

Design of Reinforced Concrete Structures I

(ECIV 4316)

Chapter 8

Serviceability

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8.1 Introduction

- ✓ A properly designed reinforced concrete element is to satisfy two requirements; **strength and serviceability**. Strength requirements are elaborately dealt with in the previous chapters.
- ✓ **Serviceability** refers to some requirements that are needed to make the structure serviceable such as no excessive deflection or cracking. These are to be discussed in this chapter.
- ✓ **Historically**, deflections and crack widths have not been a problem for reinforced concrete elements. With the introduction of the strength design method and high strength steel reinforcement, the reinforcement stresses at service loads have increased considerably. Since crack widths and deflections are related to steel stresses, each of these has become more critical.

8.2 Deflection

- ✓ Excessive deflections can impair the appearance and efficiency of a structure and cause discomfort or panic to the occupants, in addition to excessive deflections that can cause cracking and possible separation of plaster finishes, and crushing of partition walls.
- ✓ The ACI Code provisions for control of deflections are concerned only with deflections that occur at service load levels under static conditions and may not apply to loads with strong dynamic characteristics such as those due to earthquakes, transient winds, and vibrating machinery.
- ✓ Deflection of a reinforced concrete element is made up of two components; **immediate deflection and time-dependent deflection.**

8.2 Deflection

8.2.1 Deflection Control

Two methods are given in the ACI Code for controlling deflections for beams and one-way slabs not supporting or attached to partitions or other construction likely to be damaged by large deflections.

- ✓ The first method indirectly controls deflection by means of providing minimum thickness, as shown in Table 8.1,

Table 8.1: Minimum Thickness of Beams and One-way Slabs

Restraint	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	$l/20$	$l/24$	$l/28$	$l/10$
Beams or ribbed one-way slabs	$l/16$	$l/18.5$	$l/21$	$l/8$

For f_y other than 420 MPa, the values in the table are to be multiplied by $\left(0.4 + \frac{f_y}{70}\right)$

8.2 Deflection

8.2.1 Deflection Control

- ✓ The second by directly limiting computed deflections, as shown in Table 8.2 based on ACI Code 7.3.1, 9.3.1 and 24.2.

Table 8.2: Maximum Permissible Calculated Deflections

Member			Deflection to be considered	Deflection limitation
Flat roofs	Not supporting or attached to non-structural elements likely to be damaged by large deflections		Immediate deflection due to the maximum of L_r , S and R	$l/180$
Floors			Immediate deflection due to L	$l/360$
Roof or floors	Supporting to or attached to nonstructural elements	Likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflections due to all sustained loads and the immediate deflection due to any additional live load.	$l/480$
		Not likely to be damaged by large deflections		$l/240$

8.2 Deflection

8.2.2 Immediate Deflection

Immediate deflections of beams and one-way slabs occur immediately when loads are applied to a structural member. The principal factors that affect the immediate deflection of a member are:

- a) Magnitude and distribution of loads.
- b) Span and restraint conditions.
- c) Cross-sectional dimensions and amount of reinforcement.
- d) Material properties.
- e) Extent of flexural cracking.

Figure 8.1 shows short term deflections for several cases of loading and support conditions.

8.2 Deflection

8.2.2 Immediate Deflection

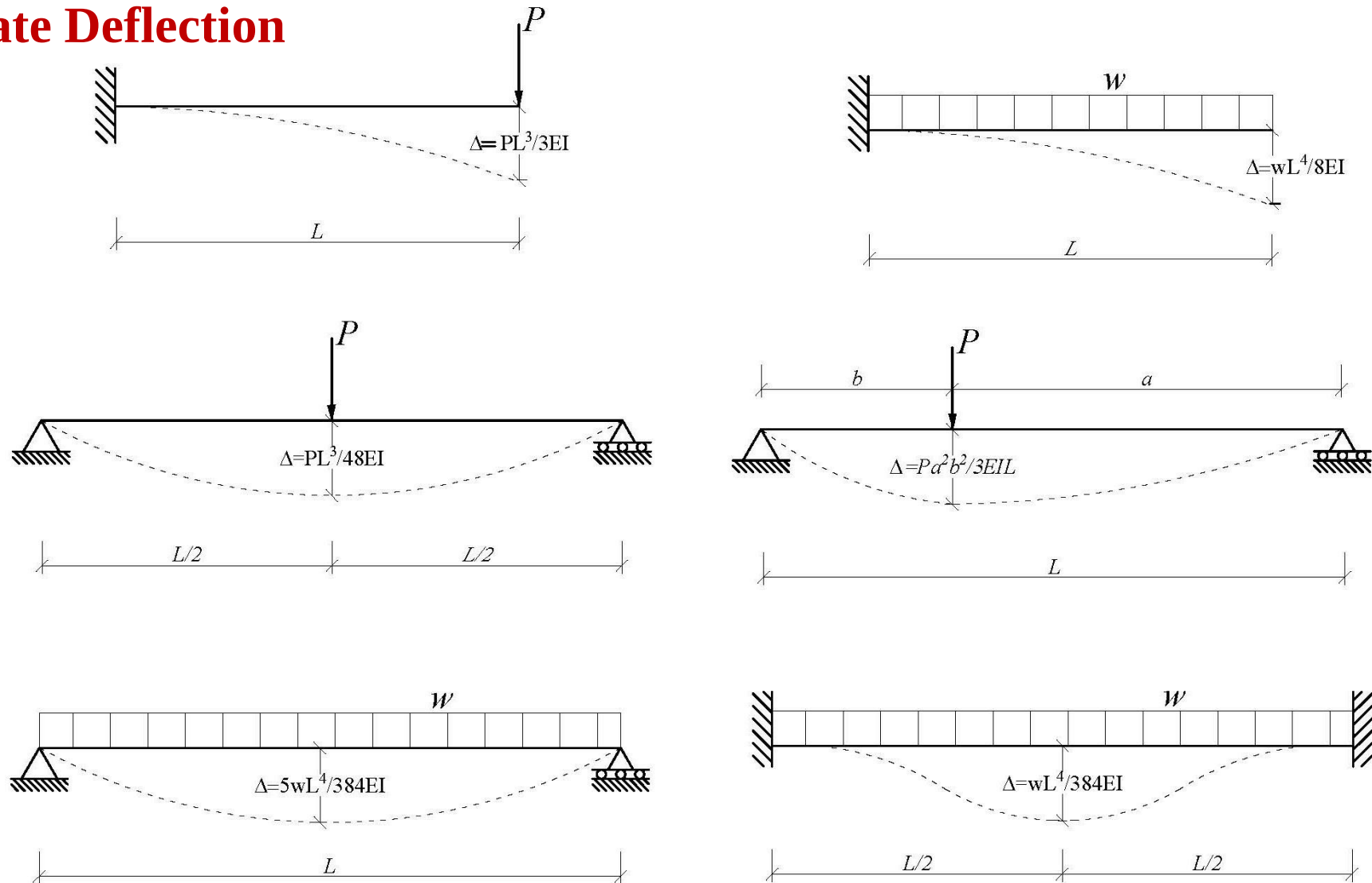


Figure 8.1: Immediate deflections for several cases of loading and support conditions

