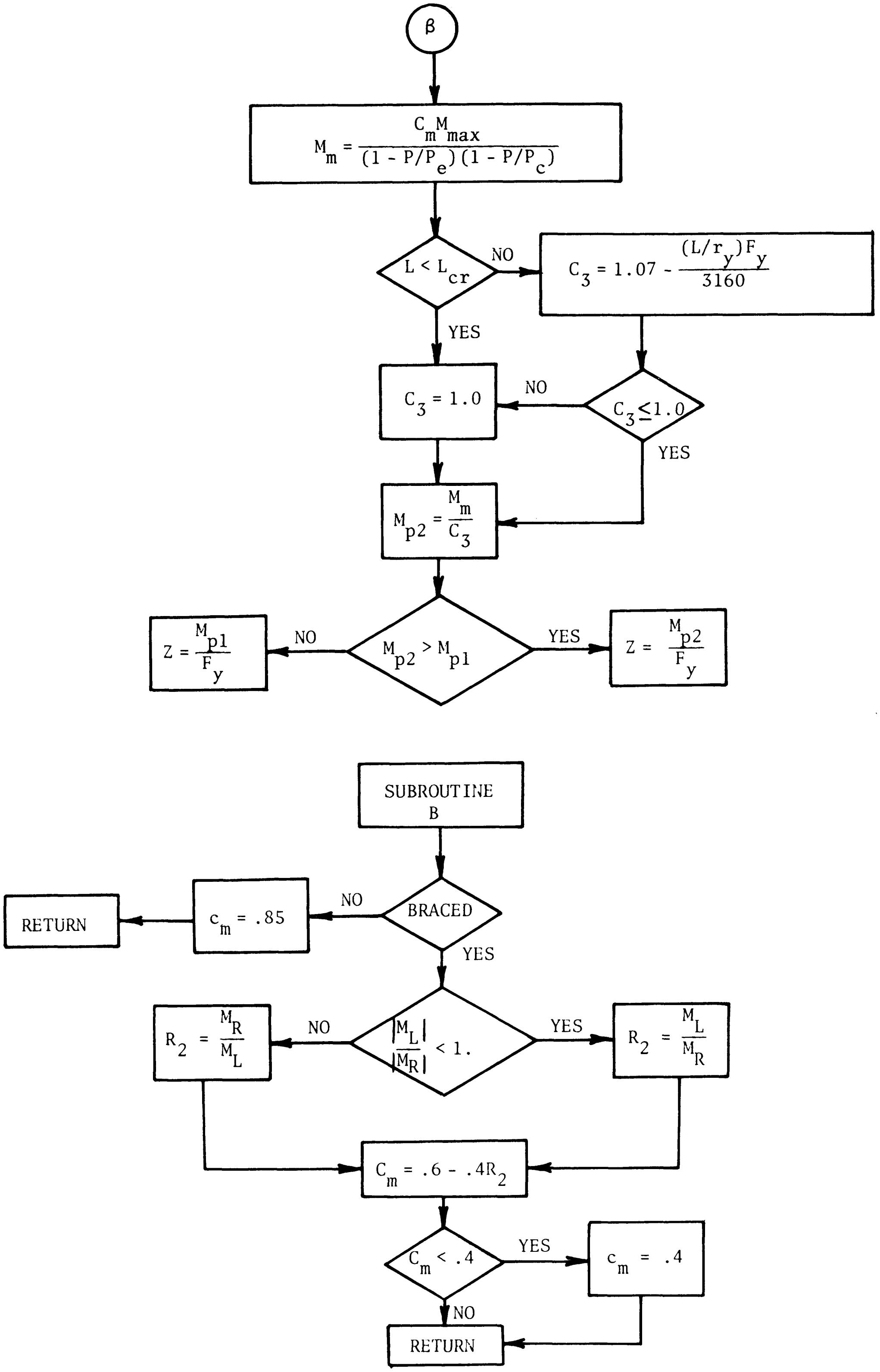


Chart 6 (cont'd)