Topics :

1- Introduction to deflection.

2- Solved problem

5-1

Source ⇒ Civil PE 2nd edition

steel Design For Civil PE 2nd edition

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Prepared by Eng.Maged Kamel.

McCormac _ Chapter 10

DEFLECTIONS

(Disadvantages)

The deflections of steel beams are usually limited to certain maximum values. Among the several excellent reasons for deflection limitations are the following:

1. Excessive deflections may damage other materials attached to or supported by the beam in question. Plaster cracks caused by large ceiling joist deflections are one example.
2. The appearance of structures is often damaged by excessive deflections.
3. Extreme deflections do not inspire confidence in the persons using a structure, although the structure may be completely safe from a strength standpoint.
4. It may be necessary for several different beams supporting the same loads to deflect equal amounts.

Bad apperance

Loss of confidence

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What are the Limits For deflection?

$\Delta = \frac{\ell}{360}$ due to Live Load Loads

SEGUI

Standard American practice for buildings has been to limit service live-load deflections to approximately 1/360 of the span length. This deflection is supposedly the largest value that ceiling joists can deflect without causing cracks in underlying plaster. The 1/360 deflection is only one of many maximum deflection values in use because of different loading situations, different engineers, and different specifications. For situations where precise and delicate machinery is supported, maximum deflections may be limited to 1/1500 or 1/2000 of the span lengths. The 2010 AASHTO Specifications limit deflections in steel beams and girders due to live load and impact to 1/800 of the span. (For bridges in urban areas that are shared by pedestrians, the AASHTO *recommends* a maximum value equal to 1/1000 of the span lengths.)

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Servicability Criteria Controls Limits of deflection

The reader should note that deflection limitations fall in the serviceability area. Therefore, deflections are determined for service loads, and thus the calculations are identical for both LRFD and ASD designs.

various Methods to Find deflection Equations

Before substituting blindly into a formula that will give the deflection of a beam for a certain loading condition, the student should thoroughly understand the theoretical methods of calculating deflections. These methods include the moment area, conjugate beam, and virtual-work procedures. From these methods, various expressions can be determined, such as the following common one for the center line deflection of a uniformly loaded simple beam:

w Load U.L

Length L

$$\Delta_{\mathbb{C}} = \frac{5wL^4}{384EI}$$

Deflection For a Simply Supported beam

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A beam deflection criterion is usually expressed as limiting deflection to the length of the span, L, divided by a specified constant; for example, $L/360$ or $L/600$. ACI 530, *Building Code Requirements for Masonry Structures*, limits the deflection of a beam that supports masonry to a maximum of the lesser of $L/600$ and 0.3 in. For some overhead traveling cranes, the deflection is limited to $L/1000$. Table 1604.3 in the IBC provides the specific deflection limitations given in Table 5.2.

Deflection Limitations

Table 5.2 *Deflection Limitations in the International Building Code*

construction	Live only live load, L	snow load, S, or wind load, W	D+L dead load plus live load, $D + L$
roof members			
supporting plaster ceiling	$L/360$	$L/360$	$L/240$
supporting nonplaster ceiling	$L/240$	$L/240$	$L/180$
not supporting ceiling	$L/180$	$L/180$	$L/120$
floor members	$L/360$	–	$L/240$
exterior walls and interior partitions			
with brittle finishes	–	$L/240$	–
with nonbrittle finishes	–	$L/120$	–
farm buildings	–	–	$L/180$
greenhouses	–	–	$L/120$

CM #15

Serviceability

Serviceability requirements, per AISC *Specification* Chapter L, should be appropriate for the application. This includes an appropriate limit on the deflection of the flexural member and the vibration characteristics of the system of which the flexural member is a part. See also AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings* (West et al., 2003), AISC Design Guide 5, *Low- and Medium-Rise Steel Buildings* (Allison, 1991), and AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems Due to Human Activity* (Murray et al., 2016).

