Chapter 1

Design for Flexure

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1.1 Introduction

Design of reinforced concrete elements for flexure involves; i) sectional design and ii) member detailing. Sectional design includes the determination of cross-sectional geometry and the required longitudinal reinforcement as per Chapter 10 of ACI 318-05. Member detailing includes the determination of bar lengths, locations of cut-off points and detailing of reinforcement as governed by the development, splice and anchorage length requirements specified in chapter 12 of ACI 318-05. This Chapter of the Handbook deals with the sectional design of members for flexure on the basis of the Strength Design Method of ACI 318-05. The Strength Design Method requires the conditions of static equilibrium and strain compatibility across the depth of the section to be satisfied.

The following are the assumptions for Strength Design Method:

i. Strains in reinforcement and concrete are directly proportional to the distance from neutral axis. This implies that the variation of strains across the section is linear, and unknown values can be computed from the known values of strain through a linear relationship.
ii. Concrete sections are considered to have reached their flexural capacities when they develop 0.003 strain in the extreme compression fiber.
iii. Stress in reinforcement varies linearly with strain up to the specified yield strength. The stress remains constant beyond this point as strains continue increasing. This implies that the strain hardening of steel is ignored.
iv. Tensile strength of concrete is neglected.
v. Compressive stress distribution of concrete can be represented by the corresponding stress-strain relationship of concrete. This stress distribution may be simplified by a rectangular stress distribution as described in Fig. 1-1.

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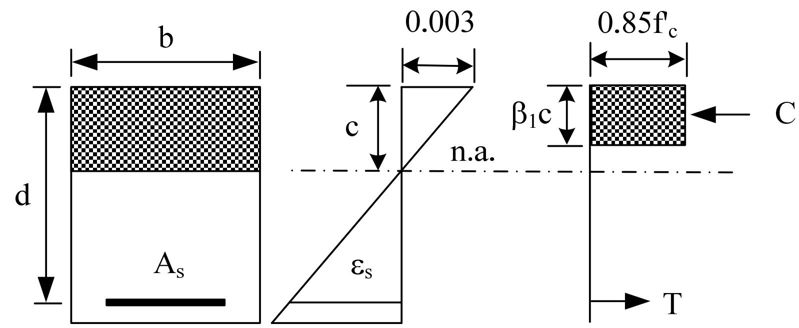


$$\beta_1 = 0.85 \quad for\ f_c' \leq 4000\ psi$$

$$\beta_1 = 0.85 - 0.05\frac{f_c' - 4000}{1000} \geq 0.65 \quad for\ f'_c > 4000 psi$$

Fig. 1-1 Ultimate strain profile and corresponding rectangular stress distribution

1.2 Nominal and Design Flexural Strengths (M_n, and ϕM_n)

Nominal moment capacity M_n of a section is computed from internal forces at ultimate strain profile (when the extreme compressive fiber strain is equal to 0.003). Sections in flexure exhibit different modes of failure depending on the strain level in the extreme tension reinforcement. Tension-controlled sections have strains either equal to or in excess of 0.005 (Section 10.3.4 of ACI 318-05). Compression-controlled sections have strains equal to or less than the yield strain, which is equal to 0.002 for Grade 60 reinforcement (Section 10.3.3 of ACI 318-05). There exists a transition region between the tension-controlled and compression-controlled sections (Section 10.3.4 of ACI 318-05). Tension-controlled sections are desirable for their ductile behavior, which allows redistribution of stresses and sufficient warning against an imminent failure. It is always a good practice to design reinforced concrete elements to behave in a ductile manner, whenever possible. This can be ensured by limiting the amount of reinforcement such that the tension reinforcement yields prior to concrete crushing. Section 10.3.5 of ACI 318-05 limits the strain in extreme tension reinforcement to 0.004 or greater. The amount of reinforcement corresponding to this level of strain defines the maximum amount of tension reinforcement that balances compression concrete. The ACI Code requires a lower strength reduction factor (ϕ-factor) for sections in the transition zone to allow for increased safety in sections with reduced ductility. Figure 1-2 illustrates the variation of ϕ-factors with tensile strain in reinforcement for Grade 60 steel, and the corresponding strain profiles at ultimate.

The ACI 318-05 Code has adopted strength reduction factors that are compatible with ASCE7-02 load combinations, except for the tension controlled section for which the ϕ-factor is increased from 0.80 to 0.90. These ϕ-factors appear in Chapter 9 of ACI 318-05. The ϕ-factors used in earlier editions of ACI 318 and the corresponding load factors have been moved to Appendix C of ACI 318-05. The designer has the option of using either the ϕ-factors in the main body of the Code (Chapter 9) or those given in Appendix C, so long as ϕ-factors are used with the corresponding load factors. The basic design inequality remains the same, irrespective of which pair of ϕ and load factors is used:

Factored (ultimate) moment $\leq$ Reduced (design) strength

$$M_u \leq \phi M_n$$

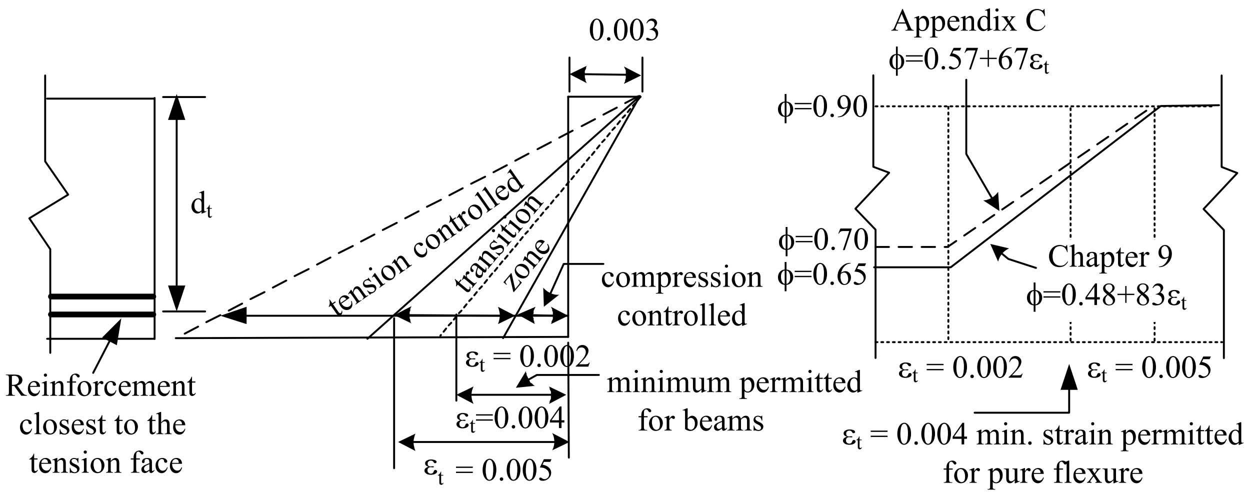


Fig. 1-2 Capacity reduction (ϕ) factors for Grade 60 reinforcement

1.2.1 Rectangular Sections with Tension Reinforcement

Nominal moment capacity of a rectangular section with tension reinforcement is computed from the internal force couple shown in Fig. 1-1. The required amount of reinforcement is computed from the equilibrium of forces. This computation becomes easier for code permitted sections where the tension steel yields prior to the compression concrete reaching its assumed failure strain of 0.003. Design aids **Flexure 1** through **Flexure 4** (included at the end of the chapter) were developed using this condition. Accordingly;

$$T = C \tag{1-1}$$

$$A_s f_y = 0.85 f'_c \, \beta_1 c b \tag{1-2}$$

$$\rho = \frac{0.85 f'_c \, \beta_1 c}{d f_y} \tag{1-3}$$

where;
$$\rho = \frac{A_s}{bd} \tag{1-4}$$

The c/d ratio in Eq. (1-3) can be written in terms of the steel strain ε_s illustrated in Fig. 1-1. For sections with single layer of tension reinforcement, d = d_t and $\varepsilon_s = \varepsilon_t$. The c/d ratio for this case becomes;

$$\frac{c}{d} = \frac{c}{d_t} = \frac{0.003}{0.003 + \varepsilon_t} \tag{1-5}$$

$$\rho = \frac{0.85 f_c' \beta_1}{f_y} \frac{0.003}{0.003 + \varepsilon_t} \tag{1-6}$$

Eq. (1-6) was used to generate the values for reinforcement ratio ρ (%) in **Flexure 1** through **Flexure 4** for sections with single layer of tension reinforcement. For other sections, where the centroid of tension reinforcement does not necessarily coincide with the centroid of extreme tension layer, the ρ values given in **Flexure 1** through **Flexure 4** should be multiplied by the ratio d_t/d.

The nominal moment capacity is computed from the internal force couple as illustrated below:

$$M_n = A_s f_y (d - \frac{\beta_1 c}{2}) \tag{1-7}$$

From Eq. (1-2);

$$\beta_1 c = \frac{A_s f_y}{0.85 f'_c\, b} \tag{1-8}$$

$$M_n = bd^2 \left[1 - \frac{\rho f_y}{1.7 f_c'} \right] \rho f_y \tag{1-9}$$

$$M_n = bd^2 K_n \tag{1-10}$$

Where;

$$K_n = \left[1 - \frac{\rho f_y}{1.7 f_c'} \right] \rho f_y \tag{1-11}$$

Flexure 1 through **Flexure 4** contains ϕK_n values computed by Eq. (1-11), where the ϕ-factor is obtained from Fig. 1-2 for selected values of ε_t listed in the design aids.

Design Examples 1 through 4 illustrate the application of **Flexure 1** to **Flexure 4**.

1.2.2 Rectangular Sections with Compression Reinforcement

Flexural members are designed for tension reinforcement. Any additional moment capacity required in the section is usually provided by increasing the section size or the amount of tension reinforcement. However, the cross-sectional dimensions in some applications may be limited by architectural or functional requirements, and the extra moment capacity may have to be provided by additional tension and compression reinforcement. The extra steel generates an internal force couple, adding to the sectional moment capacity without changing the ductility of the section. In such cases, the total moment capacity consists of two components; i) moment due to the tension reinforcement that balances the compression concrete, and ii) moment generated by the internal steel force couple consisting of compression reinforcement and equal amount of additional tension reinforcement, as illustrated in Fig. 1-3.

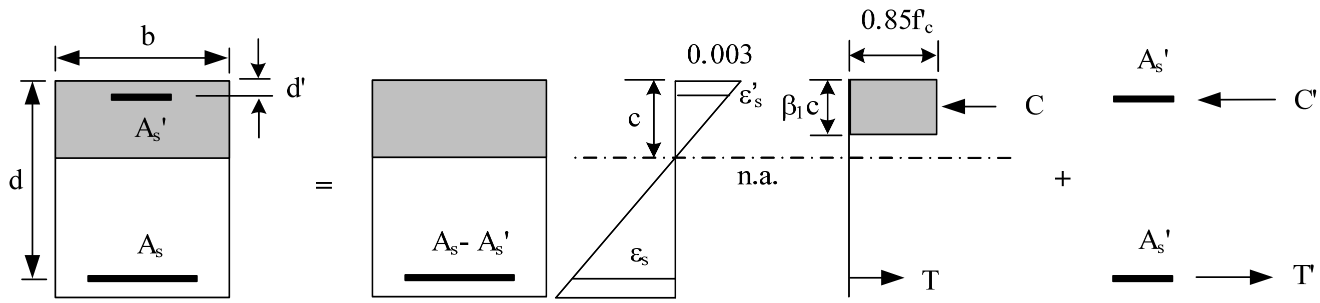


Fig. 1-3 A rectangular section with compression reinforcement

$$M_n = M_1 + M_2 \tag{1-12}$$

$$M_1 = K_n b d^2 \tag{1-13}$$

$$M_2 = A_s' f'_s (d - d') \tag{1-14}$$

Assuming f'_s is equal to or greater than f_y;

$$M_2 = K_n' b d^2 \tag{1-15}$$

where;

$$K_n' = \rho' f_y (1 - \frac{d'}{d}) \tag{1-16}$$

and

$$\rho' = \frac{A_s'}{bd} \tag{1-17}$$

Since the steel couple does not involve a force in concrete, it does not affect the ductility of the section, i.e., adding more tension steel over and above the maximum permitted by the Code does not create an over-reinforced section, and is permissible by ACI 318-05 so long as an equal amount is placed in the compression zone. This approach is employed for design, as well as in generating **Flexure 5**, which provides the amount of compression reinforcement. The underlining assumption in computing the steel force couple is that the steel in compression is at or near yield, developing compressive stress equal to the tensile yield strength. While this assumption is true in most heavily reinforced sections since the compression reinforcement is near the extreme compression fiber with a strain of 0.003, especially for Grade 60 steel with 0.002 yield strain, it is possible to design sections with non-yielding compression reinforcement. The designer, in this case, has to adjust (increase) the amount of compression reinforcement in proportion to the ratio of yield strength to compression steel stress. The strain in compression steel ε'_s can be computed from Fig. 1-3 as $\varepsilon'_s = \varepsilon_s$ (c-d')/(d-d'), once ε_s is determined from flexural design tables for sections with tension reinforcement (**Flexure 1** through **Flexure 4**) to assess if the compression steel is yielding.