8.2 Deflection

8.2.2.1 Effective Moment of Inertia

- ✓ The flexural rigidity of a beam may not be constant along its length because of varying amount of steel reinforcement and cracking at different sections along the beam.
- ✓ According to *ACI Code 24.2.3*, immediate deflection is computed using elastic methods of analysis considering effects of cracking and reinforcement on member stiffness, with the modulus of elasticity for concrete $E_c = 4700 \sqrt{f'_c}$ and with the effective moment of inertia I_e is given as follows:

$$\text{For } M_a > (2/3)M_{cr}$$
$$I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a} \right)^2 \left(1 - \frac{I_{cr}}{I_g} \right)} \leq I_g \quad (8.1)$$

$$\text{For } M_a \leq (2/3)M_{cr}$$

$$I_e = I_g$$

8.2 Deflection

The cracking moment M_{cr} is given in *Eqn. (8.2)*

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (8.2)$$

M_a = maximum moment in member at stage deflection is computed

I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement

I_{cr} = moment of inertia of cracked section transformed to concrete

f_r = modulus of rupture of concrete = $0.62 \sqrt{f'_c}$

y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber in tension.

8.2 Deflection

- ✓ For each load combination being considered, such as dead load or dead plus live load, deflections should be calculated using an effective moment of inertia I_e computed with corresponding service load moment, M_a .
- ✓ The incremental deflection caused by the addition of load, such as live load, is then computed as the difference between deflection computed for any two-load combinations.
- ✓ According to *ACI Code*, effective moment of inertia for continuous one-way slabs and beams is permitted to be taken as the **average of values** obtained from *Eqn. (8.1)* for the **critical positive and negative moment** sections.
- ✓ Furthermore, **for prismatic members**, effective moment of inertia is permitted to be taken as the value obtained from *Eqn. (8.1)* at **mid span for simple and continuous spans**, and at the **support for cantilevers**.

8.2 Deflection

8.2.2.2 Transformed Concrete Section

a) Rectangular sections with tension reinforcement only:

The moment of inertia of a cracked beam (I_{cr}) with tension reinforcement, shown in Figure 8.2, is computed as follows:

Taking moment of areas about the neutral axis

$$(b)(y) \left(\frac{y}{2} \right) = n A_s (d - y)$$

where $n = \frac{E_s}{E_c}$ = modular ratio of elasticity, but not less than 6.0,

y = distance from the neutral axis to the extreme compression fiber.

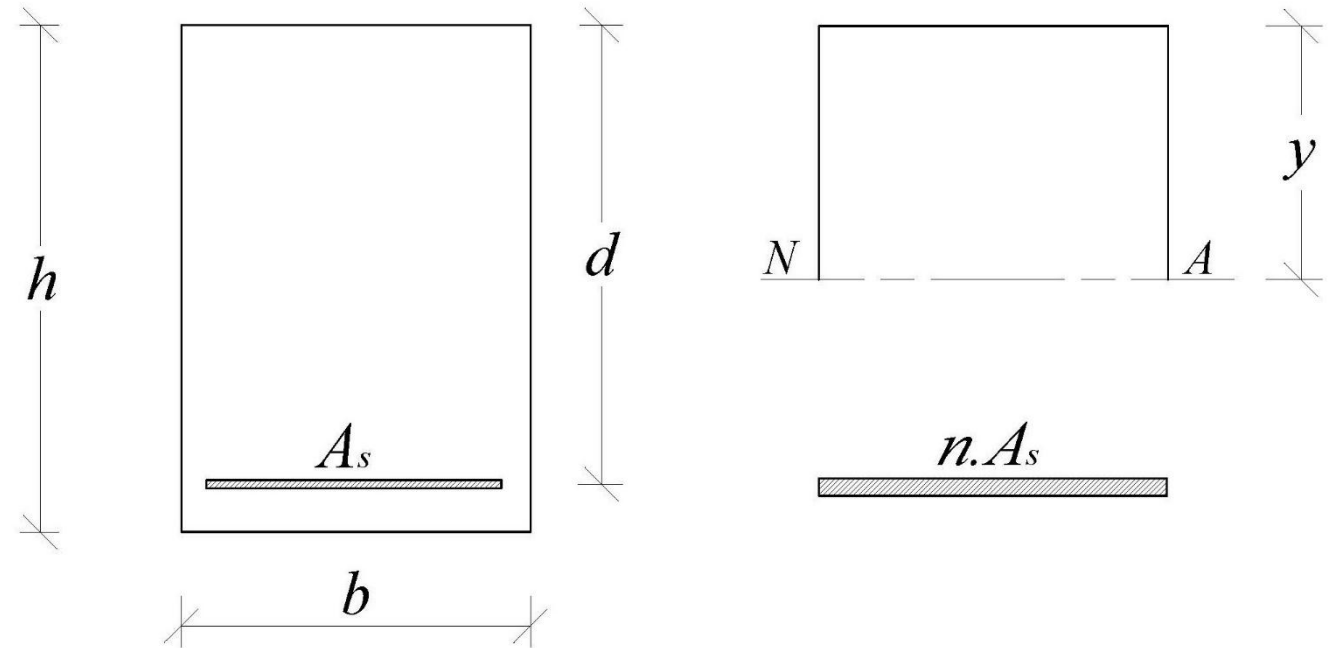


Figure 8.2: Cracked transformed section of singly reinforced beam

8.2 Deflection

8.2.2.2 Transformed Concrete Section

a) Rectangular sections with tension reinforcement only:

Letting $B = b / (n A_s)$

$$y = \frac{\sqrt{2Bd + 1} - 1}{B} \quad (8.3)$$

Moment of inertia of gross concrete section is given by

$$I_g = \frac{bh^3}{12} \quad (8.4)$$

Moment of inertia of cracked section about neutral axis,

$$I_{cr} = b y^3 / 3 + n A_s (d - y)^2 \quad (8.5)$$

8.2 Deflection

8.2.2.2 Transformed Concrete Section

b) Rectangular sections with tension and compression reinforcement:

For the rectangular section shown in Figure 8.3.

