Review of old Cb Equation

1. $C_b = 1.00$ for braced Cantilever
2. $C_b = 1.00$ for a simple beam with bracining at End Points. $(M_{max} > M_2)$
3. How Can we Find Cb value For Continuous beams at intermediate braced supports?
4. Solved Problem For Cb Value.
5. Cb recent Equation → FE Hand book
   → Table 3-1 AISC-360

Page 3-4

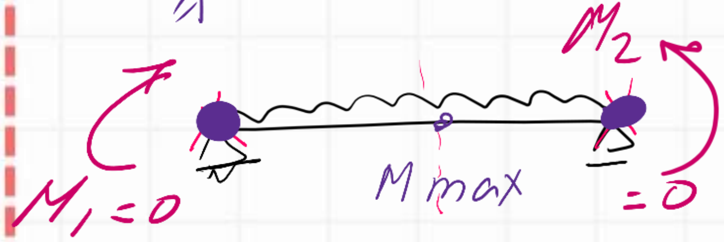
CM # 15

Cantilever

$C_b = 1$

Unbraced Cantilever

Old Cb Formula



$Cb = 1$

moment for members where moment within part middle $= \frac{wL^2}{8} > M_2$

where M_1 is the smaller and M_2 is the larger end moment for the unbraced segment of the beam under consideration. If the rotations due to end moments M_1 and M_2 are in opposite directions, then M_1/M_2 is negative; otherwise, M_1/M_2 is positive. Coefficient $C_b = 1.0$ for unbraced cantilevers and for members where the moment within part of the unbraced segment is greater than or equal to the larger segment end moment (e.g., simply supported beams, where $M_1 = M_2 = 0$).

$$C_b = \left[1.75 + 1.05\frac{M_1}{M_2} + 0.3\left(\frac{M_1}{M_2}\right)^2\right] \leq 2.3$$

$Cb = 1.0 \Rightarrow$ we have old $M_{max} > M_2$ $\xrightarrow[M_2 = 0]{M_1 = 0}$ $Cb = 1$

Beam is braced at third points

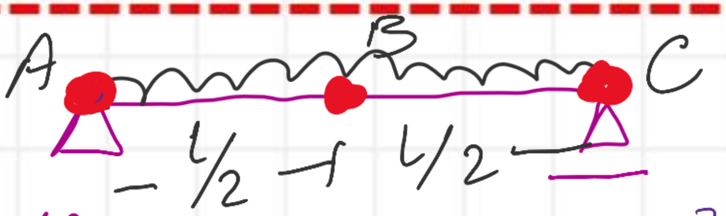
$$C_b = \left[1.75 + 1.05\frac{M_1}{M_2} + 0.3\left(\frac{M_1}{M_2}\right)^2\right] \le 2.3$$

yold

$$C_b = \left[1.75 + 1.05\left(\frac{0}{\frac{wL^2}{8}}\right) + 0.3(0)^2 \le 2.3\right.$$