Manual #15

The maximum vertical deflection, Δ, can be calculated using the equations given in Tables 3-22 and 3-23. Alternatively, for common cases of simple-span beams and I-shaped members and channels, the following equation can be used:

$$\Delta = ML^2/(C_1 I_x) \tag{3-3}$$

where

C_1 = loading constant (see Figure 3-2), which includes the numerical constants appropriate for the given loading pattern, E (29,000 ksi), and a ft-to-in. conversion factor of 1,728 in.^{3}/ft^3

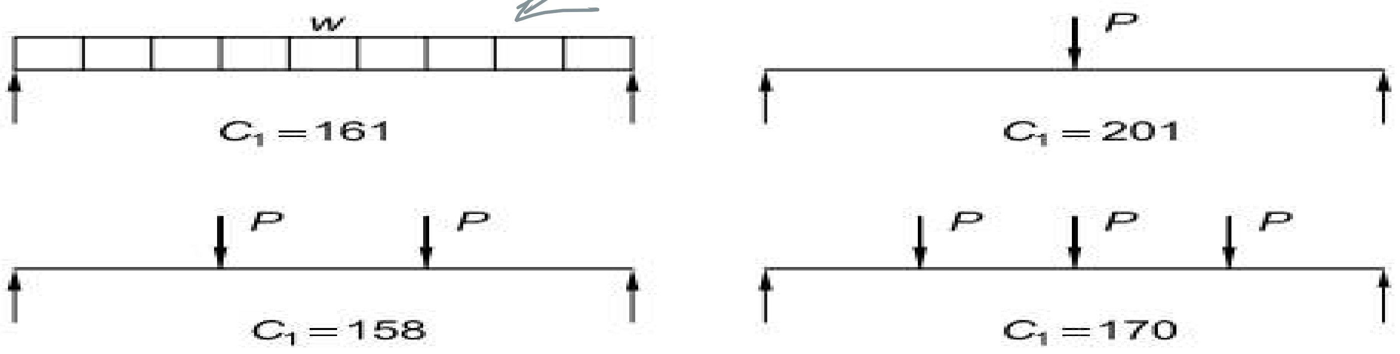
I_x = moment of inertia, in.^{4}

L = span length, ft

M = maximum service-load moment, kip-ft

$\boxed{1}$ Δ is Expresed with M, L, C_1, I_x

Case -1



C_1: Loading Constant

E = 29000 psi

Fig. 3-2. Loading constants for use in determining simple beam deflections.

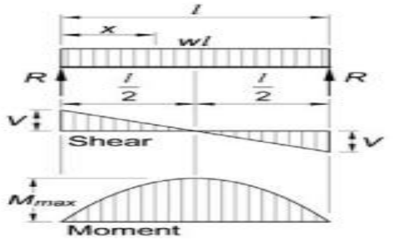
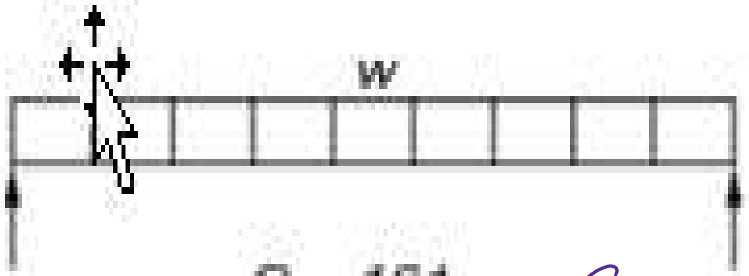


Table 3-23

Shears, Moments and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD

Total Equiv. Uniform Load	$= wl$
$R = V$	$= \frac{wl}{2}$
V_x	$= w\left(\frac{l}{2} - x\right)$
M_{max} (at center)	$= \frac{wl^2}{8}$
M_x	$= \frac{wx}{2}(l - x)$
Δ_{max} (at center)	$= \frac{5wl^4}{384EI}$
Δ_x	$= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$



$C_1 = 161$

① First Case U.L acting on span

Convert

$L \xRightarrow{Ft} (12)$ inch

New Expression

$$\Delta = \frac{5}{384}\,\frac{wL^4}{E I_x}$$

$E: 29000$ ksi

$I_x \Rightarrow \text{inch}^4$

$M:$ FT.kips $(12) \Rightarrow$ inch.kips

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

$$\Delta = \frac{ML^2}{C_1 I_x}$$

$$\Delta = \frac{5}{384}\,\frac{wL^2(8)}{(8)}\,\frac{\ell^2}{E I_x}$$

$$= \left(\frac{wL^2}{8}\right)\frac{40}{384}\,\frac{L^2}{29000 I_x} = \frac{M L^2}{\frac{384}{40}(29000) I_x}$$

$$\Delta - \frac{\text{inch.kip}(\text{inch}^2)}{\text{in}^4}$$

$$\left[\frac{\overset{M}{(12)}(12)^2(40)}{384(29000)}\right] \Rightarrow C_1 = 161.11$$