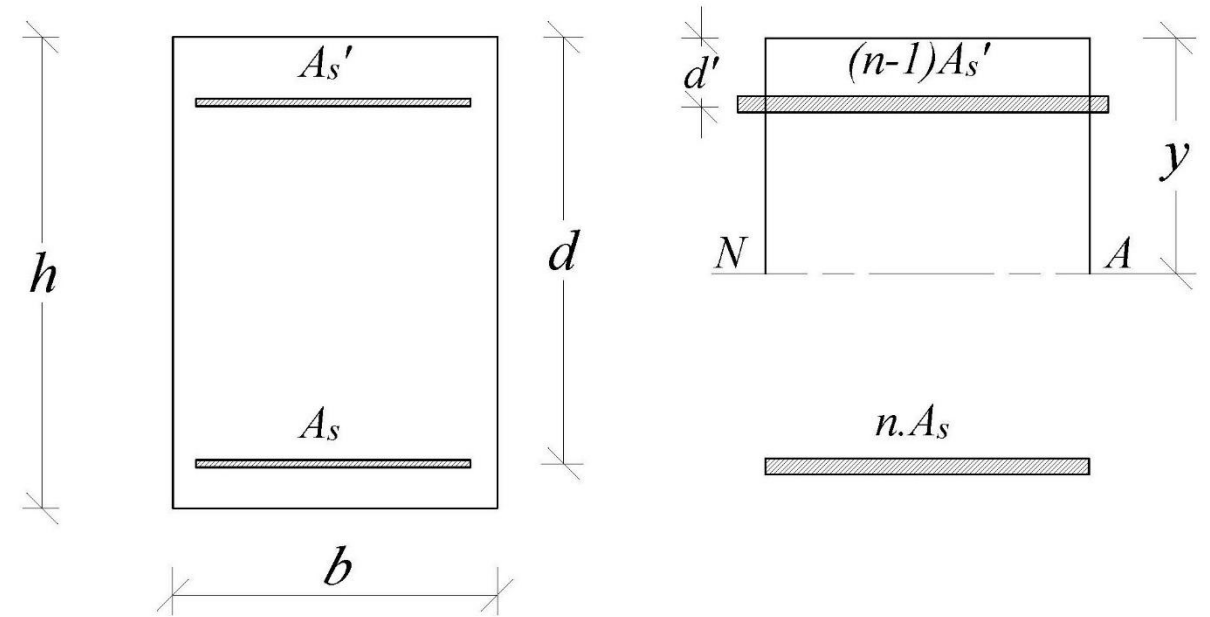


Figure 8.3: Cracked transformed section of doubly reinforced beam

$$\text{Letting } r = \frac{(n-1) A_s'}{n A_s}$$

$$y = \left[\sqrt{2dB(1 + rd'/d) + (1 + r)^2} - (1 + r) \right] / B \quad (8.6)$$

$$I_g = b h^3 / 12 \quad (8.4)$$

$$I_{cr} = b y^3 / 3 + n A_s (d - y)^2 + (n - 1) A_s' (y - d')^2 \quad (8.7)$$

8.2 Deflection

8.2.2.2 Transformed Concrete Section

c) T-sections with tension reinforcement only:

From Figure 8.4,

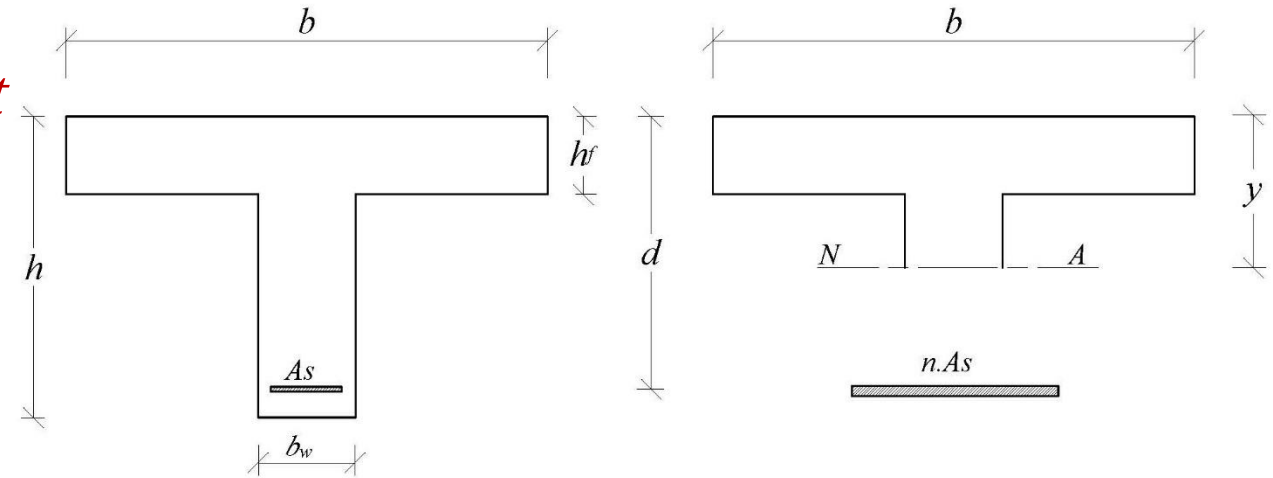


Figure 8.4: Cracked transformed section of singly reinforced T-beam

1. When $y \leq h_f$, the section is treated as a singly reinforced rectangular section with b_w replaced with b .
2. When $y > h_f$ relevant equations are given below

Letting $C = \frac{b_w}{n A_s}$, and $f = \frac{h_f(b - b_w)}{n A_s}$

$$y_t = \frac{h - 1/2[(b - b_w)h_f^2 + b_w h^2]}{(b - b_w)h_f + b_w h} \quad (8.8)$$

8.2 Deflection

8.2.2.2 Transformed Concrete Section

c) T- sections with tension reinforcement only:

$$y = \left[\sqrt{C(2d + h_f f) + (1 + f)^2} - (1 + f) \right] / C \quad (8.9)$$

$$I_g = \frac{(b - b_w) h_f^3}{12} + \frac{b_w h^3}{12} + (b - b_w) h_f \left(h - \frac{h_f}{2} - y_t \right)^2 + b_w h \left(y_t - \frac{h}{2} \right)^2 \quad (8.10)$$

$$I_{cr} = \frac{(b - b_w) h_f^3}{12} + \frac{b_w y^3}{3} + h_f (b - b_w) \left(y - \frac{h_f}{2} \right)^2 + n A_s (d - y)^2 \quad (8.11)$$