$C_b = 1.75 < 2.30$ OK

A-B & B-C

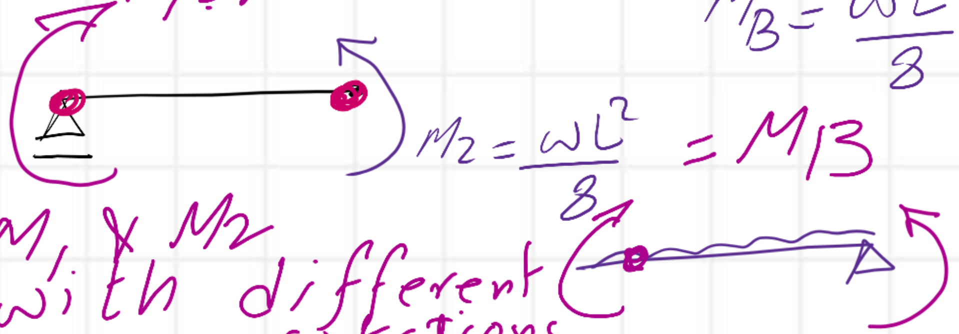


$M_1 = M_A$

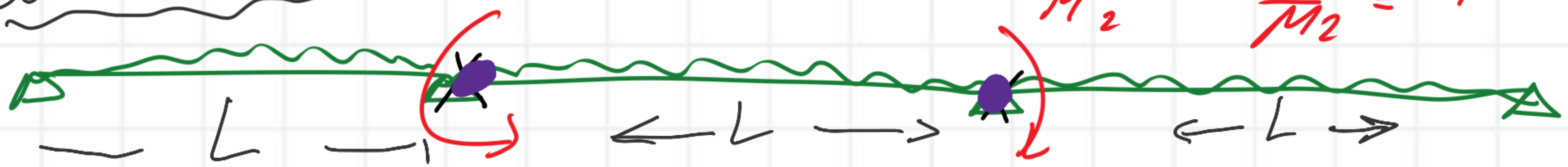
$M_B = \frac{wL^2}{8}$

$M_2 = \frac{wL^2}{8} = M_B$

M_1 & M_2 with different rotations



Continuos beams



$\frac{M_1}{M_2} = -1$

For $M_1 = M_2$ opposite rotations

$C_b = 1.75 + 1.05(-1) + 0.3(-1)^2 = 1.75 - 1.05 + 0.30 = 1.00$

< 2.30

$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \le 2.30$

Prepared by Eng.Maged Kamel.

schaum

5.11. Determine C_b for the span of the continuous beam shown in Fig. 5-12.

(a) Lateral braces are provided only at the supports.

(b) Lateral braces are provided at midspan and the supports.

Cb based on old formula

Case (a) $M_1 < M_2$

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2$$

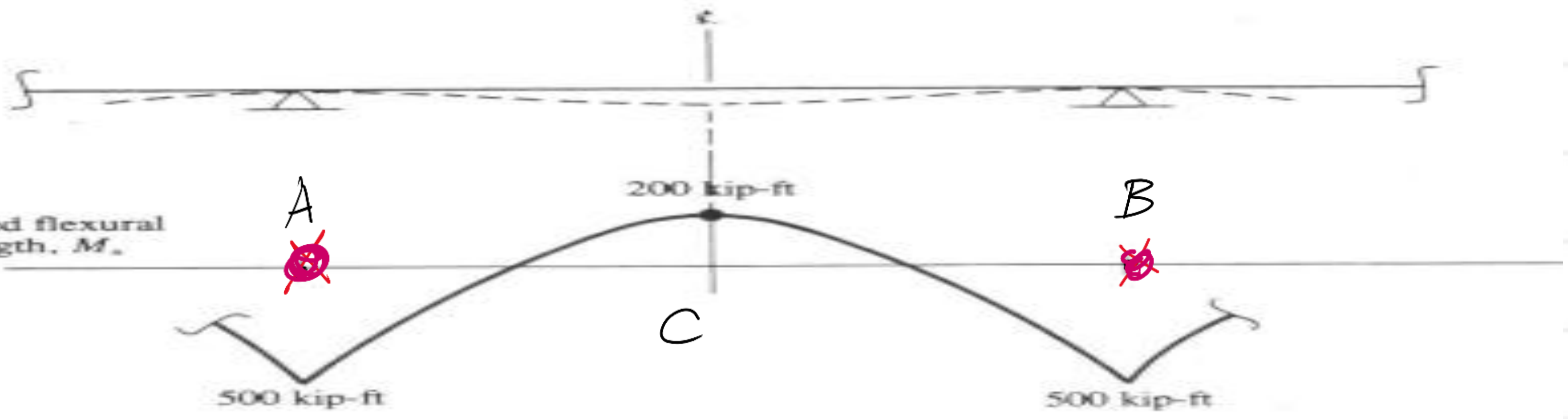


Fig. 5-12

M_1 M_2

Case (a): bracing at A, B only

$\frac{M_1}{M_2} = -ve$ $M_1 = M_A = 500$ Kips-Ft

$M_b = 500$ kips ft

Rotation in opposite direction

$C_b = 1.75 + 1.05(-1) + 0.3(-1)^2 = 0.7 + 0.3 = 1.00$

Prepared by Eng.Maged Kamel.

schaum

5.11. Determine C_b for the span of the continuous beam shown in Fig. 5-12.

(*a*) Lateral braces are provided only at the supports.