The application of **Flexure 5** is illustrated in **Design Example 5**.

1.2.3 T-Sections

Most concrete slabs are cast monolithically with supporting beams, with portions of the slab participating in flexural resistance of the beams. The resulting one-way structural system has a T-section. The flange of a T-section is formed by the effective width of the slab, as defined in Section 8.10 of ACI 318-05, and also illustrated in **Flexure 6**. The rectangular beam forms the web of the T-section. T-sections may also be produced by the precast industry as single and double T's because of their superior performance in positive moment regions. A T-section provides increased area of compression concrete in the flange, where it is needed under positive bending, with reduced dead load resulting from the reduced area of tension concrete in the web.

The flange width in most T-sections is significantly wider than the web width. Therefore, the amount of tension reinforcement placed in the web can easily be equilibrated by a portion of the flange concrete in compression, placing the neutral axis in the flange. Therefore, most T-sections behave as rectangular sections, even though they have T geometry, and are designed using **Flexure 1 through Flexure 4** as rectangular sections with section widths equal to flange widths.

Rarely, the required amount of tension reinforcement in the web (or the applied moment) is high enough to bring the neutral axis below the flange, creating an additional compression zone in the web. In such a case, the section behaves as a T-section with total moment capacity consisting of components due to; i) compression concrete in the overhangs (b-b_w) and a portion of total tension steel balancing the overhangs, ρ_f and ii) the remaining tension steel, ρ_w balancing the web concrete. The condition for T-section behavior is expressed below:

$$M_u > \phi[0.85 f_c' b h_f (d - \frac{h_f}{2})] \qquad (1\text{-}18)$$

The components of moment for T-section behavior are illustrated in Fig. 1-4, and are expressed below.

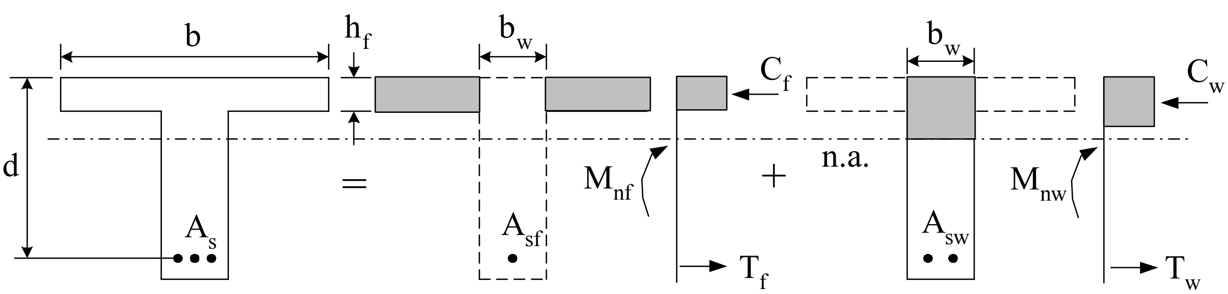


Fig. 1-4 T-section behavior

$$M_n = M_{nf} + M_{nw} \qquad (1\text{-}19)$$

$$M_{nf} = K_{nf}(b - b_w)d^2 \qquad (1\text{-}20)$$

$$M_{nw} = K_{nw} b_w d^2 \qquad (1\text{-}21)$$

and;
$$\rho_f = \frac{A_{sf}}{(b-b_w)d} \tag{1-22}$$

$$\rho_w = \frac{A_{sw}}{b_w d} \tag{1-23}$$

Moment components, M_{nf} and M_{nw} can be obtained from **Flexure 1 though Flexure 4** when the tables are entered with ρ_f and ρ_w values, respectively. For design, however, ρ_f needs to be found first and this can be done from the equilibrium of internal forces for the portion of total tension steel balancing the overhang concrete. This is illustrated below.

$$T_f = C_f \tag{1-24}$$

$$A_{sf} f_y = 0.85 f_c' h_f (b - b_w) \tag{1-25}$$

$$\rho_f = \frac{0.85 f_c'}{f_y} \frac{h_f}{d} \tag{1-26}$$

Eq. (1-26) was used to generate **Flexure 7 and Flexure 8**. **Flexure Example 6** through **Flexure Example 8** illustrate the use of **Flexure 7** and **Flexure 8**.

When T-beam flanges are in tension, part of the flexural tension reinforcement is required to be distributed over an effective area as illustrated in **Flexure 6** or a width equal to one-tenth the span, whichever is smaller (Sec. 10.6.6). This requirement is intended to control cracking that may result from widely spaced reinforcement. If one-tenth of the span is smaller than the effective width, *additional* reinforcement shall be provided in the outer portions of the flange to minimize wide cracks in these regions.

1.3 Minimum Flexural Reinforcement

Reinforced concrete sections that are larger than required for strength, for architectural and other functional reasons, may need to be protected by minimum amount of tension reinforcement against a brittle failure immediately after cracking. Reinforcement in a section becomes effective only after the cracking of concrete. However, if the area of reinforcement is too small to generate a sectional capacity that is less than the cracking moment, the section can not sustain its strength upon cracking. To safeguard against such brittle failures, ACI 318 requires a minimum area of tension reinforcement both in positive and negative moment regions (Sec. 10.5.1).

$$A_{s,\min} = \frac{3\sqrt{f_c'}}{f_y} b_w d \geq 200 b_w d / f_y \tag{1-27}$$

The above requirement is indicated in **Flexure 1** through **Flexure 4** by a horizontal line above which the reinforcement ratio ρ is less than that for minimum reinforcement.

When the flange of a T-section is in tension, the minimum reinforcement required to have a sectional capacity that is above the cracking moment is approximately twice that required for rectangular sections. Therefore, Eq. (1-27) is used with b_w replaced by $2b_w$ or the width of the flange, whichever is smaller (Sec. 10.5.2). If the area of steel provided in every section of a member is high enough to provide at least one-third greater flexural capacity than required by analysis, then the minimum steel requirement need not apply (Sec. 10.5.3). This exception prevents the use of excessive reinforcement in very large members that have sufficient reinforcement.

For structural slabs and footings, minimum reinforcement is used for shrinkage and temperature-control (Sec. 10.5.4). The minimum area of such reinforcement is 0.0018 times the gross area of concrete for Grade 60 deformed bars (Sec. 7.12.2.1). Where higher grade reinforcement is used, with yield stress measured at 0.35% strain, the minimum reinforcement ratio is proportionately adjusted as $(0.0018 \times 60{,}000)/f_y$. The maximum spacing of shrinkage and temperature reinforcement is limited to three times the slab or footing thickness or 18 in, whichever is smaller (Sec. 10.5.4).

1.4 Placement of Reinforcement in Sections

Flexural reinforcement is placed in a section with due considerations given to the spacing of reinforcement, crack control and concrete cover. It is usually preferable to use sufficient number of small size bars, as opposed to fewer bars of larger size, while also respecting the spacing requirements.

1.4.1 Minimum Spacing of Longitudinal Reinforcement

Longitudinal reinforcement should be placed with sufficient spacing to allow proper placement of concrete. The minimum spacing requirement for beam reinforcement is shown in **Flexure 9**.

1.4.2 Concrete Protection for Reinforcement

Flexural reinforcement should be placed to maximize the lever arm between internal forces for increased moment capacity. This implies that the main longitudinal reinforcement should be placed as close to the concrete surface as possible. However, the reinforcement should be protected against corrosion and other aggressive environments by a sufficiently thick concrete cover (Sec. 7.7), as indicated in **Flexure 9**. The cover concrete should also satisfy the requirements for fire protection (Sec. 7.7.7).

1.4.3 Maximum Spacing of Flexural Reinforcement and Crack Control

Beams reinforced with few large size bars may experience cracking between the bars, even if the required area of tension reinforcement is provided and the sectional capacity is achieved. Crack widths in these members may exceed what is usually regarded as acceptable limits of cracking for various exposure conditions. ACI 318-05 specifies a maximum spacing limit "s" for reinforcement closest to the tension face. This limit is specified in Eq. (1-28) to ensure proper crack control.

$$s = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c \leq 12\left(\frac{40{,}000}{f_s}\right) \tag{1-28}$$

where; c_c is the least distance from the surface of reinforcement to the tension face of concrete, and f_s is the service load stress in reinforcement. f_s can be computed from strain compatibility analysis under unfactored service loads. In lieu of this analysis, f_s may be taken as 2/3 f_y. Eq. (1-28) does not provide sufficient crack control for members subject to very aggressive exposure conditions or designed to be watertight. For such structures, special investigation is required (Sec. 10.6.5).

The maximum spacing of flexural reinforcement for one-way slabs and footings is limited to three times the slab or footing thickness or 18 in, whichever is smaller (Sec. 10.5.4).

1.4.4 Skin Reinforcement

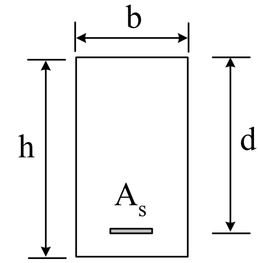
In deep flexural members, the crack control provided by the above expression may not be sufficient to control cracking near the mid-depth of the section, between the neutral axis and the tension concrete. For members with a depth $h > 36$ in, skin reinforcement with a maximum spacing of s, as defined in Eq. (1-28) and illustrated in **Flexure 10** is needed (Sec. 10.6.7). In this case, c_c is the least distance from the surface of the skin reinforcement to the side face. ACI 318 does not specify the area of steel required as skin reinforcement. However, research has indicated that bar sizes of No. 3 to No. 5 or welded wire reinforcement with a minimum area of 0.1 square inches per foot of depth provide sufficient crack control[2].

[2] Frosch, R.J., "Modeling and Control of Side Face Beam Cracking," ACI Structural Journal, V. 99, No.3, May-June 2002, pp. 376-385.

1.5 Flexure Examples

FLEXURE EXAMPLE 1 - **Calculation of area of tension reinforcement for a rectangular tension controlled cross-section.**

For a rectangular section subjected to a factored bending moment M_u, determine the required area of tension reinforcement for the dimensions given. Assume interior construction not exposed to weather.

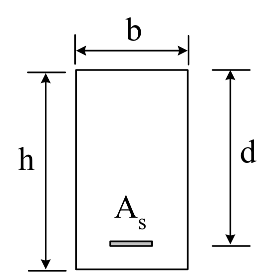


Given:
M_u = 90 kip-ft
f_c' = 4,000 psi
f_y = 60,000 psi
b = 10 in
h = 20 in

Procedure	**Calculation**	**ACI 318-05 Section**	**Design Aid**
Estimate "d" by allowing for clear cover, the radius of longitudinal reinforcement and diameter of stirrups.	Considering a minimum clear cover of 1.5 inches for interior exposure, allow 2.50 in to the centroid of main reinforcement d = 20 - 2.50 = 17.50 in	7.7.1	Flexure 9
Compute $\phi K_n = M_u$ x 12,000 / (bd^2)	ϕK_n = 90x12,000/[10x$(17.50)^2$] = 353 psi		
Select ρ from Flexure 1 Compute required area of steel; $A_s = \rho bd$ Determine the provided area of steel (For placement of reinforcement see Flexure Example 9)	For ϕK_n = 353 psi; ρ = 0.70% $A_s = \rho bd$ = 0.0070x10x17.5 = 1.22 in^2 Use 3 #6 $(A_s)_{prov.}$ = (3)(0.44) = 1.32 in^2 (Note: 3 # 6 can be placed within a 10 in. width). $(\rho)_{prov}$ = (1.32)/[(10)(17.5)] = 0.75% Note: for $(\rho)_{prov}$ = 0.75%; ε_t = 0.0163 ε_t = 0.0163 > 0.005 "tension controlled" section and ϕ = 0.9.	 7.6.1 3.3.2 10.3.4 9.3.2	Flexure 1 Flexure 9 Flexure 1

FLEXURE EXAMPLE 2 - **Calculation of nominal flexural capacity of a rectangular beam subjected to positive bending.**

For a rectangular section with specified tension reinforcement and geometry determine the nominal flexural capacity M_n.