8.2 Deflection

8.2.2.2 Transformed Concrete Section

d) T-sections with tension and compression reinforcement:

As shown in Figure 8.5,

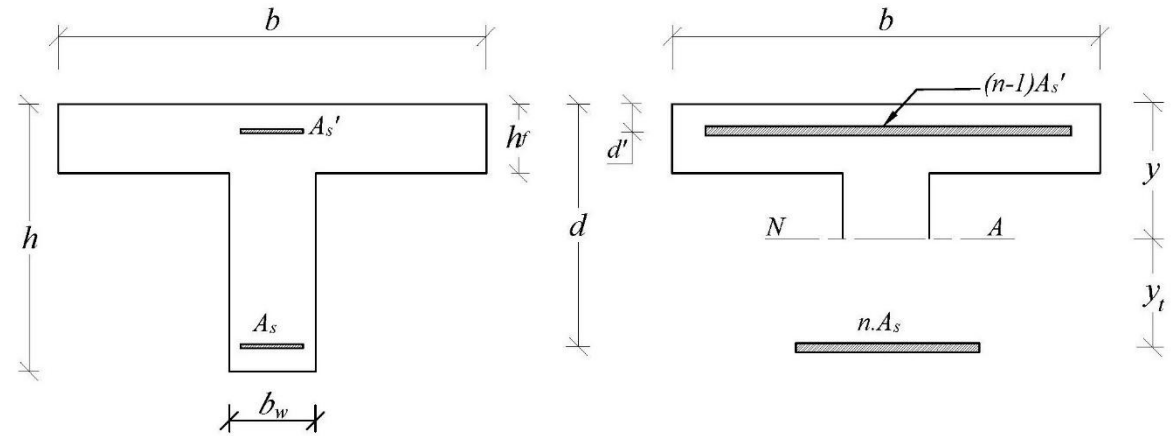


Figure 8.5: Cracked transformed section of doubly reinforced T-beam

1. When $y \leq h_f$, the section is treated as a doubly reinforced rectangular section with b_w replaced with b .
2. When $y > h_f$ relevant equations are given below

Letting $C = \frac{b_w}{n A_s}$, and $f = \frac{h_f(b - b_w)}{n A_s}$

$$y_t = \frac{h - 1/2[(b - b_w)h_f^2 + b_w h^2]}{(b - b_w)h_f + b_w h} \quad (8.8)$$

8.2 Deflection

8.2.2.2 Transformed Concrete Section

d) T- sections with tension and compression reinforcement:

$$y = \left[\sqrt{C(2d + h_f f) + (1 + f)^2} - (1 + f) \right] / C \quad (8.9)$$

$$y_t = \frac{h - 1/2[(b - b_w)h_f^2 + b_w h^2]}{(b - b_w)h_f + b_w h} \quad (8.8)$$

$$y = \frac{\left[\sqrt{C(2d + h_f f + 2rd')} + (f + r + 1)^2 - (f + r + 1) \right]}{C} \quad (8.12)$$

$$I_g = \frac{(b - b_w) h_f^3}{12} + \frac{b_w h^3}{12} + (b - b_w) h_f \left(h - \frac{h_f}{2} - y_t \right)^2 + b_w h \left(y_t - \frac{h}{2} \right)^2 \quad (8.13)$$

$$I_{cr} = \frac{(b - b_w)h_f^3}{12} + \frac{b_w y^3}{3} + h_f(b - b_w) \left(y - \frac{h_f}{2} \right)^2 + nA_s(d - y)^2 + (n - 1)A'_s(y - d')^2 \quad (8.14)$$

8.2 Deflection

8.2.3 Time-Dependent Deflection

- ✓ **Shrinkage and creep** due to sustained loads cause additional time-dependent deflections which may exceed immediate deflections of the structure.
- ✓ These deflections may be **two to three** times as large as the immediate elastic deflection that occurs when the sustained load is applied.
- ✓ Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement and magnitude of the sustained load.
- ✓ According to *ACI Code 24.2.4.1*, additional time-dependent deflection resulting from creep and shrinkage for flexural members is determined by multiplying the immediate deflection caused by the sustained load considered by the factor given by *Eqn. (2.7)*

8.2 Deflection

8.2.3 Time-Dependent Deflection

$$\lambda_{\Delta} = \frac{\xi}{1 + 50\rho'} \quad (2.7)$$

where,

λ_{Δ} = multiplier for additional time-dependent deflection.

$\rho' = \frac{A'_s}{bd}$ is the compression reinforcement ratio taken at mid span for simple and continuous spans and at the support for cantilevers.

A'_s = cross sectional area of compression reinforcement

b = width of cross section

d = effective depth of cross section

ξ = time-dependent factor for a sustained load, given in Table 8.3.

Table 8.3: Time-dependent factors

Period (months)	ξ
3	1.0
6	1.2
12	1.4
60 or more	2.0

Example (8.1)

For the simply supported beam shown in Figure 8.6 calculate the maximum deflection at an age of 5 years. It is given that service dead load on beam, including own weight, is equal to 10 kN/m and a concentrated service live load of 30 kN , 100% of which is sustained.

Use $f'_c = 30 \text{ MPa}$ and $f_y = 420 \text{ MPa}$

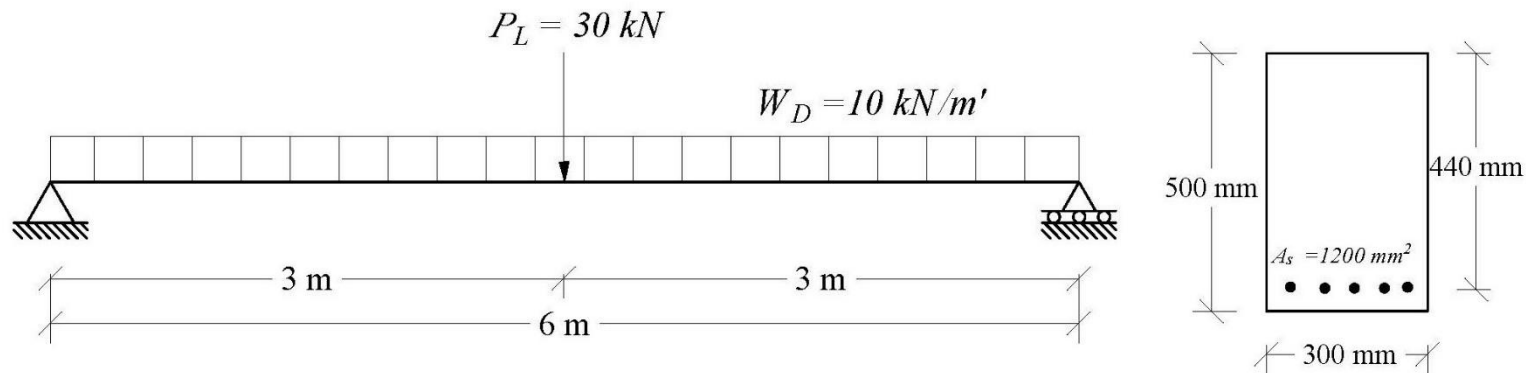


Figure 8.6: Loaded beam and its cross section

Example (8.1)

