Selection of a "Trial" Column Section

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A MEMBER SUBJECTED to both axial and bending loads must comply with the AISC interaction equations (1.6-1a), (1.6-1b), and (1.6-2). The normal procedure for designing a column so loaded is to select a trial section and check it for compliance with one or more of the interaction equations. Since the checking is an arduous task, it is desirable that the number of checks of trial sections be held to a minimum. The process can be extremely discouraging when designing a structural system where preliminary beams and columns must be sequentially selected and tried several times, as in a rigid framed tiered structure.

A method is presented which allows a column section to be selected quickly and in a direct manner. The selection will always be a practical one (approaches closely the value of 1.0 in the interaction equations) and usually is the most economical. It may be used for preliminary design without further checking by the column equations. The members of the final structural system, however, should be formally checked by the interaction equations for the record computations.

METHOD

The method utilizes the following tables and the tables in the AISC Manual[1] for "Allowable Concentric Loads on Columns." As in most methods of selecting a trial section, an equivalent axial load is used. It is arrived at by successive approximation. The equation for the effective axial load is:

$$P_{eff} = P_0 + M_{xm} + M_y m U$$

where

- P_0 = actual axial load, kips
- M_x = bending moment about the strong axis, kip-ft
- M_y = bending moment about the weak axis, kip-ft
- m = factor taken from Table 1
- U = factor taken from Table 2

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The procedure is as follows:

1. With the known value of KL (effective length), select a first approximate value of m from Table 1. Let U equal 3.
2. Solve for P_{eff}.
3. From the AISC tables for "Allowable Concentric Loads on Columns", select a tentative section to support P_{eff}.
4. Based on the section selected in Step 3, select a "subsequent approximate" value of m from Table 1 and a U value from Table 2.
5. With the values selected in Step 4, solve for P_{eff}.
6. Repeat Steps 3 and 4 until the values of m and U stabilize.

Table 1. Values of m*

KL (ft)	10	12	14	16	18	20	22 and over
F_y = 36 ksi							
1st Approx.	2.4	2.3	2.2	2.2	2.1	2.0	1.9
Subsequent Approx. W8	3.0	2.9	2.8	2.6	2.3	2.0	2.0
W10	2.6	2.5	2.5	2.4	2.3	2.1	2.0
W12	2.1	2.1	2.0	2.0	2.0	2.0	2.0
W14	1.8	1.7	1.7	1.7	1.7	1.7	1.7
F_y = 50 ksi							
1st Approx.	2.4	2.3	2.2	2.0	1.9	1.8	1.7
Subsequent Approx. W8	3.0	2.8	2.5	2.2	1.9	1.6	1.6
W10	2.5	2.5	2.4	2.3	2.1	1.9	1.7
W12	2.0	2.0	2.0	1.9	1.9	1.8	1.7
W14	1.8	1.7	1.7	1.7	1.7	1.7	1.7

* *Values of m are for C_m = 0.85. When C_m is other than 0.85, multiply the tabular value of m by $C_m/0.85$.*

Table 2. Values of U

Nominal Size	Weight per Foot (lbs)	U F_y = 36 ksi	U F_y = 50 ksi
W14	730	2.14	2.14
(14 × 16)	665	2.15	2.15
	605	2.16	2.16
	550	2.17	2.17
	500	2.18	2.18
	455	2.19	2.19
	426	2.20	2.20
	398	2.21	2.21
	370	2.22	2.22
	342	2.23	2.23
	320	2.22	2.22
	314	2.24	2.24
	287	2.25	2.25
	264	2.26	2.26
	246	2.27	2.27
	237	2.28	2.28
	228	2.28	2.28
	219	2.29	2.29
	211	2.29	2.29
	202	2.30	2.30
	193	2.30	2.30
	184	2.31	2.31
	176	2.32	2.32
	167	2.32	2.32
	158	2.33	2.33
	150	2.33	2.33
	142	2.34	2.34
W14	136	2.47	2.47
(14 × 14½)	127	2.47	2.47
	119	2.48	3.09
	111	2.49	3.07
	103	3.11	3.06
	95	3.10	3.03
	87	3.08	3.01
W14	84	3.07	3.82
(14 × 12)	78	3.09	3.81
W14	74	3.73	3.73
(14 × 10)	68	3.75	3.75
	61	3.78	4.71
W14	53	4.79	4.79
(14 × 8)	48	4.84	4.84
	43	4.89	6.10