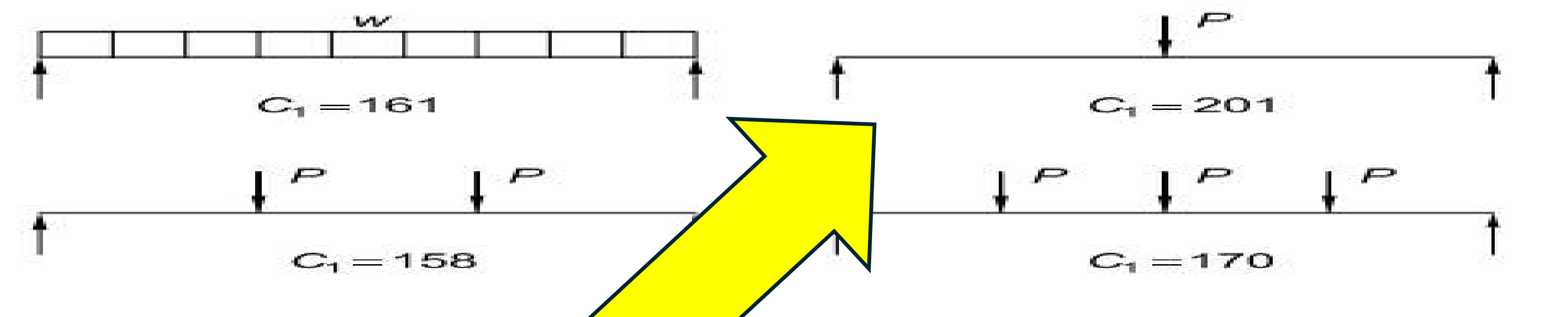


Fig. 3-2. Loading constants e in determining simple beam deflections.

L: Feet
Convert to inch
multiply by 12

(2) Second Case P C.L

Table 3.23 Case 7

New Expression

$$\Delta = \frac{PL^3}{48EI}, \quad M = \frac{PL}{4}$$

$E: 29000$ ksi

$I_x \Rightarrow$ inch^4

M: FT.kips
$(12) \Rightarrow$ inch.kips

$$\Delta = \left(\frac{PL}{4}\right)\frac{L^2}{12EI}$$

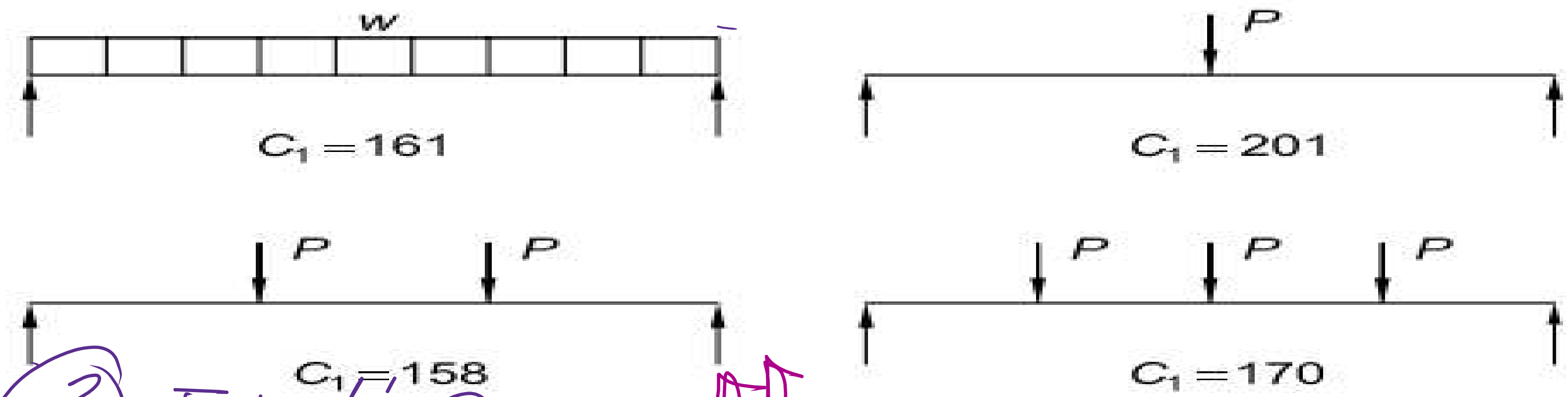
$$\Delta = \frac{ML^2}{C_1 I_x}$$

$$\Delta = \frac{ML^2}{12EI}$$

again

$$C_1 = \frac{12E}{ML^2} \Rightarrow \frac{12(29000)}{(12)(12)^2}$$

$$\boxed{C_1 = \frac{29000}{144} = 201.38 \approx 201}$$



③ Third Case

Fig. 3-2. Loading constants for use in determining simple beam deflections.