Solution:

The dead load moment is given as

$$M_D = \frac{10 (6.0)^2}{8} = 45.0 \text{ kN.m}$$

The live load moment is given by

$$M_L = \frac{30 (6.0)}{4} = 45 \text{ kN.m}$$

Total dead and live load moment is

$$M_D + M_L = 45.0 + 45.0 = 90 \text{ kN.m}$$

The sustained bending moment is

$$M_D + M_L = 45.0 + 45.0 = 90 \text{ kN.m}$$

Modulus of rupture $f_r = 0.62 \sqrt{f'_c} = 0.62 \sqrt{30} = 3.4 \text{ MPa}$

$$E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{30} = 25743 \text{ MPa}$$

Example (8.1)

$$n = \frac{E_s}{E_c} = \frac{2.04 (10)^5}{25743} = 7.9, \quad I_g = \frac{(300)(500)^3}{12} = 3.125 \times 10^9 \text{ mm}^4$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{3.4 (3.125 \times 10^9)}{250(10^6)} = 42.5 \text{ kN.m}$$

$$B = \frac{b}{n A_s} = \frac{300}{7.90 (1200)} = 0.032 \text{ mm}^{-1}$$

$$y = \frac{\sqrt{2 d B + 1} - 1}{B} = \frac{\sqrt{2 (440) (0.032) + 1} - 1}{0.032} = 137.5 \text{ mm}$$

$$I_{cr} = \frac{b y^3}{3} + n A_s [d - y]^2$$

$$I_{cr} = \frac{300 (137.5)^3}{3} + 7.90 (1200) [440 - 137.5]^2 = 1.127 \times 10^9 \text{ mm}^4$$

a. Effective moment of inertia under dead load only:

$$\frac{M_D}{M_{cr}} = \frac{45.0}{42.5} = 1.06 > \frac{2}{3}$$

Example (8.1)

$$I_e = \frac{I_{cr}}{1 - \left(\left(\frac{2}{3} \right) \frac{M_{cr}}{M_a} \right)^2 \left(1 - \frac{I_{cr}}{I_g} \right)} \leq I_g$$

$$I_e = \frac{1.127 \times 10^9}{1 - \left(\frac{(2/3)42.5}{45} \right)^2 \left(1 - \frac{1.127 \times 10^9}{3.125 \times 10^9} \right)} = 1.509 \times 10^9 < I_g \quad \text{OK}$$

b. Effective moment of inertia under dead and live loads:

$$\frac{M_{D+L}}{M_{cr}} = \frac{90.0}{42.5} = 2.12 > \frac{2}{3}$$

$$I_e = \frac{1.127 \times 10^9}{1 - \left(\frac{(2/3)42.5}{90} \right)^2 \left(1 - \frac{1.127 \times 10^9}{3.125 \times 10^9} \right)} = 1.203 \times 10^9 < I_g \quad \text{OK}$$

Example (8.1)

Initial or immediate deflections:

Deformation caused by dead load only

$$(\Delta_i)_D = \frac{5 w l^4}{384 (E_c I_e)} = \frac{5 (10) (6000)^4}{384 (25743) (1.509 \times 10^9)} = 4.34 \text{ mm}$$

Deformation caused by dead and live loads

$$\begin{aligned} (\Delta_i)_{D+L} &= \frac{5 w l^4}{384 (E_c I_e)} + \frac{P l^3}{48 (E_c I_e)} \\ &= \frac{5 (10) (6000)^4}{384 (25743) (1.203 \times 10^9)} + \frac{(30 \times 10^3) (6000)^3}{48 (25743) (1.203 \times 10^9)} = 9.81 \text{ mm} \end{aligned}$$

Deformation caused by live load only

$$(\Delta_i)_L = (\Delta_i)_{D+L} - (\Delta_i)_D = 9.81 - 4.34 = 5.47 \text{ mm}$$

Example (8.1)

Allowable deflections:

For flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections, $(\Delta_i)_L \leq l/180$

$$l/180 = 6000/180 = 33.33 \text{ mm} > 5.47 \text{ mm} \quad O.K$$

For floors, not supporting or attached to nonstructural elements, are likely to be damaged by large deflections, $(\Delta_i)_L \leq l/360$

$$l/360 = 6000/360 = 16.67 \text{ mm} > 5.47 \text{ mm} \quad O.K$$

Time-dependent deflection:

$$\rho' = 0.00$$

Table 8.4 shows the calculation of the total time dependent deflection

Example (8.1)

Table 8.4: Time dependent deflection

Duration	ξ	$\lambda_{\Delta} = \frac{\xi}{1 + 50\rho'}$	$(\Delta_i)_{sus}$	$(\Delta_i)_L$	$\lambda_{\Delta} (\Delta_i)_{sus}$	$(\Delta_i)_{total}$
5 years	2.0	2.0	9.81 mm	5.47 mm	19.62 mm	25.1 mm

For roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections, $\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_L \leq l/480$

$$\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_L = 25.1 \text{ mm}$$

$$l/480 = 600/480 = 12.5 \text{ mm} < 25.1 \text{ mm}$$

i.e. thickness of beam needs to be increased.

For roof or floor construction, is supporting or attached to nonstructural elements, not likely to be damaged by large deflections, $\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_l \leq l/240$

$$\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_l = 25.1 \text{ mm}$$

$$l/240 = 6000/240 = 25.0 \text{ mm} \approx 25.1 \text{ mm} \text{ O.K}$$

Example (8.2)

For the simply supported beam shown in Figure 8.7 calculate the maximum deflection at an age of 5 years. It is given that service dead load on beam, not including own weight, = 2 kN/m , service live load = 5 kN/m , 50 % of which is sustained.