Nominal Size	Weight per Foot (lbs)	U F_y = 36 ksi	U F_y = 50 ksi
W12	190	2.49	2.49
(12 × 12)	161	2.52	2.52
	133	2.55	2.55
	120	2.57	2.57
	106	2.58	2.58
	99	2.60	2.60
	92	2.61	2.61
	85	2.62	3.26
	79	2.63	3.25
	72	3.30	3.23
	65	3.28	3.21
W12	58	3.21	3.99
(12 × 10)	53	3.24	3.97
W12	50	4.08	4.08
(12 × 8)	45	4.12	4.12
	40	4.15	5.16
W10	112	2.46	2.46
(10 × 10)	100	2.48	2.48
	89	2.50	2.50
	77	2.52	2.52
	72	2 53	2.53
	66	2.54	2.54
	60	2.55	2.55
	54	2.56	3.17
	49	3.22	3.15
W10	45	3.26	3.26
(10 × 8)	39	3.30	4.12
	33	4.18	4.10
W8	67	2.48	2.48
(8 × 8)	58	2.51	2.51
	48	2.53	2.53
	40	2.57	2.57
	35	2.59	3.20
	31	3.24	3.18
W8	28	3.23	3.23
(8 × 6½)	24	3.26	4.04
W8	20	4.28	4.28
(8 × 5½)	17	4.40	5.41

The procedure is demonstrated in the following examples.

EXAMPLE 1

Given:

P_0 = 350 kips
M_x = 300 kip-ft
KL = 16 ft
F_y = 36 ksi
C_m = 0.85

Solution:

Step 1. From Table 1, first trial, select m = 2.2.

Step 2. $P_{eff} = P_0 + M_x m$
$= 350 + 300\ (2.2)$
$= 350 + 660 = 1010$ kips

Step 3. From "Allowable Concentric Loads on Columns," for KL = 16 ft and P = 1010 kips, select W14×193.

Step 4. Select a "second trial" value of m = 1.7 from Table 1 for a 14-in. section.

Step 5. $P_{eff} = 350 + 300(1.7)$
$= 350 + 510 = 860$ kips

Step 3. (repeat) Select W14×158

Step 4. (repeat) $m = 1.7$, stabilized

Use: W14×158

The sums of the terms of Eqs. (1.6-1a) and (1.6-1b) of the AISC Specification equal 0.967 and 0.943, respectively, which is satisfactory.

EXAMPLE 2

Given:

$P_0 = 200$ kips
$M_x = 120$ kips
$M_y = 40$ kips
$KL = 14$ ft
$F_y = 36$ ksi
$C_m = 0.85$

Solution:

Step 1. For $KL = 14$ ft, from Table 1 select a "first trial" value of $m = 2.2$. Let $U = 3.0$.

Step 2. $P_{eff} = P_0 + M_x m + M_y m U$
$= 200 + 120(2.2) + 40(2.2)(3.0)$
$= 200 + 264 + 88(3.0) = 728$ kips

Step 3. From column tables, select W14×136.

Step 4. Select a "second trial" value of $m = 1.7$ from Table 1 and $U = 2.47$ from Table 2.

Step 5. $P_{eff} = 200 + 120(1.7) + 40(1.7)(2.47)$
$= 200 + 204 + 168 = 572$ kips

Step 3. (repeat) Select W14×111.

Step 4. (repeat) $m = 1.7$, $U = 2.49$.

Step 5. (repeat) $P_{eff} = 200 + 120(1.7) + 40(1.7)(2.49) = 571$ kips

Step 3. (repeat) Select W14×111.

Use: W14×111.

COMMENTARY

The capacity of a column section of given length may be computed for varying bending moments and axial loads and a curve plotted. A typical curve is shown in Fig. 1 for a W12×79 having a length of 16 ft.[2] The section is capable of carrying any combination of axial load and bending moment that intersect below the curve. For example, an axial load of 200 kips and bending moment of 100 kip-ft are shown to be within the capabilities of the section. In fact, any combination of axial load and bending moment that falls within the shaded area would economically dictate the selection of a W12×79 section. The curves plotted in Ref. 2 afford visibility of the range of column sections.