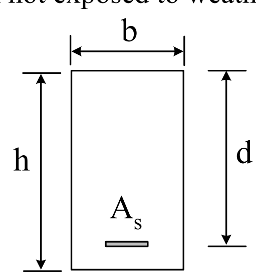
Given:
3 #6 Bars as bottom tension reinforcement
f_c' = 4,000 psi
f_y = 60,000 psi
b = 10 in
d = 18 in

Procedure	Calculation	ACI 318 2005 Section	Design Aid
Compute the area and percentage of steel provided	$A_s = 3 \times 0.44 = 1.32\ in^2$ $\rho = A_s/bd = 1.32/(10)(18) = 0.73\%$		
Select ϕK_n from Flexure 1 Compute $\phi M_n = \phi K_n\ bd^2 /12{,}000$ Select corresponding ϕ from Flexure 1 Compute $M_n = \phi M_n / \phi$	For $\rho = 0.73\%$; $\phi K_n = 370$ psi $\phi M_n = 370 \times 10 \times (18)^2 / 12{,}000 = 100$ k-ft $\phi = 0.9$ ($\varepsilon_t = 0.01675 > 0.005$ "tension controlled") $M_n = 100/0.9 = 111$ k-ft	 10.3.4 9.3.2	Flexure 1 Flexure 1

FLEXURE EXAMPLE 3 - Calculation of area of tension reinforcement for a rectangular cross section in the transition zone.

For a rectangular section subjected to a factored bending moment M_u, determine the required area of tension reinforcement for the dimensions given. Assume interior construction not exposed to weather.

Given:
$M_u = 487$ kip-ft
$f_c' = 4{,}000$ psi
$f_y = 60{,}000$ psi
$b = 14$ in
$h = 26$ in
1.0 in. maximum aggregate size



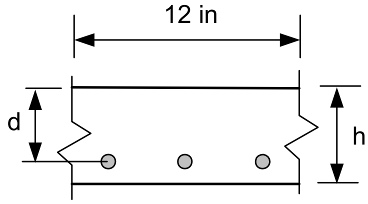
Procedure	Calculation	ACI 318 2005 Section	Design Aid
Estimate "d" by allowing for clear cover, the radius of longitudinal reinforcement and diameter of stirrups.	Considering a minimum clear cover of 1.5 in for interior exposure, allow 2.50 in. to the centroid of main reinforcement $d = 26 - 2.50 = 23.50$ in	7.7.1	Flexure 9
Compute $\phi K_n = M_u \times 12{,}000 / (bd^2)$	$\phi K_n = 487 \times 12{,}000/[14 \times (23.50)^2] = 756$ psi		
Select ρ from Flexure 1 Compute $A_s = \rho bd$ Determine the area of steel provided	For $\phi K_n = 756$ psi; $\rho = 1.63\%$ $A_s = \rho bd = 0.0163 \times 14 \times 23.5 = 5.36\ in^2$ Try #8 bars; 5.36 /0.79 = 6.8 Need 7 #8 bars in a single layer. However 7 #8 bars can not be placed in a single layer within a 14 in. width without violating the spacing limits. Therefore, try placing them in double layers. Allow 3.5 in. from the extreme tension fiber to the centroid of double layers of reinforcement. Revise d = 26 – 3.5 = 22.5 in.	7.6.1 3.3.2	Flexure 1 Flexure 9

	$\phi K_n = 487 x 12{,}000/[14x(22.50)^2] = 825$ psi For $\phi K_n = 825$ psi; $\rho = 1.98\%$ $A_s = \rho bd = 0.0198x14 \times 22.5 = 6.24$ in^2 Try #8 bars; 6.24 /0.79 = 7.9 Select 8 #8 bars in two layers (4 # 8 in each layer). Note that 4 #8 bars can be placed within a 14 in. width. $(A_s)_{prov.} = (8)(0.79) = 6.32$ in^2 $(\rho)_{prov.} = 6.32 / [(14)(22.5)] = 0.020$ For $(\rho)_{prov.} = 0.020$ $\phi K_n = 826$ psi $\varepsilon_t = 0.0042$ <u>Note:</u> $\varepsilon_t = 0.0042 < 0.005$ "transition zone"; $\phi = 0.83$ and $\phi K_n > M_u$	7.6.1 3.3.2 10.3.4 9.3.2	Flexure 1 Flexure 9 Flexure 1 Flexure 1

FLEXURE EXAMPLE 4 - Selection of slab thickness and area of flexural reinforcement.

For a slab subject to a factored bending moment M_u, determine the thickness h and required area of tension reinforcement. The slab has interior exposure.

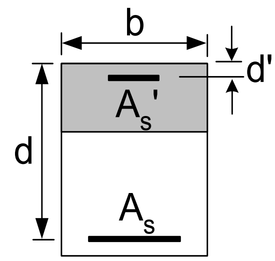
<u>**Given:**</u>
$M_u = 11$ kip-ft/ft
$f_c' = 4{,}000$ psi
$f_y = 60{,}000$ psi



Procedure	Calculation	ACI 318 2005 Section	Design Aid
Unless a certain slab thickness is desired, a trial thickness can be selected such that a section with good ductility, stiffness and bar placement characteristics is obtained. Try ρ = 50% of ρ at max. limit of tension controlled section.	$\rho = 0.5(\rho \text{ at } \varepsilon_t = 0.005)$ $= 0.5 \times 0.018 = 0.0091$ for $\rho = 0.0091$; $\phi K_n = 453$ psi $\phi K_n = M_u \times 12{,}000 / (bd^2)$ $d^2 = M_u \times 12{,}000/(\phi K_n\, b)$ $d^2 = 11 \times 12{,}000/(453 \times 12) = 24.3$ in^2 d = 5.0 in		Flexure 1
Select bar size and cover concrete. (For reinforcement placement see Flexure Example 10).	$A_s = \rho bd = 0.0091 (12)(5.0) = 0.55$ in^2 #5 at 6 in. provides $A_s = 0.62$ in.^{2} O.K. Cover = 0.75 in	7.7.1	
Compute h with due considerations given to cover and bar radius. Note that the slab thickness must also satisfy deflection control. (For placement of reinforcement see Flexure Example 10)	$h = d + d_b/2 + \text{cover} = 5.0 + 0.625/2 + 0.75$ h = 6.1 in Use h = 6.5 in *<u>Note:</u>* The slab thickness should be checked to satisfy the requirements of Table 9.5(a) for deflection control.	9.5.2 and Table 9.5(a)	

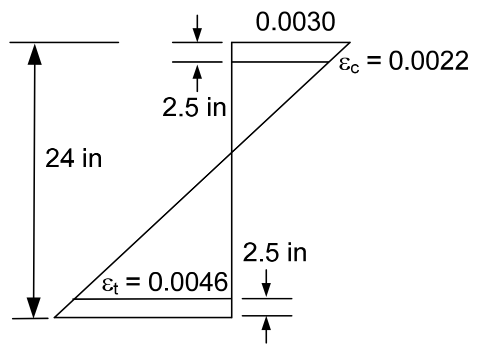
FLEXURE EXAMPLE 5 - Calculation of tension and compression reinforcement area for a rectangular beam section, subjected to positive bending.

For a rectangular section subjected to a factored positive moment M_u, determine the required area of tension and compression reinforcement for the dimensions given below.



<u>Given:</u>
M_u = 580 kip-ft
f_c' = 4,000 psi
f_y = 60,000 psi
b = 14 in
h = 24 in
d' = 2.5 in

Procedure	Calculation	ACI 318 2005 Section	Design Aid
Estimate "d" by allowing for clear cover, the radius of longitudinal reinforcement and diameter of stirrups.	Considering a minimum clear cover of 1.5 in for interior exposure, allow 2.5 in. to the centroid of main reinforcement. d = 24 - 2.5 = 21.5 in	7.7.1	Flexure 9
Compute $\phi K_n = M_u$ x 12,000 / (bd^2)	ϕK_n = 580x12,000/[14x$(21.5)^2$] =1075 psi		
Select ρ from Flexure 1	ϕK_n = 1075 psi is outside the range of Flexure 1. This indicates that the amount of steel needed exceeds the maximum allowed if only tension steel were to be provided. Therefore, compression steel is needed.		Flexure 1
Compute (A_s-A_s') In this problem select a reinforcement ratio close to the maximum allowed to take advantage of the full capacity of compression concrete. Select ρ = 1.80 % (slightly below ρ_{max} = 2.06% so that when the actual bars are placed ρ_{max} is not exceeded). Compute moment to be resisted by	Select; ρ = 0.018 (ε_t = 0.005) A_s-A_s'= ρbd = 0.018 x 14 x 21.5 = 5.42 in^2 Try #8 bars; 5.42 / 0.79 = 6.9 Select 7 #8 bars for A_s-A_s' However, 7 # 8 can not be placed in a single layer. Try using double layers. Allow 3.5 in. from the extreme tension fiber to the centroid of double layers of # 8 bars. Revise d = 24 – 3.5 = 20.5 in A_s-A_s'= ρbd = 0.018 x 14 x 20.5 = 5.17 in^2 Try #8 bars; 5.17 / 0.79 = 6.5 Select 7 #8 bars for (A_s-A'$_s$) to be placed in double layers. A_s-A_s' = (7)(0.79) = 5.53 in.^{2} Corresponding ρ = 5.53/[(14)(20.5)] = 0.019 < ρ_{max} = 0.0206 O.K. For ρ = 0.019 ϕK_n = 823 psi; ε_t = 0.0046	7.6.1 3.3.2 10.3.4 9.3.2	Flexure 1 Flexure 9 Flexure 1

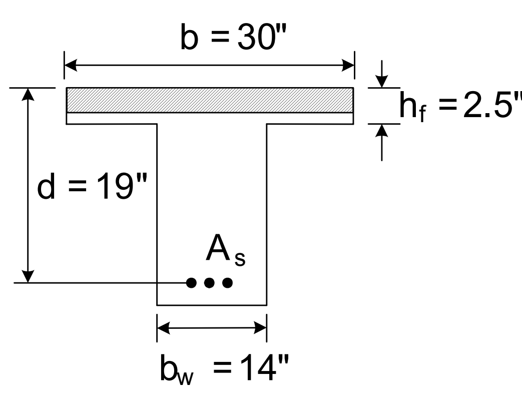
compression concrete and corresponding tension steel (A_s-A_s').	and $\phi = 0.87$ $\phi M_n = \phi K_n\, b\, d^2 / 12000$ $\phi M_n = 823 x 14\ (20.5)^2 / 12{,}000 = 404$ k-ft		
Compute moment to be resisted by the steel couple (with an equal tension and compression steel area of A_s')	$\phi M_n' = M_u - \phi M_n$ $\phi M_n' = 580 - 404 = 176$ k-ft		
Compute A_s' Note: As' is determined from Flexure 5, which was developed based on the assumption that at ultimate the compression steel is at or near yield. The strain diagram shown indicates the yielding of compression steel ($f'_s = f'_y$). If the compression steel does not yield ($f'_s < f'_y$) then the area of compression steel used should be reduced by f'_s/f'_y. Determine the area of compression steel provided.	$\phi K_n' = \phi M_n'$ x 12,000 / (bd^2) $\phi K_n' = 176 x 12{,}000 / [14 \text{ x } (20.5)^2] = 359$ psi $K_n' = 359/0.87 = 413$ psi $d'/d = 2.5/20.5 = 0.12$; $\rho' = 0.78\%$ $A_s' = \rho' bd = 0.0078 \text{ x } 14 \text{ x } 20.5 = 2.24$ in^2  Use 3 #8 bars as compression reinforcement. $(A_s')_{prov.} = (3)(0.79)$ = 2.37 in^2		Flexure 5
Add equal area of steel to the bottom bars to facilitate steel force couple. Determine the total area of bottom reinforcement, A_s	Add 3 #8 bars to the bottom reinforcement. Total bottom reinforcement: 7 #8 + 3 # 8 = 10 # 8 to be provided in double layers (5 #8 in each layer). 5 #8 can be placed within a width of 14 in. $A_s = 5.53 + 2.37 = 7.90$ in^2 *Note:* For this section: $\varepsilon_t = 0.00460$ and $\phi = 0.87$	7.6.1 3.3.2	Flexure 9

FLEXURE EXAMPLE 6 - Calculation of tension reinforcement area for a T beam section subjected to positive bending, behaving as a rectangular section.

For a T section subjected to a factored bending moment M_u, determine the required area of tension reinforcement for the dimensions given.