Use $f'_c = 25 \text{ MPa}$ and $f_y = 420 \text{ MPa}$

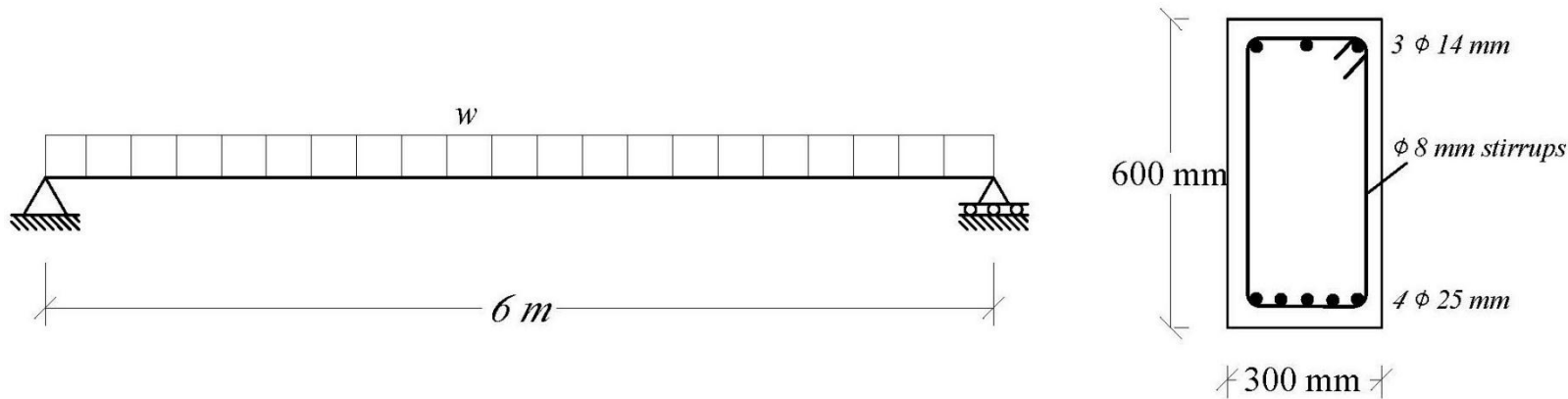


Figure 8.7: Loaded beam and its cross section

Example (8.2)

Solution:

Moments:

$$w_D = 2 + 0.30 (0.60) (25.0) = 6.5 \text{ kN/m}$$

The dead load moment is given as

$$M_D = \frac{6.5 (7.5)^2}{8} = 45.7 \text{ kN.m}$$

The live load moment is given by

$$M_L = \frac{5 (7.5)^2}{8} = 35.15 \text{ kN.m}$$

Total dead and live load moment is

$$M_D + M_L = 45.7 + 35.1 = 80.8 \text{ kN.m}$$

The sustained bending moment is

$$M_{sus} = M_D + 0.50 M_L = 45.7 + 0.50 (35.1) = 63.2 \text{ kN.m}$$

Example (8.2)

$$\text{Modulus of rupture } f_r = 0.62 \sqrt{f'_c} = 0.62 \sqrt{25} = 3.1 \text{ MPa}$$

$$E_c = 4700 \sqrt{f'_c} = 4700 \sqrt{25} = 23500 \text{ MPa}$$

$$n = \frac{E_s}{E_c} = \frac{2.04 (10)^5}{23500} = 8.68$$

$$I_g = \frac{(300)(600)^3}{12} = 5.4 \times 10^9 \text{ mm}^4$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{3.1 (5.4 \times 10^9)}{300(10^6)} = 55.8 \text{ kN.m}$$

$$B = \frac{b}{n A_s} = \frac{300}{8.68 (1570)} = 0.022 \text{ mm}^{-1}$$

$$r = \frac{(n - 1) A'_s}{n A_s} = \frac{(8.68 - 1) (462)}{8.68 (1570)} = 0.26$$

$$d = 600 - 40 - 8.0 - 10.0 = 542.0 \text{ mm}$$

$$d' = 40.0 + 8.0 + 7.0 = 55.0 \text{ mm}$$

Example (8.2)

$$y = \left[\sqrt{2dB(1 + rd'/d) + (1 + r)^2} - (1 + r) \right] / B$$

$$y = \left(\frac{1}{0.022} \right) \left[\sqrt{2 (542) (0.022) \left(1 + 0.26 \frac{(55)}{(542)} \right) + (1 + 0.26)^2} - (1 + 0.26) \right]$$
$$= \mathbf{174.8 \text{ mm}}$$

$$I_{cr} = b y^3 / 3 + n A_s (d - y)^2 + (n - 1) A'_s (y - d')^2$$

$$I_{cr} = \frac{300 (174.8)^3}{3} + 8.68 (1570) [542 - 174.8]^2 + (7.68) (462) [174.8 - 5.5]^2$$
$$= \mathbf{2.47 \times 10^9 \text{ mm}^4}$$

a) Effective moment of inertia under dead load only:

$$\frac{M_D}{M_{cr}} = \frac{45.7}{55.8} = 0.819 \leq \frac{2}{3}$$

$$\text{i.e. } I_e = I_g = 5.4 \times 10^9 \text{ mm}^4$$

Example (8.2)

b. Effective moment of inertia under sustained load:

$$\frac{M_{sus}}{M_{cr}} = \frac{63.2}{55.8} = 1.13 > \frac{2}{3}$$

$$I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a} \right)^2 \left(1 - \frac{I_{cr}}{I_g} \right)} \leq I_g$$

$$I_e = \frac{2.47 \times 10^9}{1 - \left(\frac{(2/3)55.8}{63.2} \right)^2 \left(1 - \frac{2.47 \times 10^9}{5.4 \times 10^9} \right)} = 3.042 \times 10^9 < I_g \text{ OK}$$

c. Effective moment of inertia under dead and live load:

$$\frac{M_{D+L}}{M_{cr}} = \frac{80.8}{55.8} = 1.45 > \frac{2}{3}$$

$$I_e = \frac{2.47 \times 10^9}{1 - \left(\frac{(2/3)55.8}{80.8} \right)^2 \left(1 - \frac{2.47 \times 10^9}{5.4 \times 10^9} \right)} = 2.79 \times 10^9 < I_g \text{ OK}$$

Example (8.2)

Immediate deflections:

Deformation caused by dead load only

$$(\Delta_i)_D = \frac{5 w l^4}{384 (E_c I_e)} = \frac{5 (6.5) (7500)^4}{384 (23500) (5.4 \times 10^9)} = 2.11 \text{ mm}$$

Deformation caused by sustained load

$$(\Delta_i)_{\text{sus}} = \frac{5 w l^4}{384 (E_c I_e)} = \frac{5 (9.0) (7500)^4}{384 (23500) (3.042 \times 10^9)} = 5.19 \text{ mm}$$

Deformation caused by dead and live loads

$$(\Delta_i)_{D+L} = \frac{5 w l^4}{384 (E_c I_e)} = \frac{5 (11.5) (7500)^4}{384 (23500) (2.79 \times 10^9)} = 7.23 \text{ mm}$$

Deformation caused by live load only

$$(\Delta_i)_L = (\Delta_i)_{D+L} - (\Delta_i)_D = 7.23 - 2.11 = \mathbf{5.12 \text{ mm}}$$

Example (8.2)

Allowable deflections:

For flat roofs, not supporting or attached to nonstructural elements likely to be damaged by large deflections, $(\Delta_i)_L \leq l/180$

$$l/180 = 7500/180 = 41.67 \text{ mm} > 5.12 \text{ mm} \quad O.K$$

For floors, not supporting or attached to nonstructural elements likely to be damaged by large deflections, $(\Delta_i)_l \leq l/360$

$$l/360 = 7500/360 = 20.83 \text{ mm} > 5.12 \text{ mm} \quad O.K$$

Time-dependent deflection:

$$\rho' = \frac{4.62}{30 (54.2)} = 0.00284$$

The following Table shows the calculation of the total time dependent deflection

Duration	ξ	$\lambda_{\Delta} = \frac{\xi}{1 + 50\rho'}$	$(\Delta_i)_{sus}$	$(\Delta_i)_L$	$\lambda_{\Delta} (\Delta_i)_{sus}$	$(\Delta_i)_{total}$
5 years	2.0	1.751	5.19 mm	5.12 mm	9.09 mm	14.2 mm

Example (8.2)

For roof or floor construction, supporting or attached to nonstructural elements likely to be damaged by large deflections, $\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_L \leq l/480$

$$\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_L = 14.2 \text{ mm}$$

$$l/480 = 750/480 = 15.62 \text{ mm} > 14.2 \text{ mm} \quad O.K$$

For roof or floor construction, supporting or attached to nonstructural elements not likely to be damaged by large deflections, $\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_L \leq l/240$

$$\lambda_{\Delta} (\Delta_i)_{sus} + (\Delta_i)_L = 14.2 \text{ mm}$$

$$l/240 = 750/240 = 31.25 \text{ mm} > 14.2 \text{ mm} \quad O.K$$

8.3 Crack width control

- ✓ With the advent of high-strength steels and with the use of strength design methods which allow higher stresses in reinforcement, control of flexural cracking has assumed more importance.
- ✓ Crack widths are of concern for three main reasons: appearance, leakage, and corrosion.
- ✓ Wide cracks are unsightly and sometimes lead to concern by owners and occupants.
- ✓ Crack control is important in the design of liquid-retaining structures. Leakage is a function of crack width.
- ✓ Corrosion of reinforcement has traditionally been related to crack width. Modern studies suggest that the factors governing the eventual development of corrosion are independent of the crack width.
- ✓ A simple and more practical equation has been adopted starting with the 1995 code which directly limits the maximum reinforcement spacing.

8.3 Crack width control

- ✓ The new method is intended to control surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure.
- ✓ The new method, for this reason, does not support predicting crack widths in the field.
- ✓ According to the new method given in *ACI Code 24.3.2*, the spacing of reinforcement closest to tension surface shall not exceed the lesser of *Eqns.* (8.14) and (8.15)

$$S = 380 \left(\frac{280}{f_s} \right) - 2.5 C_c \quad (8.14)$$

and maximum spacing of reinforcement is given as

$$S_{max} = 300 \left(\frac{280}{f_s} \right) \quad (8.15)$$

where S = center-to-center spacing of flexural tension reinforcement nearest to the extreme tension face.

8.3 Crack width control

- ✓ f_s = calculated stress in reinforcement at service load computed as the unfactored moment divided by the product of steel area and internal moment arm. It is permitted to take f_s as $2/3f_y$.
- ✓ C_c = Clear cover from the nearest surface in tension to the surface of flexural tension reinforcement.

8.4 Skin Reinforcement in Deep Flexural Members

- ✓ According to ACI Code 9.7.2.3, for deep flexural members with overall height h **exceeding 900 mm**, additional longitudinal reinforcement for crack control must be distributed along the side faces over the full depth of the flexural tension zone.
- ✓ Without such additional reinforcement, the width of the cracks in the web may exceed widths at the level of the flexural steel reinforcement, as shown in Figure 8.8.
- ✓ The required skin reinforcement must be uniformly distributed along both side faces of the member within the flexural tension zone, considered to extend for a distance $h/2$ from the tension face.

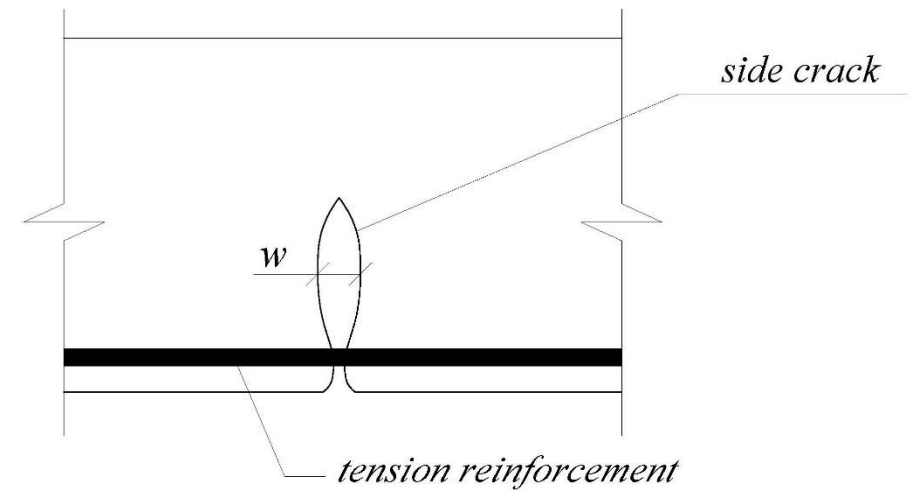


Figure 8.8: Side cracks in deep members

8.4 Skin Reinforcement in Deep Flexural Members

- ✓ The spacing of the skin reinforcement, S shall be computed using Eqns. (8.14) and (8.15), where C_c is the least distance from the surface of the skin reinforcement to the side face, as shown in Figure 8.9.
- ✓ The code does not specify the size of the skin reinforcement. Research has shown that the spacing rather than bar size is of primary importance.
- ✓ Typically bars ϕ 10 mm to ϕ 16 mm, with minimum area of 0.2 mm^2 per mm of depth is provided.