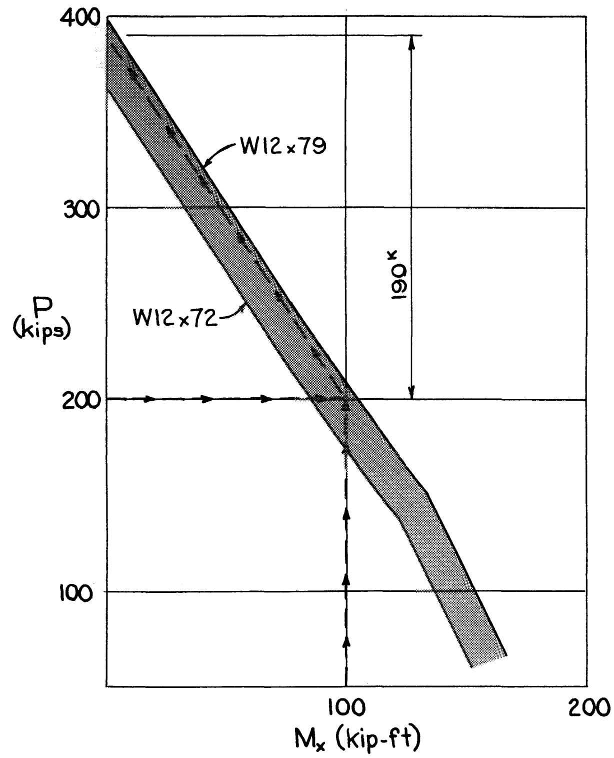

Figure 1

Note that if a line is drawn approximately parallel with the curve, it will intersect the left edge (where bending is zero). The effect of the bending might then be equated to an additional axial load of 190 kips as shown in Fig. 1. For the purpose of selecting a trial section, the percentage error is relatively not sensitive.

Values of the slope, axial load ÷ bending moment, can be approximated for groups of columns of different lengths. This value multiplied by the moment gives an equivalent axial load to be added to the actual axial load.

Using nominal values for each group of columns (8-in., 10-in., 12-in., 4-in.), the group of curves in Fig. 2 was plotted. The "first trial m" is represented by the heavy dashed line. "Second trial" values of m, listed in Table I, are taken from the curves. Precise values of m for selecting a section are not warranted.

The values of U, $F_{bx}S_x/F_{by}S_y$, are given in Table I for various column sections.[2] Using a value of 3 for the first trial of U will result in a quick convergence to the desired section.

REFERENCES

1. Manual of Steel Construction, 7th Edition *American Institute of Steel Construction, New York, N. Y.*
2. Column Design Curves *United States Steel Corporation, Pittsburgh, Pa., Pub. No. ADUSS 27-3065-01, 1969.*

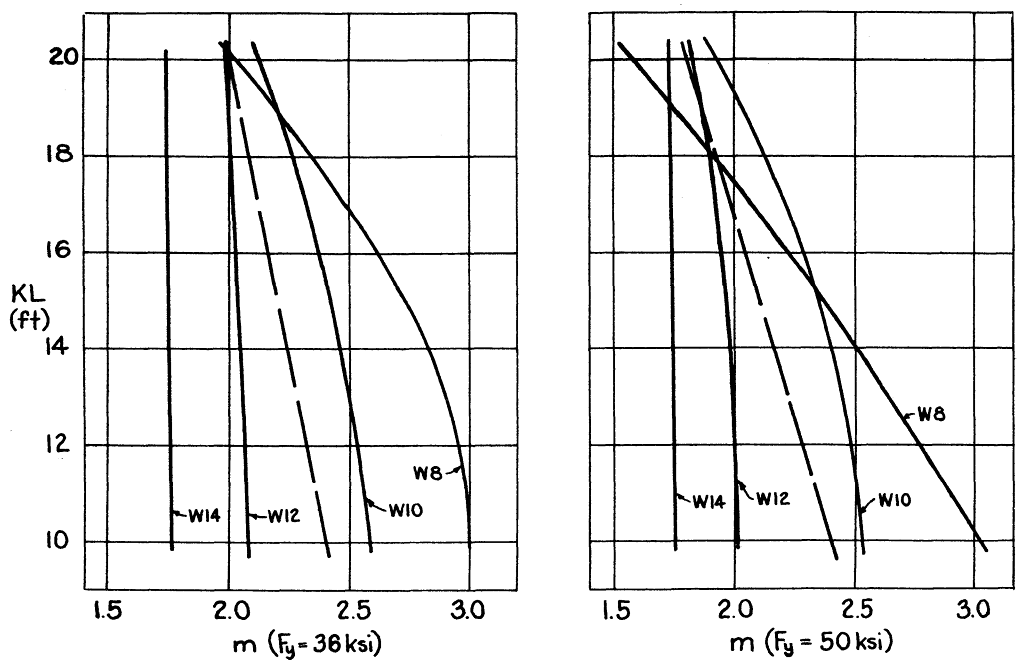
20
18
16
KL (ft)
14
12
10
W14
W12
W10
W8
1.5
2.0
2.5
3.0
m (Fy = 36 ksi)
W14
W12
W10
W8
1.5
2.0
2.5
3.0
m (Fy = 50 ksi)

Figure 2