Given:

$M_u = 230$ kip-ft
$f_c' = 4{,}000$ psi
$f_y = 60{,}000$ psi
$b = 30$ in
$b_w = 14$ in
$d = 19$ in
$h_f = 2.5$ in

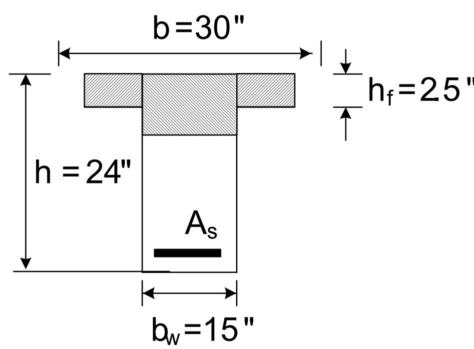


Procedure	Calculation	ACI 318 2005 Section	Design Aid
Assume tension controlled section (ϕ = 0.9). Determine if the section behaves as a T or a rectangular section. If $M_u > \phi[0.85f'_c\ b\ h_f\ (d-h_f/2)]$ T-section, otherwise rectangular section behavior.	$0.9\ [0.85f'_c b\ h_f\ (d - h_f/2)]$ = 0.9[0.85(4)(30)(2.5)(19-2.5/2)] = 4073 k-in = 340 k-ft > M_u = 230 k-ft Therefore, the neutral axis is within the flange and the section behaves as a rectangular section with width b = 30 in.		
Compute $\phi K_n = M_u$ x 12,000 / (bd^2)	$\phi K_n = (230)(12{,}000)/[(30)(19)^2] = 255$ psi		
Select ρ from Flexure 1 Compute $A_s = \rho bd$ Find provided area of steel. Read ε_t and ϕ from Flexure 1.	For ϕK_n = 255 psi; ρ = 0.50 % $A_s = \rho bd$ = 0.0050x30x19 = 2.85 in^2 Use 5 #7 with A_s = (5)(0.6) = 3.00 in^2 ρ = 3.00 / [(30)(19)] = 0.0053 For ρ = 0.0053; ε_t = 0.025 > 0.005 "tension controlled" section and ϕ = 0.9	10.3.4 9.3.2	Flexure 1 Flexure 1

FLEXURE EXAMPLE 7 - Computation of the area of tension reinforcement for a T beam section, subjected to positive bending, behaving as a tension controlled T-section.

For a T section subjected to a factored bending moment M_u, determine the required area of tension reinforcement for the dimensions given. The beam has interior exposure.

Given:
M_u= 400 kip-ft
f_c' = 4,000 psi
f_y = 60,000 psi
b = 30 in
b_w =15 in
h = 24 in
h_f = 2.5 in
Max. aggregate size = 1.0 in.



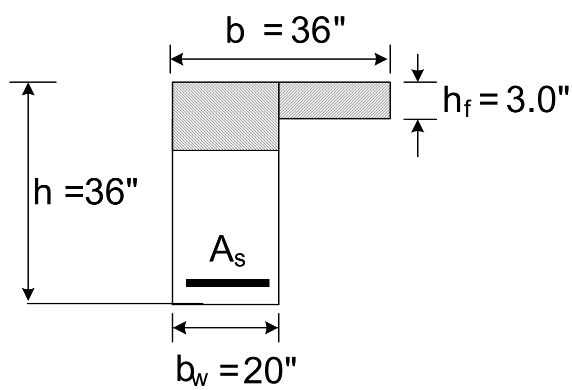
Procedure	Calculation	ACI 318 2005 Section	Design Aid
Estimate "d" by allowing for clear cover, the radius of longitudinal reinforcement and diameter of stirrups.	Considering a minimum clear cover of 1.5 in. for interior exposure, allow 2.5 in. to the centroid of longitudinal reinforcement. d = 24 - 2.5 = 21.5 in	7.7.1	Flexure 9
Assume tension controlled section and determine if the section behaves as a T section or a rectangular section. If $M_u > \phi[0.85f'_c\ b\ h_f\ (d-h_f/2)]$ T-section, otherwise rectangular section behavior.	$0.9\ [0.85f'_c b\ h_f\ (d - h_f/2)]$ = 0.9(0.85)(4)(30)(2.5)(21.5-2.5/2) = 4647 k-in = 387 k-ft < M_u = 400 k-ft Therefore, the neutral axis is below the flange and the section behaves as a T section.		

Compute the amount of steel that balances compression concrete in flange overhangs from Flexure 7.	$d/h_f = 21.5/2.5 = 8.6$ $\rho_f = 0.66\%$		Flexure 7
Find the amount of moment resisted by ρ_f from Flexure 1.	For $\rho_f = 0.66\%$ $\phi K_n = 334$ psi and $\phi = 0.9$ $\phi M_f = \phi K_n (b-b_w)d^2/12{,}000$ $= 334(30 - 15)(21.5)^2/12{,}000 = 193$ k-ft		Flexure 1
Determine the amount of steel needed to resist the remaining moment, that is to be resisted by the web; ρ_w	$\phi M_w = M_u - \phi M_f = 400 - 193 = 207$ k-ft $\phi K_n = \phi M_w \times 12{,}000 / [(b_w)(d)^2] = 358$ psi for $\phi K_n = 358$ psi; $\rho_w = 0.71\%$		Flexure 1
Compute the total area of tension reinforcement	$A_f = \rho_f (b-b_w)d = 0.0066(30-15)(21.5)$ $= 2.13$ in^2 $A_w = \rho_w b_w d = 0.0071(15)(21.5) = 2.29$ in^2 $A_s = A_f + A_w = 2.13 + 2.29 = 4.42$ in^2 Try using # 9 bars; 4.42/1.00 = 4.42 Use 5 #9 bars in a single layer $(A_s)_{prov.} = (5)(1.0) = 5.00$ in^2	7.6.1 3.3.2	Flexure 9
Note: The ϕ factor can be computed using the reinforcement ratio that balances web concrete.	Provided area of steel that balances web concrete: 5.00 – 2.13 = 2.87 in^2 $(\rho_w)_{prov.} = 2.87/(15)(21.5) = 0.0089$ This corresponds to $\varepsilon_t = 0.0132$ and $\phi = 0.9$ (tension controlled section)	10.3.4 9.3.2	Flexure 1

FLEXURE EXAMPLE 8 - Calculation of the area of tension reinforcement for an L beam section, subjected to positive bending, behaving as an L-section in the transition zone.

For an L section subjected to a factored bending moment M_u, determine the required area of tension reinforcement for the dimensions given. The beam has interior exposure.

<u>Given:</u>
$M_u = 1800$ kip-ft
$f_c' = 4{,}000$ psi
$f_y = 60{,}000$ psi
$b = 36$ in
$b_w = 20$ in
$h = 36$ in
$h_f = 3.0$ in
Max. aggregate size = 3/4 in.



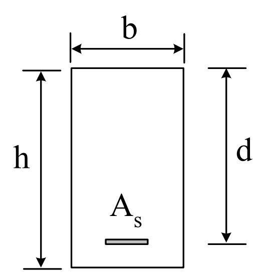
Procedure	Calculation	ACI 318 2005 Section	Design Aid
Estimate "d" by allowing for clear cover, diameter of stirrups and the radius of longitudinal reinforcement.	Considering a minimum clear cover of 1.5 in. for interior exposure, allow 2.5 in. to the centroid of main reinforcement. $d = 36 - 2.5 = 33.5$ in	7.7.1	Flexure 9

Assume tension controlled section and determine if the section behaves as a T-section (in this case as an L-section) or a rectangular section. If $M_u > \phi\ [0.85f'_c\ b\ h_f\ (d-h_f/2)]$ T-section otherwise rectangular section behavior.	$0.9\ (0.85)f'_c b\ h_f\ (d - h_f/2)$ $= 0.9(0.85)(4)(36)(3.0)(33.5-3.0/2)$ = 10,575 k-in = 881 k-ft < M_u = 1800 k-ft Therefore, the neutral axis is below the flange and the section behaves as a T section.		
Compute the amount of steel that balances compression concrete in the flange overhang, from Flexure 7	$d/h_f = 33.5/3.0 = 11.2$ $\rho_f = 0.51\ \%$		Flexure 7
Find the amount of moment resisted by ρ_f from Flexure 1.	For $\rho_f = 0.51\ \%$ $\phi K_n = 261$ psi ($\phi = 0.90$) $\phi M_f = \phi K_n\ (b-b_w)d^2/12{,}000$ $= 261\ (36 - 20)(33.5)^2/12{,}000 = 391$ k-ft		Flexure 1
Determine the amount of steel required to resist the remaining moment. This additional moment is to be resisted by the web; ρ_w	$\phi M_w = M_u - \phi M_f = 1800 - 391 = 1409$ k-ft $\phi K_n = \phi M_w \times 12{,}000 / [(b_w)(d)^2]$ $\phi K_n = 1409 \times 12{,}000/[(20)(33.5)^2] = 753$ psi for $\phi K_n = 753$ psi; $\rho_w = 1.63\ \%$ Note: $\phi = 0.90$ (tension controlled).		Flexure 1
Compute the total area of tension reinforcement.	$A_f = \rho_f\ (b-b_w)d = 0.0051(36-20)(33.5)$ $= 2.73\ in^2$ $A_w = \rho_w\ b_w d = 0.0163(20)(33.5) = 10.92\ in^2$ $A_s = A_f + A_w = 2.73 + 10.92 = 13.65\ in^2$ Select #9 bars; 14 - #9 bars are needed. 14 - #9 bars can not be placed in a single layer. Therefore, use double layers of reinforcement and revise the design.	7.6.1 3.3.2	Flexure 9
Recalculate the effective depth "d" and revise design. Assume cover of 3.5 in. to the centroid of double layers of reinforcement.	d = 36 – 3.5 = 32.5 in. Note: Reduced "d" will result in increased area of steel and the beam will continue behaving as a T-Beam (no need to check again).		
Compute the amount of steel that balances compression concrete in the flange overhang, from Flexure 7	$d/h_f = 32.5/3.0 = 10.8$ $\rho_f = 0.53\ \%$		Flexure 7
Find the amount of moment resisted by ρ_f from Flexure 1.	For $\rho_f = 0.53\ \%$ $\phi K_n = 271$ psi ($\phi = 0.90$) $\phi M_f = \phi K_n\ (b-b_w)d^2/12{,}000$ $= 271\ (36 - 20)(32.5)^2/12{,}000 = 382$ k-ft		Flexure 1
Determine the amount of steel required to resist the remaining moment. This additional moment is to be resisted by the web; ρ_w	$\phi M_w = M_u - \phi M_f = 1800 - 382 = 1418$ k-ft $\phi K_n = \phi M_w \times 12{,}000 / [(b_w)(d)^2]$ $\phi K_n = 1418 \times 12{,}000/[(20)(32.5)^2] = 806$psi for $\phi K_n = 806$ psi; $\rho_w = 1.77\ \%$ Note: $\phi = 0.90$.		Flexure 1
Compute the total area of tension reinforcement required.	$A_f = \rho_f\ (b-b_w)d = 0.0053(36-20)(32.5)$ $= 2.76\ in^2$ $A_w = \rho_w\ b_w d = 0.0177(20)(32.5) = 11.51\ in^2$ $A_s = A_f + A_w = 2.76 + 11.51 = 14.27\ in^2$ Use 16 #9 bars in two layers (8 #9 in each layer, which can be placed within 20 in. width.	7.6.1 3.3.2	Flexure 9

Ensure $\phi M_n \geq M_u$ based on provided reinforcement.	$A_w = A_s - A_f = 16.0 - 2.76 = 13.24$ in^2 Provided ρ_w that balances web concrete; $\rho_w = 13.24 / [(20)(32.5)] = 0.0204 = 2.04\%$ For $\rho_w = 2.04$ %; $\phi K_n = 827$ and $\phi = 0.82$ $\phi M_w = \phi\ K_n\ b_w d^2 / 12{,}000$ $= (827)(20)(32.5)^2 / 12{,}000 = 1456$ k-ft For the contribution of flange overhang $(0.90)K_n = 271$ psi (found earlier) $(0.82)K_n = 271\ (0.82/0.90) = 247$ psi $\phi M_f = \phi K_n\ (b-b_w)d^2/12{,}000$ $\phi M_f = (247)(36-20)(32.5)^2 / 12{,}000 = 348$k-ft $\phi M_n = \phi M_w + \phi M_f = 1456 + 348 = 1804$ k-ft $\phi M_n = 1804$ k-ft $> M_u = 1800$ k-ft O.K.		Flexure 1

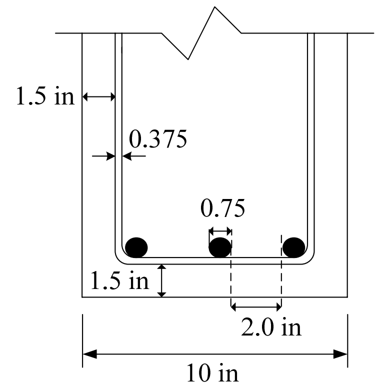
FLEXURE EXAMPLE 9 - Placement of reinforcement in the rectangular beam section designed in Flexure Example 1.

Select and place flexural beam reinforcement in the section provided below, with due considerations given to spacing and cover requirements.