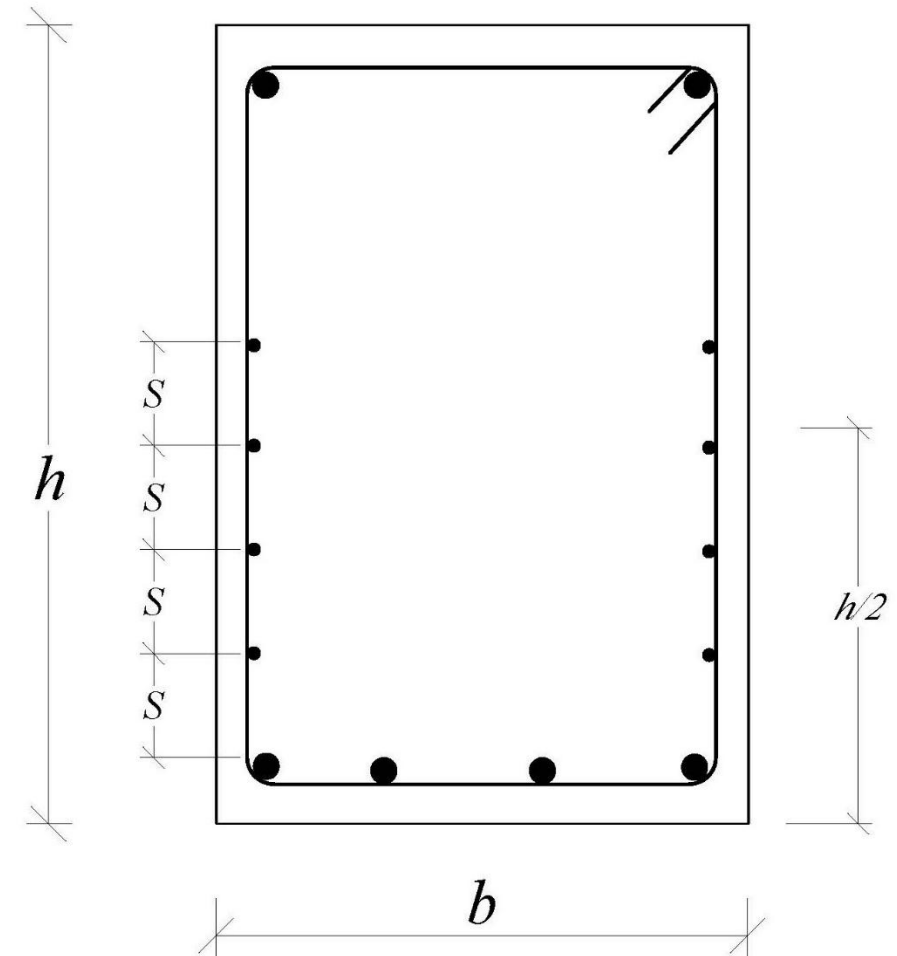


Figure 8.9: Skin reinforcement

Example (8.3):

For the cross section shown Figure 8.10, determine whether the reinforcement satisfies ACI Code requirements for crack control.

Use $f'_c = 28 \text{ MPa}$ and $f_y = 420 \text{ MPa}$.

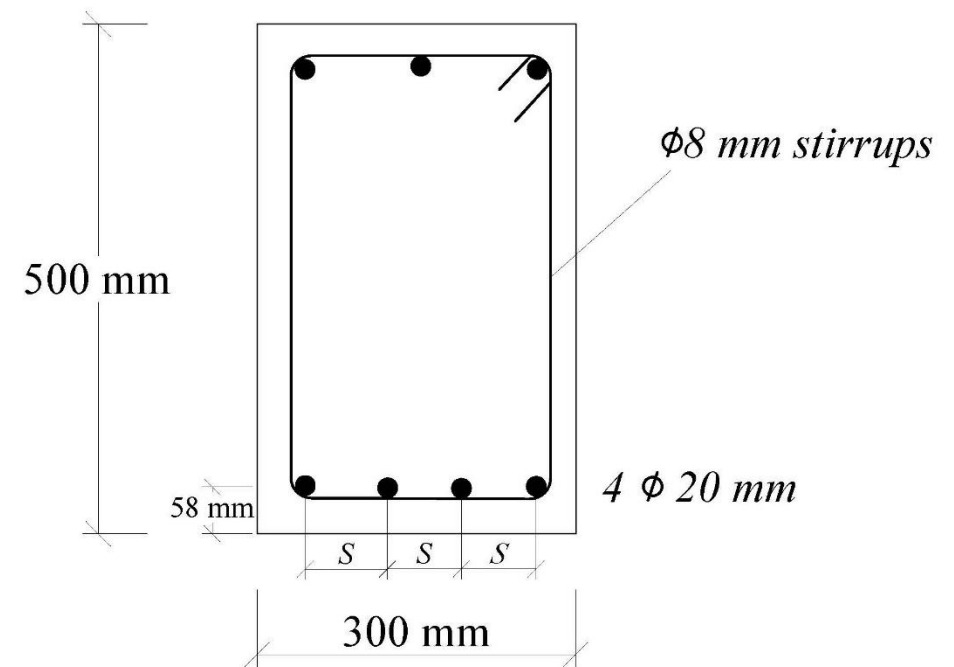


Figure 8.10: Cross section

Example (8.3):

$$S = 380 \left(\frac{280}{f_s} \right) - 2.5 C_c = \frac{106400}{f_s} - 2.5 C_c$$

$$C_c = 40 + 8.0 = 48 \text{ mm}$$

$$S = \frac{106400}{2/3(420)} - 2.5(48) = 260 \text{ mm}$$

$$S_{max} = \frac{84000}{f_s} = \frac{84000}{2/3 (420)} = 300 \text{ mm} > 260 \text{ mm} \text{ O.K}$$

Actual bar spacing (center-to-center) =

$$\frac{300 - 2 (40.0) - 2 (8.0) - 20.0}{3} = 61.3 \text{ mm} < 260 \text{ mm} \text{ O.K}$$