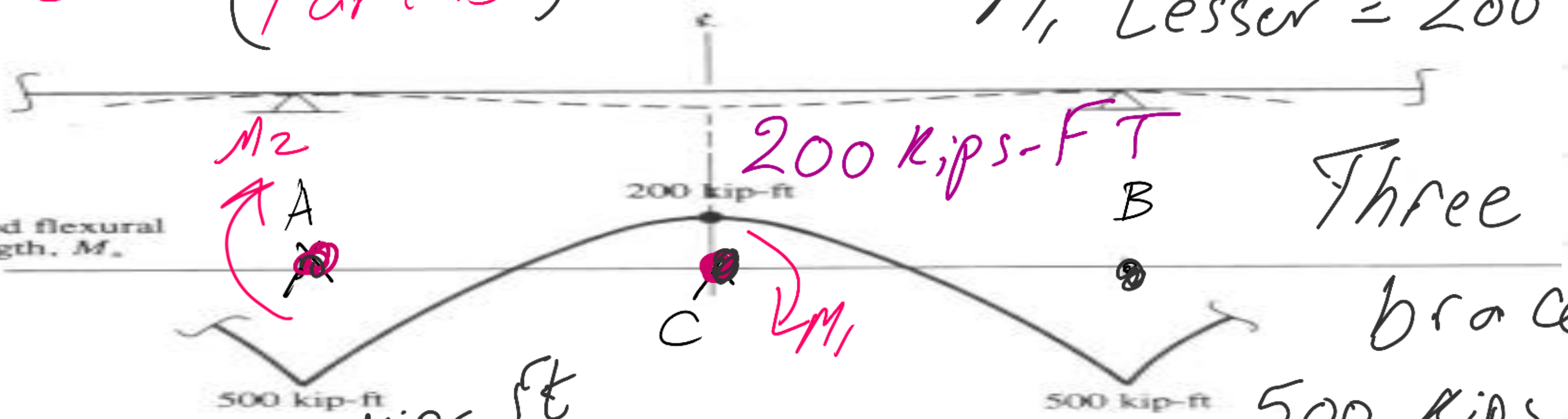
(*b*) Lateral braces are provided at midspan and the supports.

Cb based on old formula

$M_1 < M_2$ (Part b)

M_1 Lesser = 200 kips-FT

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2$$



500 kips-ft

Fig. 5-12

Three braces

500 kips-FT

Part A-C

$M_1 = M_C = +200$ kip-ft

$M_2 = M_A = 500$ kip-ft

$\frac{M_1}{M_2} = \frac{200}{500} = 0.40$

Rotation is in the same direction +ve

$C_b = 1.75 + 1.05(+0.4) + 0.3(+0.4)^2$

$C_b = 1.75 + 0.42 + 0.048 = 2.22 < 2.30$

Prepared by Eng.Maged Kamel.

Civil Engineering

Page: 281

FE-Hand book

10.40

Design of Steel Components

(ANSI/AISC 360-16) LRFD, ASD E = 29,000 ksi

Beams

For doubly symmetric compact I-shaped members bent about their major axis, the *design flexural strength* $\phi_b M_n$ is determined with $\phi_b = 0.90$ and $\Omega = 1.67$ as follows:

Yielding

$$M_n = M_p = F_y Z_x$$

$$M_n = M_p = = F_y * Z_X$$

where

F_y = specified minimum yield stress

Z_x = plastic section modulus about the x-axis

Lateral-Torsional Buckling

Based on bracing where L_b is the length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section with respect to the length limits L_p and L_r:

When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

where

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

$$Cb = (12.50\, M_{max} / 2.5M_{max} + 3M_A + 4M_B + 3M_C)$$

M_{max} = absolute value of maximum moment in the unbraced segment

M_A = absolute value of maximum moment at quarter point of the unbraced segment

M_B = absolute value of maximum moment at centerline of the unbraced segment

M_C = absolute value of maximum moment at three-quarter of the unbraced segment

VALUES OF C_b FOR SIMPLY SUPPORTED BEAMS		
LOAD	LATERAL BRACING ALONG SPAN	C_b
P	NONE LOAD AT MIDPOINT	1.32
	AT LOAD POINT	1.67 1.67
P P	NONE LOADS AT THIRD POINTS	1.14
	AT LOAD POINTS LOADS SYMMETRICALLY PLACED	1.67 1.00 1.67
P P P	NONE LOADS AT QUARTER POINTS	1.14
	AT LOAD POINTS LOADS AT QUARTER POINTS	1.67 1.11 1.11 1.67
w	NONE	1.14
	AT MIDPOINT	1.30 1.30
	AT THIRD POINTS	1.45 1.01 1.45
	AT QUARTER POINTS	1.52 1.06 1.06 1.52
	AT FIFTH POINTS	1.56 1.12 1.00 1.12 1.56
NOTE: LATERAL BRACING MUST ALWAYS BE PROVIDED AT POINTS OF SUPPORT PER AISC SPECIFICATION CHAPTER F.		

Source: Adapted from *Steel Construction Manual*, 14th ed., AISC, 2011.

FE- Handbook
10.40

Civil Engineering
Pg- 282

VALUES OF C_b FOR SIMPLY SUPPORTED BEAMS		
LOAD	LATERAL BRACING ALONG SPAN	C_b
P	NONE LOAD AT MIDPOINT	1.32
	AT LOAD POINT	1.67 1.67
P P	NONE LOADS AT THIRD POINTS	1.14
	AT LOAD POINTS LOADS SYMMETRICALLY PLACED	1.67 1.00 1.67
P P P	NONE LOADS AT QUARTER POINTS	1.14
	AT LOAD POINTS LOADS AT QUARTER POINTS	1.67 1.11 1.11 1.67
w	NONE	1.14
	AT MIDPOINT	1.30 1.30
	AT THIRD POINTS	1.45 1.01 1.45
	AT QUARTER POINTS	1.52 1.06 1.06 1.52
	AT FIFTH POINTS	1.56 1.12 1.00 1.12 1.56

NOTE: LATERAL BRACING MUST ALWAYS BE PROVIDED AT POINTS OF SUPPORT PER AISC SPECIFICATION CHAPTER F.

Adapted from *Steel Construction Manual*, 14th ed., AISC, 2011.

Table 3-1
Page 3-18

Prepared by Eng.Maged Kamel.

AISC - 360-16
CM # 15

Page 3-4

CM#15

AISC-360 -16

FLEXURAL STRENGTH

The nominal flexural strength of W-shapes is illustrated as a function of the unbraced length, L_b, in Figure 3-1. The available strength is determined as ϕM_n or M_n/Ω, which must equal or exceed the required strength (bending moment), M_u or M_a, respectively. The available flexural strength, ϕM_n or M_n/Ω, is determined per AISC *Specification* Chapter F. Table User Note F1.1 outlines the sections of Chapter F and the corresponding limit states applicable to each member type.

Braced, Compact Flexural Members

When flexural members are braced ($L_b \le L_p$) and compact ($\lambda \le \lambda_p$), yielding must be considered in the nominal moment strength of the member, in accordance with the requirements of AISC *Specification* Chapter F.

Unbraced Flexural Members

When flexural members are unbraced ($L_b > L_p$), have flange width-to-thickness ratios such that $\lambda > \lambda_p$, or have web width-to-thickness ratios such that $\lambda > \lambda_p$, lateral-torsional and elastic buckling effects must be considered in the calculation of the nominal moment strength of the member.

Noncompact or Slender Cross Sections

For flexural members that have width-to-thickness ratios such that $\lambda > \lambda_p$, local buckling must be considered in the calculation of the nominal moment strength of the member.

Available Flexural Strength for Weak-Axis Bending

The design of flexural members subject to weak-axis bending is similar to that for strong-axis bending, except that lateral-torsional buckling and web local buckling do not apply. See AISC *Specification* Section F6.