<u>Given:</u>
$A_s = 1.22$ in^2
$b = 10$ in
$h = 20$ in
$f_y = 60{,}000$ psi
#3 stirrups
Max. aggregate size: 3/4 in
Interior exposure

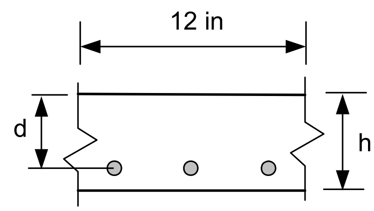
Procedure	Calculation	ACI 318-05 Section	Design Aid
Determine bar size and number of bars.	Select # 6 bars; No. of bars = 1.22/0.44 = 2.8 Use 3 # 6 bars		Appendix A
Determine bar spacing.	Considering minimum clear cover of 1.5 inches on each side for interior exposure and allowing 2 stirrup bar diameters; $s = [10 - 2(1.5) - 2(0.375) - 3(0.75)]/ 2 = 2.0$ in	7.7.1	Flexure 9 Appendix A
Check against minimum spacing	$(s)_{min} = \{d_b;\ 1\frac{1}{3}\ a_{max};\ 1\text{ in}\}$ $(s)_{min} = \{0.75\text{ in};\ 1\frac{1}{3}(3/4\text{ in});\ 1\text{ in}\} = 0.75$ in s = 2.0 in > 0.75 in O.K.	7.6	Flexure 9
Check against maximum spacing as governed by crack control	$(s)_{maz} = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c \leq 12\left(\frac{40{,}000}{f_s}\right)$ $f_s = 2/3 f_y = 2/3\ (60{,}000) = 40{,}000$ psi	10.6.4	Eq.(1-28)

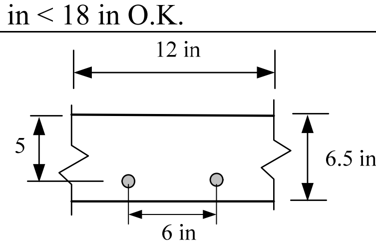
	$c_c = (1.5 + 0.375) = 1.875$ in $(s)_{max} = 15\,(1.0) - 2.5\,(1.875) = 10.3$ in $s = 2.0$ in < 12 in O.K.		
Final bar placement	Provide 3 # 6 as indicated below.  		Flexure 9

FLEXURE EXAMPLE 10 - Placement of reinforcement in the slab section designed in Example 4.

Select and place reinforcement in the 6 in slab shown below.

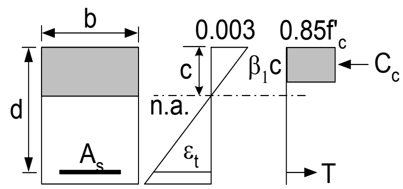
<u>Given:</u>
$A_s = 0.62$ in^2/ft
$f_y = 60{,}000$ psi
$d = 5$ in
$h = 6.5$ in



Procedure	Calculation	ACI 318 2005 Section	Design Aid
Determine bar size and number of bars for a one-foot slab width.	Select # 5 bars; No. of bars = 0.62/0.31= 2 Use 2 # 5 bars per foot of slab width.		Appendix A
Check for minimum area of reinforcement needed for temperature and shrinkage control. Note that the same minimum reinforcement must also be provided in the transverse direction.	For Grade 60 steel $A_{s,min} = 0.0018\, A_g$ $A_{s,min} = 0.0018\,(6.0)(12.0) = 0.13$ in^2 $A_s = 2 \times 0.31 = 0.62 > 0.13$ O.K.	7.12.2.1	
Check for maximum spacing of reinforcement.	2 # 5 bars per foot results in s = 6 in $(s)_{max} = 3h$ or 18 in, whichever is smaller $(s)_{max} = 3(6) = 18$ in $s = 6$ in < 18 in O.K.	10.5.4	
Final reinforcement placement	 Note clear cover = (6.5 – 5) – 0.625/2 = 1.2 in > ¾ in O.K.	7.7.1	Flexure 9

1.6 Flexure Design Aids

Flexure 1 - Flexural coefficients for rectangular beams with tension reinforcement, f_y = 60,000 psi



$\phi M_n \geq M_u$ $\quad \phi M_n = \phi K_n \, b \, d^2 / 12000$ $\quad \rho = A_s / b\,d$

where, M_n is in kip-ft; K_n is in psi; b and d are in inches

f_y = 60000 psi

f_c' (psi) :			3000		4000		5000		6000	
β_1 :			0.85		0.85		0.80		0.75	
ρ_{min} :			0.0033		0.0033		0.0035		0.0039	
ε_t	ϕ	$\phi_{App\ C}$	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)
0.20000	0.90	0.90	0.05	29	0.07	38	0.08	45	0.09	51
0.15000	0.90	0.90	0.07	38	0.09	51	0.11	60	0.13	67
0.10000	0.90	0.90	0.11	56	0.14	75	0.17	88	0.19	99
0.07500	0.90	0.90	0.14	74	0.19	98	0.22	116	0.25	130
0.05000	0.90	0.90	0.20	108	0.27	144	0.32	169	0.36	191
0.04000	0.90	0.90	0.25	132	0.34	176	0.40	208	0.44	234
0.03500	0.90	0.90	0.29	149	0.38	198	0.45	234	0.50	264
0.03000	0.90	0.90	0.33	170	0.44	227	0.52	268	0.58	302
0.02500	0.90	0.90	0.39	199	0.52	266	0.61	314	0.68	354
0.02000	0.90	0.90	0.47	240	0.63	320	0.74	378	0.83	427
0.01900	0.90	0.90	0.49	251	0.66	334	0.77	395	0.87	445
0.01800	0.90	0.90	0.52	262	0.69	349	0.81	412	0.91	465
0.01700	0.90	0.90	0.54	274	0.72	365	0.85	431	0.96	487
0.01600	0.90	0.90	0.57	287	0.76	383	0.89	453	1.01	511
0.01500	0.90	0.90	0.60	302	0.80	403	0.94	476	1.06	538
0.01400	0.90	0.90	0.64	318	0.85	425	1.00	502	1.13	567
0.01300	0.90	0.90	0.68	337	0.90	449	1.06	531	1.20	600
0.01250	0.90	0.90	0.70	347	0.93	462	1.10	546	1.23	618
0.01200	0.90	0.90	0.72	357	0.96	476	1.13	563	1.28	637
0.01150	0.90	0.90	0.75	368	1.00	491	1.17	581	1.32	657
0.01100	0.90	0.90	0.77	380	1.03	507	1.21	600	1.37	678
0.01050	0.90	0.90	0.80	393	1.07	523	1.26	620	1.42	701
0.01000	0.90	0.90	0.83	406	1.11	541	1.31	641	1.47	726
0.00950	0.90	0.90	0.87	420	1.16	561	1.36	664	1.53	752
0.00900	0.90	0.90	0.90	436	1.20	581	1.42	689	1.59	780

Flexure 1 - (Cont'd)

f_y = 60000 psi

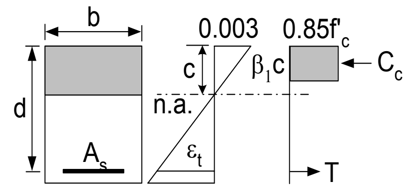
f_c' (psi) :			3000		4000		5000		6000	
β_1 :			0.85		0.85		0.80		0.75	
ρ_{min} :			0.0033		0.0033		0.0035		0.0039	
ε_t	ϕ	$\phi_{App\ C}$	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)
0.00870	0.90	0.90	0.93	446	1.24	594	1.45	704	1.63	798
0.00840	0.90	0.90	0.95	456	1.27	608	1.49	720	1.68	817
0.00810	0.90	0.90	0.98	467	1.30	622	1.53	738	1.72	836
0.00770	0.90	0.90	1.01	482	1.35	642	1.59	762	1.79	864
0.00740	0.90	0.90	1.04	494	1.39	658	1.63	781	1.84	886
0.00710	0.90	0.90	1.07	506	1.43	675	1.68	801	1.89	909
0.00680	0.90	0.90	1.11	519	1.47	693	1.73	822	1.95	933
0.00650	0.90	0.90	1.14	533	1.52	711	1.79	844	2.01	958
0.00620	0.90	0.90	1.18	548	1.57	731	1.85	868	2.08	985
0.00590	0.90	0.90	1.22	563	1.62	751	1.91	892	2.15	1014
0.00560	0.90	0.90	1.26	580	1.68	773	1.98	918	2.22	1044
0.00530	0.90	0.90	1.31	597	1.74	796	2.05	946	2.30	1076
0.00500	0.90	0.90	1.35	615	1.81	820	2.13	975	2.39	1109
0.00480	0.88	0.89	1.39	616	1.85	821	2.18	977	2.45	1112
0.00460	0.87	0.87	1.43	617	1.90	823	2.24	979	2.52	1115
0.00440	0.85	0.86	1.46	618	1.95	824	2.30	982	2.58	1118
0.00430	0.84	0.85	1.48	619	1.98	825	2.33	983	2.62	1119
0.00420	0.83	0.85	1.51	619	2.01	826	2.36	984	2.66	1121
0.00410	0.82	0.84	1.53	620	2.04	827	2.39	985	2.69	1122
0.00400	0.82	0.83	1.55	620	2.06	827	2.43	986	2.73	1124

Notes: - The vaues of ρ above the rule are less than ρ_{min}.
- ϕK_n values are based on ϕ factors specified in Chapter 9 of ACI 318-05. When App. C values of ϕ are used, ϕK_n values in the transition zone may be up to 2.4% higher (more conservative)

Flexure 2 - Flexural coefficients for rectangular beams with tension reinforcement, f_y = 60,000 psi

$\phi M_n \geq M_u$ $\phi M_n = \phi K_n b d^2 / 12000$ $\rho = A_s / b d$

Where, M_n is in kip-ft; b and d are in inches



f_y = 60000 psi

f_c' (psi) :			**7000**		**8000**		**9000**		**10000**	
β_1 :			0.7		0.65		0.65		0.65	
ρ_{min} **:**			0.0042		0.0045		0.0047		0.0050	
ε_t	ϕ	ϕ_{Appc}	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**
0.20000	0.90	0.90	0.10	55	0.11	59	0.12	66	0.14	73
0.15000	0.90	0.90	0.14	73	0.14	78	0.16	87	0.18	97
0.10000	0.90	0.90	0.20	108	0.21	115	0.24	129	0.27	143
0.07500	0.90	0.90	0.27	142	0.28	151	0.32	170	0.35	189
0.05000	0.90	0.90	0.39	208	0.42	221	0.47	249	0.52	276
0.04000	0.90	0.90	0.48	255	0.51	271	0.58	305	0.64	339
0.03500	0.90	0.90	0.55	288	0.58	306	0.65	344	0.73	382
0.03000	0.90	0.90	0.63	330	0.67	351	0.75	395	0.84	439
0.02500	0.90	0.90	0.74	387	0.79	411	0.89	463	0.99	514
0.02000	0.90	0.90	0.91	467	0.96	497	1.08	559	1.20	621
0.01900	0.90	0.90	0.95	487	1.00	518	1.13	583	1.26	648
0.01800	0.90	0.90	0.99	509	1.05	542	1.18	610	1.32	677
0.01700	0.90	0.90	1.04	533	1.11	568	1.24	639	1.38	710
0.01600	0.90	0.90	1.10	559	1.16	596	1.31	670	1.45	745
0.01500	0.90	0.90	1.16	588	1.23	627	1.38	705	1.53	784
0.01400	0.90	0.90	1.23	621	1.30	662	1.46	744	1.63	827
0.01300	0.90	0.90	1.30	657	1.38	700	1.55	788	1.73	876
0.01250	0.90	0.90	1.34	676	1.43	722	1.60	812	1.78	902
0.01200	0.90	0.90	1.39	697	1.47	744	1.66	837	1.84	930
0.01150	0.90	0.90	1.44	719	1.52	768	1.71	864	1.91	960
0.01100	0.90	0.90	1.49	743	1.58	793	1.78	892	1.97	991
0.01050	0.90	0.90	1.54	768	1.64	820	1.84	923	2.05	1025
0.01000	0.90	0.90	1.60	795	1.70	849	1.91	955	2.13	1061
0.00950	0.90	0.90	1.67	824	1.77	880	1.99	990	2.21	1100
0.00900	0.90	0.90	1.74	855	1.84	914	2.07	1028	2.30	1142