L: Feet
Convert to inch
multiply by 12

New Expression
$I_x \Rightarrow inch^4$
M: FT.Kips
$(12) \Rightarrow$ inch.kips

Refer to Table 3-23
Case 9

$$\Delta_{max} = \frac{23PL^3}{648EI} \Rightarrow M = \frac{PL}{3}$$

E: 29000 ksi

$$\Delta = \frac{ML^2}{C_1 I_x}$$

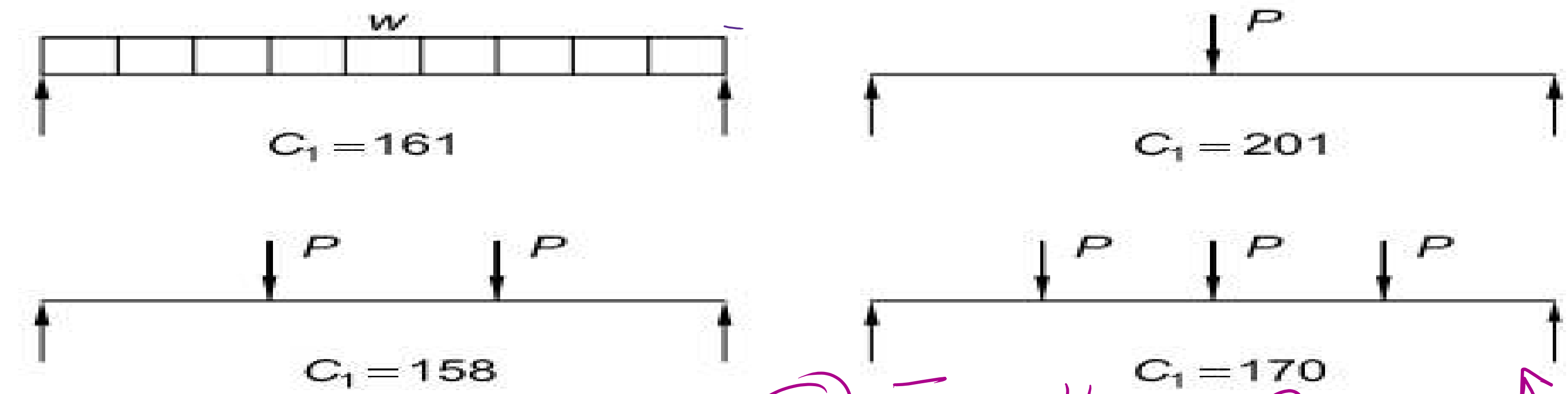
$$\Delta_{max} = \frac{23ML^2}{216EI}$$

again

$$C_1 = \frac{216}{23} \frac{(29000)}{(12)(12)^2 ML^2}$$

inch inch²

$$\boxed{C_1 = 157.60 \simeq 158}$$



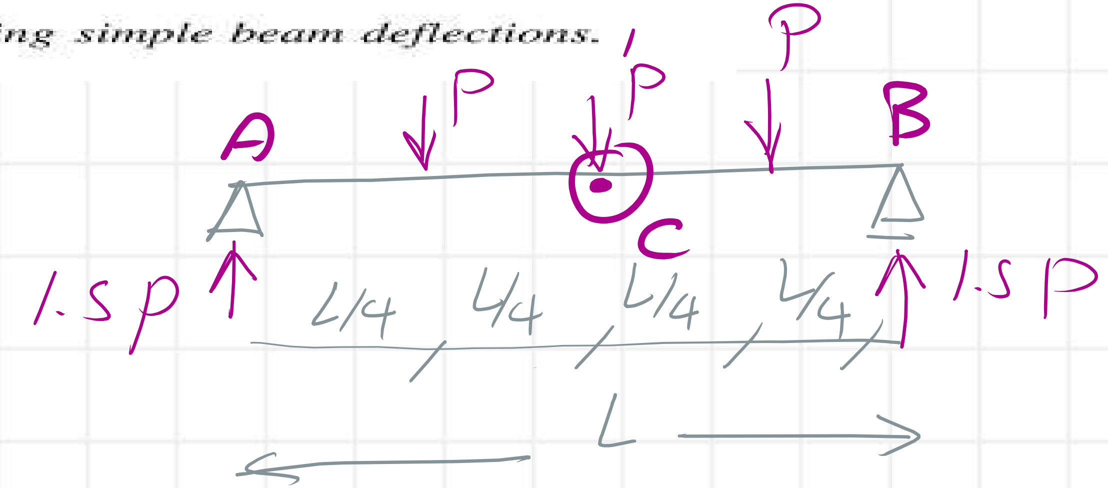
④ Fourth Case

Fig. 3-2. Loading constants for use in determining simple beam deflections.

L: Feet
Convert to inch
multiply by 12

Refer to Table

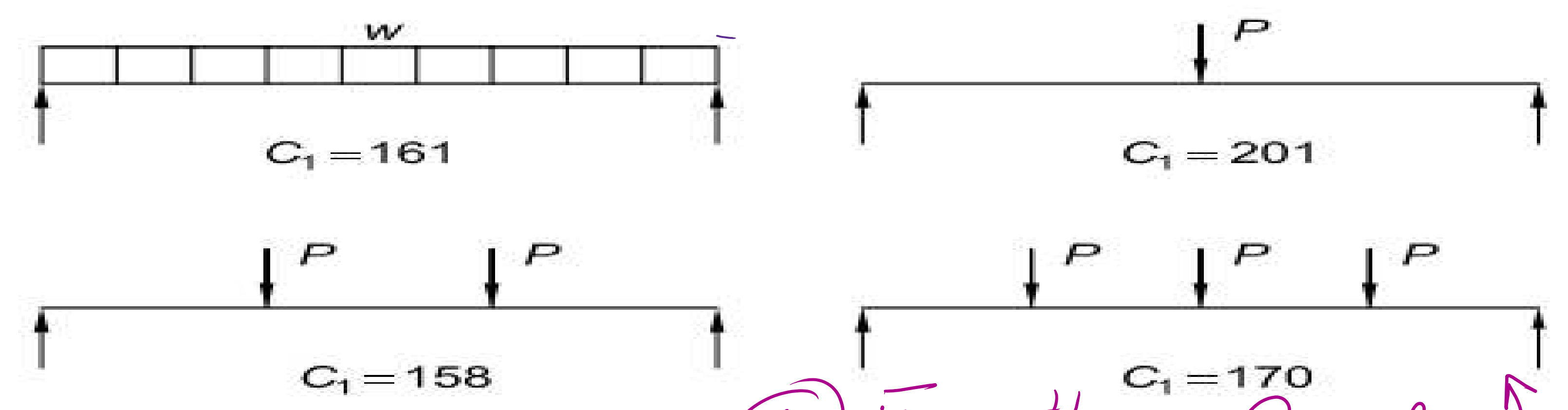
3-22a Case 4



$$M_C = 1.5P\frac{L}{2} - P\frac{L}{4}$$

$$M_C = \frac{(3-1)PL}{4} = \frac{PL}{2} = 0.50\,PL$$

$$\Delta_C = 0.05\,PL^3/EI$$



L: Feet
Convert to inch
multiply by 12

4 Fourth Case

Fig. 3-2. Loading constants for use in determining simple beam deflections.

New Expresion

$$\Delta = 0.05 \frac{PL^3}{EI}$$

$E: 29000$ ksi

$I_x \Rightarrow \text{inch}^4$

$M = 0.50\ PL$

M: FT.kips

$(12) \Rightarrow$ inch.kips

$$\Delta = \frac{ML^2}{C_1 I_x}$$

$$\Delta = 0.05 \left(\frac{0.50\ PL}{0.50}\right) \frac{L^2}{EI}$$

$$\Delta = \frac{0.05\ ML^2}{0.5\ EI} \qquad C_1 = \frac{0.5\ E}{0.05(12)^3}$$

$$\boxed{C_1 = \frac{0.5(29000)}{0.05(12)^3} = 167.80 \Rightarrow 170}$$