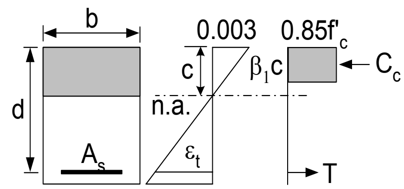
Flexure 2 - Cont'd

f_y = 60000 psi

f_c' (psi) :			**7000**		**8000**		**9000**		**10000**	
β_1 :			0.7		0.65		0.65		0.65	
ρ_{min} :			0.0042		0.0045		0.0047		0.0050	
ε_t	ϕ	**$\phi_{App\ C}$**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**
0.00870	0.90	0.90	1.78	875	1.89	935	2.13	1052	2.36	1169
0.00840	0.90	0.90	1.83	896	1.94	957	2.18	1077	2.42	1197
0.00810	0.90	0.90	1.88	917	1.99	981	2.24	1103	2.49	1226
0.00770	0.90	0.90	1.95	948	2.07	1014	2.32	1140	2.58	1267
0.00740	0.90	0.90	2.00	972	2.13	1040	2.39	1170	2.66	1300
0.00710	0.90	0.90	2.06	998	2.19	1068	2.46	1201	2.74	1334
0.00680	0.90	0.90	2.13	1025	2.26	1097	2.54	1234	2.82	1371
0.00650	0.90	0.90	2.19	1053	2.33	1127	2.62	1268	2.91	1409
0.00620	0.90	0.90	2.26	1083	2.40	1160	2.70	1305	3.00	1450
0.00590	0.90	0.90	2.34	1114	2.48	1194	2.79	1343	3.10	1493
0.00560	0.90	0.90	2.42	1148	2.57	1230	2.89	1384	3.21	1538
0.00530	0.90	0.90	2.51	1183	2.66	1269	3.00	1428	3.33	1586
0.00500	0.90	0.90	2.60	1221	2.76	1310	3.11	1474	3.45	1637
0.00480	0.88	0.89	2.67	1225	2.83	1314	3.19	1478	3.54	1642
0.00460	0.87	0.87	2.74	1228	2.91	1318	3.27	1483	3.63	1648
0.00440	0.85	0.86	2.81	1232	2.99	1322	3.36	1488	3.73	1653
0.00430	0.84	0.85	2.85	1233	3.03	1325	3.41	1490	3.78	1656
0.00420	0.83	0.85	2.89	1235	3.07	1327	3.45	1493	3.84	1659
0.00410	0.82	0.84	2.93	1237	3.11	1329	3.50	1495	3.89	1661
0.00400	0.82	0.83	2.98	1239	3.16	1332	3.55	1498	3.95	1664

Notes: - The vaues of ρ above the rule are less than ρ_{min}.
- ϕK_n values are based on ϕ factors specified in Chapter 9 of ACI 318-05. When App. C values of ϕ are used, ϕK_n values in the transition zone may be up to 2.4% higher (more conservative)

Flexure 3 - Flexural coefficients for rectangular beams with tension reinforcement, f_y = 75,000 psi



$\phi M_n \geq M_u$ $\phi M_n = \phi K_n\, b\, d^2 / 12000$ $\rho = A_s / b\, d$

where, M_n is in kip-ft; b and d are in inches

f_y = 75000 psi

f_c' (psi) :			3000		4000		5000		6000	
β_1 :			0.85		0.85		0.80		0.75	
ρ_{min} :			0.0027		0.0027		0.0028		0.0031	
ε_t	ϕ	$\phi_{App\ C}$	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)
0.15000	0.90	0.90	0.06	38	0.08	51	0.09	60	0.10	67
0.10000	0.90	0.90	0.08	56	0.11	75	0.13	88	0.15	99
0.07500	0.90	0.90	0.11	74	0.15	98	0.17	116	0.20	130
0.05000	0.90	0.90	0.16	108	0.22	144	0.26	169	0.29	191
0.04000	0.90	0.90	0.20	132	0.27	176	0.32	208	0.36	234
0.03500	0.90	0.90	0.23	149	0.30	198	0.36	234	0.40	264
0.03000	0.90	0.90	0.26	170	0.35	227	0.41	268	0.46	302
0.02500	0.90	0.90	0.31	199	0.41	266	0.49	314	0.55	354
0.02000	0.90	0.90	0.38	240	0.50	320	0.59	378	0.67	427
0.01900	0.90	0.90	0.39	251	0.53	334	0.62	395	0.70	445
0.01800	0.90	0.90	0.41	262	0.55	349	0.65	412	0.73	465
0.01700	0.90	0.90	0.43	274	0.58	365	0.68	431	0.77	487
0.01600	0.90	0.90	0.46	287	0.61	383	0.72	453	0.81	511
0.01500	0.90	0.90	0.48	302	0.64	403	0.76	476	0.85	538
0.01400	0.90	0.90	0.51	318	0.68	425	0.80	502	0.90	567
0.01300	0.90	0.90	0.54	337	0.72	449	0.85	531	0.96	600
0.01250	0.90	0.90	0.56	347	0.75	462	0.88	546	0.99	618
0.01200	0.90	0.90	0.58	357	0.77	476	0.91	563	1.02	637
0.01150	0.90	0.90	0.60	368	0.80	491	0.94	581	1.06	657
0.01100	0.90	0.90	0.62	380	0.83	507	0.97	600	1.09	678
0.01050	0.90	0.90	0.64	393	0.86	523	1.01	620	1.13	701
0.01000	0.90	0.90	0.67	406	0.89	541	1.05	641	1.18	726
0.00950	0.90	0.90	0.69	420	0.92	561	1.09	664	1.22	752
0.00900	0.90	0.90	0.72	436	0.96	581	1.13	689	1.28	780

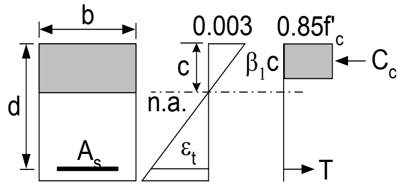
Flexure 3 - Cont'd

f_y = 75000 psi

f_c' (psi) :			3000		4000		5000		6000	
β_1 :			0.85		0.85		0.80		0.75	
ρ_{min} :			0.0027		0.0027		0.0028		0.0031	
ε_t	ϕ	$\phi_{App\ C}$	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)	ρ(%)	ϕK_n (psi)
0.00870	0.90	0.90	0.74	446	0.99	594	1.16	704	1.31	798
0.00840	0.90	0.90	0.76	456	1.01	608	1.19	720	1.34	817
0.00810	0.90	0.90	0.78	467	1.04	622	1.23	738	1.38	836
0.00770	0.90	0.90	0.81	482	1.08	642	1.27	762	1.43	864
0.00740	0.90	0.90	0.83	494	1.11	658	1.31	781	1.47	886
0.00710	0.90	0.90	0.86	506	1.14	675	1.35	801	1.51	909
0.00680	0.90	0.90	0.88	519	1.18	693	1.39	822	1.56	933
0.00650	0.90	0.90	0.91	533	1.22	711	1.43	844	1.61	958
0.00620	0.90	0.90	0.94	548	1.26	731	1.48	868	1.66	985
0.00590	0.90	0.90	0.97	563	1.30	751	1.53	892	1.72	1014
0.00560	0.90	0.90	1.01	580	1.34	773	1.58	918	1.78	1044
0.00530	0.90	0.90	1.04	597	1.39	796	1.64	946	1.84	1076
0.00500	0.90	0.90	1.08	615	1.45	820	1.70	975	1.91	1109
0.00480	0.88	0.89	1.11	616	1.48	821	1.74	977	1.96	1112
0.00460	0.87	0.87	1.14	617	1.52	823	1.79	979	2.01	1115
0.00440	0.85	0.86	1.17	618	1.56	824	1.84	982	2.07	1118
0.00430	0.84	0.85	1.19	619	1.58	825	1.86	983	2.10	1119
0.00420	0.83	0.85	1.20	619	1.61	826	1.89	984	2.13	1121
0.00410	0.82	0.84	1.22	620	1.63	827	1.92	985	2.15	1122
0.00400	0.82	0.83	1.24	620	1.65	827	1.94	986	2.19	1124

Notes: - The vaues of ρ above the rule are less than ρ_{min}.
- ϕK_n values are based on ϕ factors specified in Chapter 9 of ACI 318-05. When App. C values of ϕ are used, ϕK_n values in the transition zone may be up to 2.4% higher (more conservative)

Flexure 4 - Flexural coefficients for rectangular beams with tension reinforcement, f_y = 75,000 psi



$\phi M_n \geq M_u$ $\quad \phi M_n = \phi K_n\, b\, d^2 / 12000$ $\quad \rho = A_s / b\, d$

where, M_n is in kip-ft; b and d are in inches

f_y = 75000 psi

f_c' (psi) :			**7000**		**8000**		**9000**		**10000**	
β_1 :			0.7		0.65		0.65		0.65	
ρ_{min} **:**			0.0033		0.0036		0.0038		0.0040	
ε_t	ϕ	ϕ_{Appc}	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**
0.20000	0.90	0.90	0.08	55	0.09	59	0.10	66	0.11	73
0.15000	0.90	0.90	0.11	73	0.12	78	0.13	87	0.14	97
0.10000	0.90	0.90	0.16	108	0.17	115	0.19	129	0.21	143
0.07500	0.90	0.90	0.21	142	0.23	151	0.26	170	0.28	189
0.05000	0.90	0.90	0.31	208	0.33	221	0.38	249	0.42	276
0.04000	0.90	0.90	0.39	255	0.41	271	0.46	305	0.51	339
0.03500	0.90	0.90	0.44	288	0.47	306	0.52	344	0.58	382
0.03000	0.90	0.90	0.50	330	0.54	351	0.60	395	0.67	439
0.02500	0.90	0.90	0.60	387	0.63	411	0.71	463	0.79	514
0.02000	0.90	0.90	0.72	467	0.77	497	0.86	559	0.96	621
0.01900	0.90	0.90	0.76	487	0.80	518	0.90	583	1.00	648
0.01800	0.90	0.90	0.79	509	0.84	542	0.95	610	1.05	677
0.01700	0.90	0.90	0.83	533	0.88	568	0.99	639	1.11	710
0.01600	0.90	0.90	0.88	559	0.93	596	1.05	670	1.16	745
0.01500	0.90	0.90	0.93	588	0.98	627	1.11	705	1.23	784
0.01400	0.90	0.90	0.98	621	1.04	662	1.17	744	1.30	827
0.01300	0.90	0.90	1.04	657	1.11	700	1.24	788	1.38	876
0.01250	0.90	0.90	1.07	676	1.14	722	1.28	812	1.43	902
0.01200	0.90	0.90	1.11	697	1.18	744	1.33	837	1.47	930
0.01150	0.90	0.90	1.15	719	1.22	768	1.37	864	1.52	960
0.01100	0.90	0.90	1.19	743	1.26	793	1.42	892	1.58	991
0.01050	0.90	0.90	1.23	768	1.31	820	1.47	923	1.64	1025
0.01000	0.90	0.90	1.28	795	1.36	849	1.53	955	1.70	1061
0.00950	0.90	0.90	1.33	824	1.41	880	1.59	990	1.77	1100
0.00900	0.90	0.90	1.39	855	1.47	914	1.66	1028	1.84	1142

Flexure 4 – Cont'd

f_y = 75000 psi

f_c' (psi) :			**7000**		**8000**		**9000**		**10000**	
β_1 :			0.7		0.65		0.65		0.65	
ρ_{min} **:**			0.0033		0.0036		0.0038		0.0040	
ε_t	ϕ	$\phi_{App\,C}$	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**	ρ(%)	**ϕK_n (psi)**
0.00870	0.90	0.90	1.42	875	1.51	935	1.70	1052	1.89	1169
0.00840	0.90	0.90	1.46	896	1.55	957	1.74	1077	1.94	1197
0.00810	0.90	0.90	1.50	917	1.59	981	1.79	1103	1.99	1226
0.00770	0.90	0.90	1.56	948	1.65	1014	1.86	1140	2.07	1267
0.00740	0.90	0.90	1.60	972	1.70	1040	1.91	1170	2.13	1300
0.00710	0.90	0.90	1.65	998	1.75	1068	1.97	1201	2.19	1334
0.00680	0.90	0.90	1.70	1025	1.80	1097	2.03	1234	2.26	1371
0.00650	0.90	0.90	1.75	1053	1.86	1127	2.09	1268	2.33	1409
0.00620	0.90	0.90	1.81	1083	1.92	1160	2.16	1305	2.40	1450
0.00590	0.90	0.90	1.87	1114	1.99	1194	2.23	1343	2.48	1493
0.00560	0.90	0.90	1.94	1148	2.06	1230	2.31	1384	2.57	1538
0.00530	0.90	0.90	2.01	1183	2.13	1269	2.40	1428	2.66	1586
0.00500	0.90	0.90	2.08	1221	2.21	1310	2.49	1474	2.76	1637
0.00480	0.88	0.89	2.14	1225	2.27	1314	2.55	1478	2.83	1642
0.00460	0.87	0.87	2.19	1228	2.33	1318	2.62	1483	2.91	1648
0.00440	0.85	0.86	2.25	1232	2.39	1322	2.69	1488	2.99	1653
0.00430	0.84	0.85	2.28	1233	2.42	1325	2.72	1490	3.03	1656
0.00420	0.83	0.85	2.31	1235	2.46	1327	2.76	1493	3.07	1659
0.00410	0.82	0.84	2.35	1237	2.49	1329	2.80	1495	3.11	1661
0.00400	0.82	0.83	2.38	1239	2.53	1332	2.84	1498	3.16	1664

Notes: - The vaues of ρ above the rule are less than ρ_{min}.
- ϕK_n values are based on ϕ factors specified in Chapter 9 of ACI 318-05. When App. C values of ϕ are used, ϕK_n values in the transition zone may be up to 2.4% higher (more conservative)

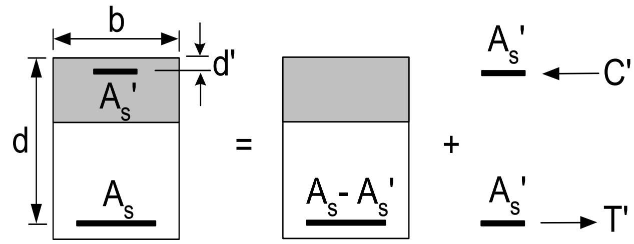
Flexure 5 - Reinforcement ratio (ρ') for compression reinforcement

$\phi M_n + \phi M_n' \geq M_u$

$\phi M_n = \phi K_n \, b \, d^2 / 12000$

$\rho = (A_s - A_s') / b \, d$

(From Flexure 1 through 4)



$\phi M_n' = \phi K_n' \, b \, d^2 / 12000$

$\rho' = A_s' / b \, d$

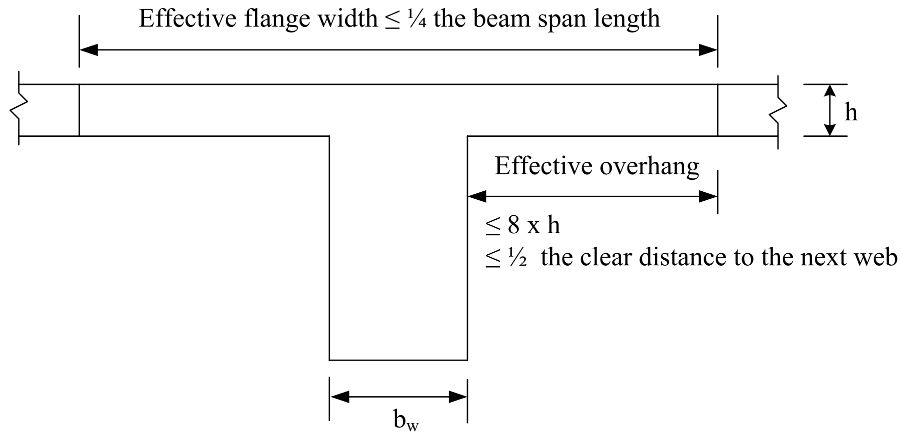
where, M_n' is in kip-ft; K_n' is in psi; b and d are in inches

f_y	60000 (psi)						75000 (psi)					
d/d'	0.02	0.06	0.1	0.14	0.18	0.22	0.02	0.06	0.1	0.14	0.18	0.22
K_n' (psi)	ρ' (%)						ρ' (%)					
20	0.03	0.04	0.04	0.04	0.04	0.04	0.03	0.03	0.03	0.03	0.03	0.03
40	0.07	0.07	0.07	0.08	0.08	0.09	0.05	0.06	0.06	0.06	0.07	0.07
60	0.10	0.11	0.11	0.12	0.12	0.13	0.08	0.09	0.09	0.09	0.10	0.10
80	0.14	0.14	0.15	0.16	0.16	0.17	0.11	0.11	0.12	0.12	0.13	0.14
100	0.17	0.18	0.19	0.19	0.20	0.21	0.14	0.14	0.15	0.16	0.16	0.17
120	0.20	0.21	0.22	0.23	0.24	0.26	0.16	0.17	0.18	0.19	0.20	0.21
140	0.24	0.25	0.26	0.27	0.28	0.30	0.19	0.20	0.21	0.22	0.23	0.24
160	0.27	0.28	0.30	0.31	0.33	0.34	0.22	0.23	0.24	0.25	0.26	0.27
180	0.31	0.32	0.33	0.35	0.37	0.38	0.24	0.26	0.27	0.28	0.29	0.31
200	0.34	0.35	0.37	0.39	0.41	0.43	0.27	0.28	0.30	0.31	0.33	0.34
220	0.37	0.39	0.41	0.43	0.45	0.47	0.30	0.31	0.33	0.34	0.36	0.38
240	0.41	0.43	0.44	0.47	0.49	0.51	0.33	0.34	0.36	0.37	0.39	0.41
260	0.44	0.46	0.48	0.50	0.53	0.56	0.35	0.37	0.39	0.40	0.42	0.44
280	0.48	0.50	0.52	0.54	0.57	0.60	0.38	0.40	0.41	0.43	0.46	0.48
300	0.51	0.53	0.56	0.58	0.61	0.64	0.41	0.43	0.44	0.47	0.49	0.51
320	0.54	0.57	0.59	0.62	0.65	0.68	0.44	0.45	0.47	0.50	0.52	0.55
340	0.58	0.60	0.63	0.66	0.69	0.73	0.46	0.48	0.50	0.53	0.55	0.58
360	0.61	0.64	0.67	0.70	0.73	0.77	0.49	0.51	0.53	0.56	0.59	0.62
380	0.65	0.67	0.70	0.74	0.77	0.81	0.52	0.54	0.56	0.59	0.62	0.65
400	0.68	0.71	0.74	0.78	0.81	0.85	0.54	0.57	0.59	0.62	0.65	0.68
420	0.71	0.74	0.78	0.81	0.85	0.90	0.57	0.60	0.62	0.65	0.68	0.72
440	0.75	0.78	0.81	0.85	0.89	0.94	0.60	0.62	0.65	0.68	0.72	0.75
460	0.78	0.82	0.85	0.89	0.93	0.98	0.63	0.65	0.68	0.71	0.75	0.79
480	0.82	0.85	0.89	0.93	0.98	1.03	0.65	0.68	0.71	0.74	0.78	0.82

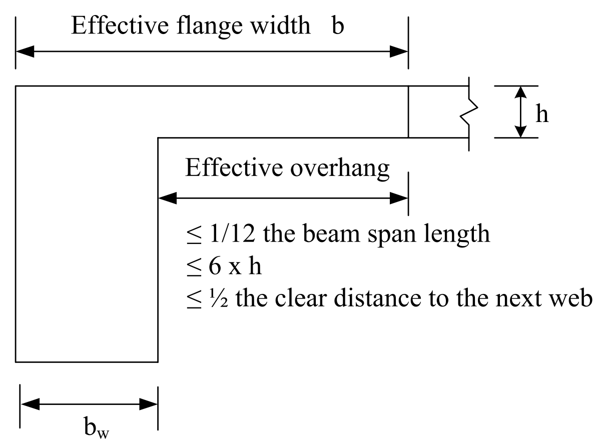
Flexure 5 - Cont'd

f_y	60000 (psi)						75000 (psi)					
d/d'	0.02	0.06	0.1	0.14	0.18	0.22	0.02	0.06	0.1	0.14	0.18	0.22
K_n' (psi)	ρ' (%)						ρ' (%)					
500	0.85	0.89	0.93	0.97	1.02	1.07	0.68	0.71	0.74	0.78	0.81	0.85
520	0.88	0.92	0.96	1.01	1.06	1.11	0.71	0.74	0.77	0.81	0.85	0.89
540	0.92	0.96	1.00	1.05	1.10	1.15	0.73	0.77	0.80	0.84	0.88	0.92
560	0.95	0.99	1.04	1.09	1.14	1.20	0.76	0.79	0.83	0.87	0.91	0.96
580	0.99	1.03	1.07	1.12	1.18	1.24	0.79	0.82	0.86	0.90	0.94	0.99
600	1.02	1.06	1.11	1.16	1.22	1.28	0.82	0.85	0.89	0.93	0.98	1.03
620	1.05	1.10	1.15	1.20	1.26	1.32	0.84	0.88	0.92	0.96	1.01	1.06
640	1.09	1.13	1.19	1.24	1.30	1.37	0.87	0.91	0.95	0.99	1.04	1.09
660	1.12	1.17	1.22	1.28	1.34	1.41	0.90	0.94	0.98	1.02	1.07	1.13
680	1.16	1.21	1.26	1.32	1.38	1.45	0.93	0.96	1.01	1.05	1.11	1.16
700	1.19	1.24	1.30	1.36	1.42	1.50	0.95	0.99	1.04	1.09	1.14	1.20
720	1.22	1.28	1.33	1.40	1.46	1.54	0.98	1.02	1.07	1.12	1.17	1.23
740	1.26	1.31	1.37	1.43	1.50	1.58	1.01	1.05	1.10	1.15	1.20	1.26
760	1.29	1.35	1.41	1.47	1.54	1.62	1.03	1.08	1.13	1.18	1.24	1.30
780	1.33	1.38	1.44	1.51	1.59	1.67	1.06	1.11	1.16	1.21	1.27	1.33
800	1.36	1.42	1.48	1.55	1.63	1.71	1.09	1.13	1.19	1.24	1.30	1.37
820	1.39	1.45	1.52	1.59	1.67	1.75	1.12	1.16	1.21	1.27	1.33	1.40
840	1.43	1.49	1.56	1.63	1.71	1.79	1.14	1.19	1.24	1.30	1.37	1.44
860	1.46	1.52	1.59	1.67	1.75	1.84	1.17	1.22	1.27	1.33	1.40	1.47

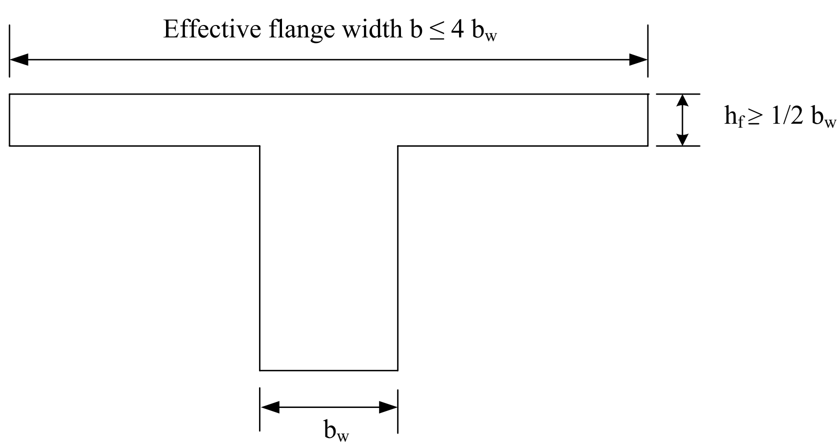
Flexure 6 - T-Beam construction and definition of effective flange width



T-sections resulting from monolotically built slabs and beams



L-sections resulting from monolotically built slabs and beams



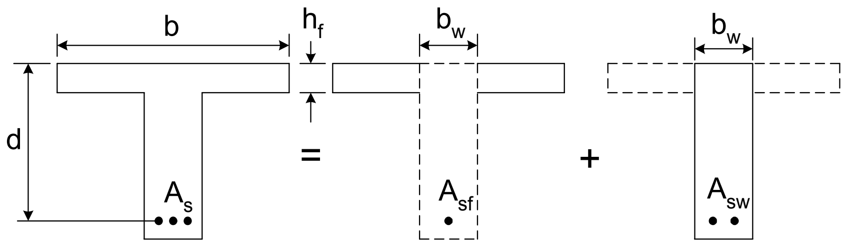
Isolated Precast T-Beams cast to have a T-shape

Flexure 7 - Reinforcement ratio ρ_f(%) balancing concrete in overhang(s) in T or L beams; f_y = 60,000 psi

$\phi M_{nf} + \phi M_{nw} \geq M_u$

$\rho_f = A_{sf}/[(b-b_w)d]$

$\rho_w = A_{sw}/(b_w d)$



$A_s = A_{sf} + A_{sw}$

Use Flexure 1 or 2 with ρ_f and $(b-b_w)$ to find ϕM_{nf} Use Flexure 1 or 2 with ρ_w and b_w to find ϕM_{nw}

f_y = 60000 psi

f_c' (psi) :	3000	4000	5000	6000	7000	8000	9000	10000
d/h_f	ρ_f (%)							
2	2.13	2.83	3.54	4.25	4.96	5.67	6.38	7.08
3	1.42	1.89	2.36	2.83	3.31	3.78	4.25	4.72
4	1.06	1.42	1.77	2.13	2.48	2.83	3.19	3.54
5	0.85	1.13	1.42	1.70	1.98	2.27	2.55	2.83
6	0.71	0.94	1.18	1.42	1.65	1.89	2.13	2.36
7	0.61	0.81	1.01	1.21	1.42	1.62	1.82	2.02
8	0.53	0.71	0.89	1.06	1.24	1.42	1.59	1.77
9	0.47	0.63	0.79	0.94	1.10	1.26	1.42	1.57
10	0.43	0.57	0.71	0.85	0.99	1.13	1.28	1.42
11	0.39	0.52	0.64	0.77	0.90	1.03	1.16	1.29
12	0.35	0.47	0.59	0.71	0.83	0.94	1.06	1.18
13	0.33	0.44	0.54	0.65	0.76	0.87	0.98	1.09
14	0.30	0.40	0.51	0.61	0.71	0.81	0.91	1.01
15	0.28	0.38	0.47	0.57	0.66	0.76	0.85	0.94
16	0.27	0.35	0.44	0.53	0.62	0.71	0.80	0.89
17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.83
18	0.24	0.31	0.39	0.47	0.55	0.63	0.71	0.79
19	0.22	0.30	0.37	0.45	0.52	0.60	0.67	0.75
20	0.21	0.28	0.35	0.43	0.50	0.57	0.64	0.71
21	0.20	0.27	0.34	0.40	0.47	0.54	0.61	0.67
22	0.19	0.26	0.32	0.39	0.45	0.52	0.58	0.64
23	0.18	0.25	0.31	0.37	0.43	0.49	0.55	0.62
24	0.18	0.24	0.30	0.35	0.41	0.47	0.53	0.59

Flexure 7 - Cont'd

f_y = 60000 psi

f_c' (psi) :	**3000**	**4000**	**5000**	**6000**	**7000**	**8000**	**9000**	**10000**
d/h_f	**ρ_f (%)**							
25	0.17	0.23	0.28	0.34	0.40	0.45	0.51	0.57
26	0.16	0.22	0.27	0.33	0.38	0.44	0.49	0.54
27	0.16	0.21	0.26	0.31	0.37	0.42	0.47	0.52
28	0.15	0.20	0.25	0.30	0.35	0.40	0.46	0.51
29	0.15	0.20	0.24	0.29	0.34	0.39	0.44	0.49
30	0.14	0.19	0.24	0.28	0.33	0.38	0.43	0.47
31	0.14	0.18	0.23	0.27	0.32	0.37	0.41	0.46
32	0.13	0.18	0.22	0.27	0.31	0.35	0.40	0.44
33	0.13	0.17	0.21	0.26	0.30	0.34	0.39	0.43
34	0.13	0.17	0.21	0.25	0.29	0.33	0.38	0.42
35	0.12	0.16	0.20	0.24	0.28	0.32	0.36	0.40
36	0.12	0.16	0.20	0.24	0.28	0.31	0.35	0.39
37	0.11	0.15	0.19	0.23	0.27	0.31	0.34	0.38
38	0.11	0.15	0.19	0.22	0.26	0.30	0.34	0.37
39	0.11	0.15	0.18	0.22	0.25	0.29	0.33	0.36
40	0.11	0.14	0.18	0.21	0.25	0.28	0.32	0.35

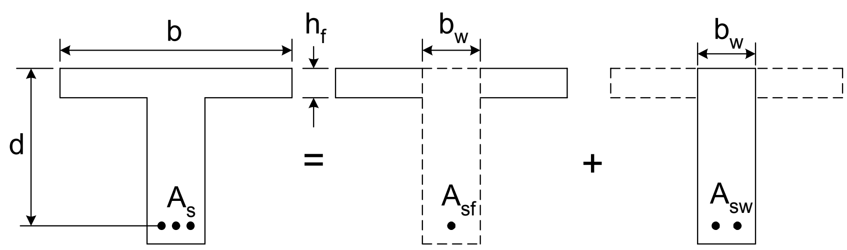
Flexure 8 - Reinforcement ratio ρ_f(%) balancing concrete in overhang(s) in T or L beams; f_y = 75,000 psi

$\phi M_{nf} + \phi M_{nw} \geq M_u$

$\rho_f = A_{sf}/[(b-b_w)d]$

$\rho_w = A_{sw}/(b_w d)$

$A_s = A_{sf} + A_{sw}$



Use Flexure 3 or 4 with ρ_f and $(b-b_w)$ to find ϕM_{nf}

Use Flexure 3 or 4 with ρ_w and b_w to find ϕM_{nw}

f_y = 75000 psi

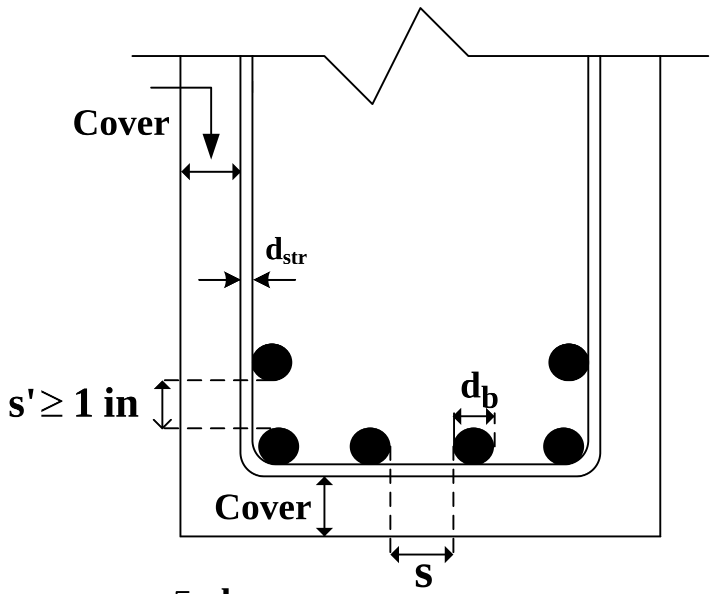
f_c' (psi) :	3000	4000	5000	6000	7000	8000	9000	10000
d/h_f	ρ_f (%)							
2	1.70	2.27	2.83	3.40	3.97	4.53	5.10	5.67
3	1.13	1.51	1.89	2.27	2.64	3.02	3.40	3.78
4	0.85	1.13	1.42	1.70	1.98	2.27	2.55	2.83
5	0.68	0.91	1.13	1.36	1.59	1.81	2.04	2.27
6	0.57	0.76	0.94	1.13	1.32	1.51	1.70	1.89
7	0.49	0.65	0.81	0.97	1.13	1.30	1.46	1.62
8	0.43	0.57	0.71	0.85	0.99	1.13	1.28	1.42
9	0.38	0.50	0.63	0.76	0.88	1.01	1.13	1.26
10	0.34	0.45	0.57	0.68	0.79	0.91	1.02	1.13
11	0.31	0.41	0.52	0.62	0.72	0.82	0.93	1.03
12	0.28	0.38	0.47	0.57	0.66	0.76	0.85	0.94
13	0.26	0.35	0.44	0.52	0.61	0.70	0.78	0.87
14	0.24	0.32	0.40	0.49	0.57	0.65	0.73	0.81
15	0.23	0.30	0.38	0.45	0.53	0.60	0.68	0.76
16	0.21	0.28	0.35	0.43	0.50	0.57	0.64	0.71
17	0.20	0.27	0.33	0.40	0.47	0.53	0.60	0.67
18	0.19	0.25	0.31	0.38	0.44	0.50	0.57	0.63
19	0.18	0.24	0.30	0.36	0.42	0.48	0.54	0.60
20	0.17	0.23	0.28	0.34	0.40	0.45	0.51	0.57

Flexure 8 - Cont'd

f_y = 75000 psi

f_c' (psi) :	3000	4000	5000	6000	7000	8000	9000	10000
d/h_f	ρ_f (%)							
21	0.16	0.22	0.27	0.32	0.38	0.43	0.49	0.54
22	0.15	0.21	0.26	0.31	0.36	0.41	0.46	0.52
23	0.15	0.20	0.25	0.30	0.34	0.39	0.44	0.49
24	0.14	0.19	0.24	0.28	0.33	0.38	0.43	0.47
25	0.14	0.18	0.23	0.27	0.32	0.36	0.41	0.45
26	0.13	0.17	0.22	0.26	0.31	0.35	0.39	0.44
27	0.13	0.17	0.21	0.25	0.29	0.34	0.38	0.42
28	0.12	0.16	0.20	0.24	0.28	0.32	0.36	0.40
29	0.12	0.16	0.20	0.23	0.27	0.31	0.35	0.39
30	0.11	0.15	0.19	0.23	0.26	0.30	0.34	0.38
31	0.11	0.15	0.18	0.22	0.26	0.29	0.33	0.37
32	0.11	0.14	0.18	0.21	0.25	0.28	0.32	0.35
33	0.10	0.14	0.17	0.21	0.24	0.27	0.31	0.34
34	0.10	0.13	0.17	0.20	0.23	0.27	0.30	0.33
35	0.10	0.13	0.16	0.19	0.23	0.26	0.29	0.32
36	0.09	0.13	0.16	0.19	0.22	0.25	0.28	0.31
37	0.09	0.12	0.15	0.18	0.21	0.25	0.28	0.31
38	0.09	0.12	0.15	0.18	0.21	0.24	0.27	0.30
39	0.09	0.12	0.15	0.17	0.20	0.23	0.26	0.29
40	0.09	0.11	0.14	0.17	0.20	0.23	0.26	0.28

Flexure - 9 Bar spacing and cover requirements



$$s \geq \begin{cases} d_b \\ 1\frac{1}{3}\, a_{max} \\ 1 \text{ in} \end{cases} \qquad a_{max} = \text{Max. aggregate size}$$

Minimum Cover for protection of reinforcement (Section 7.7.1)

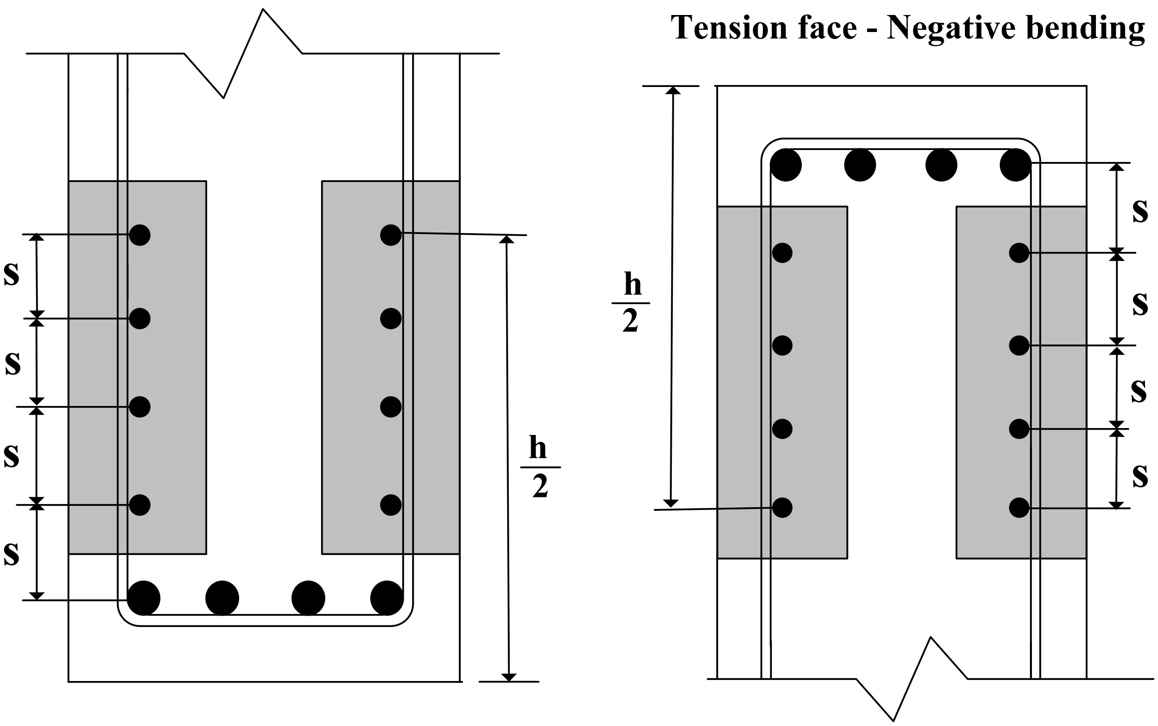
Not exposed to weather or in contact with ground	
Beams and columns	1 ½ in
Slabs, walls, joists with No. 11 and smaller bars	¾ in
Slabs, walls, joists with No. 14 and 18 bars	1 ½ in
Exposed to earth or weather	
Members with No. 5 and smaller bars	1 ½ in
Members with No. 6 through 18 bars	2 in
Cast against and permanently exposed to earth	3 in

Notes: i) The minimum cover is measured from the concrete surface to the outermost surface of stirrups; or to the outermost surface of main bars if more than one layer is used without stirrups.

ii) In corrosive environments or other severe exposure conditions, the amount of cover shall be suitably increased (Section 7.7.5).

ii) The minimum cover shall also satisfy the fire protection requirement (Section 7.7.7).

Flexure 10 - Skin Reinforcement



Tension face - Positive Bending