A. Steps in determining the Tension Steel Area As of a T-Beam with given Mu

I. Assume that the entire flange is in compression and solve for Mu1:

Compression force in concrete:

$$C = 0.85\ fc'\ b_f\ t$$

$$Mu1 = \phi C(d - t/2)$$

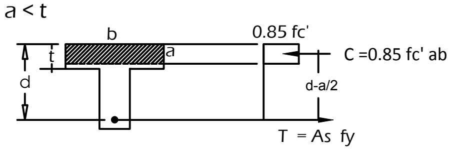
$$Mu1 = \phi 0.85\ fc'\ b_f t(d - t/2)$$

Mu1 = ______________

If Mu1 > Mu, then a < t, proceed to Step II

If Mu1 < Mu, then a > t, proceed to Step III

II. a < t



Solve for a:

$$Mu = \phi\ C\ (d\text{-}a/2)$$

$$Mu = \phi\ 0.85\ fc'\ ab\ (d\text{-}a/2)$$

a = _______

T = C

As fy = 0.85 fc' ab

As = _______

Solve for ρ_{max} and compare with $\frac{As}{b_f\,d}$

If $\frac{As}{b_f\,d}$ < ρ_{max}, design is OK!

If $\frac{As}{b_f\,d}$ > ρ_{max}, beam needs compression steel (seldom happen)

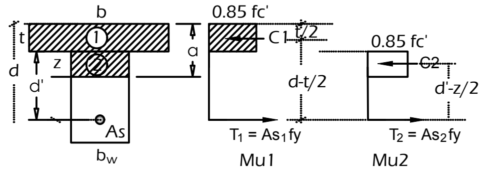
Solve fo ρ_{min} = 1.4 / fy and compare with $\frac{As}{b_w\,d}$

If $\frac{As}{b_w\,d}$ > ρ_{min}, design is OK!

If $\frac{As}{b_w\,d}$ < ρ_{min}, use $\rho = \rho_{min}$ (seldom)

$$\boxed{As = \rho_{min}\ b_w\ d}$$

III. a > t



Mu = Mu1 + Mu2

Mu1 = the same value in Step 1

Mu2 = Mu - Mu1

$$Mu2 = \phi\ C_2\ (d'\text{-}a/2)$$

$$Mu2 = \phi\ 0.85\ fc'\ b_w\ z\ (d'\text{-}z/2)$$

z = _______

T = C

As fy = C1 + C2

As fy = 0.85 fc' b t + 0.85 fc' b_wz

As = _______

Solve fo ρ_{min} = 1.4 / fy and compare with $\frac{As}{b_w\,d}$

If $\frac{As}{b_w\,d}$ > ρ_{min}, design is OK!

If $\frac{As}{b_w\,d}$ < ρ_{min}, use $\rho = \rho_{min}$(seldom)

$$As = \rho_{min}\ b_w\ d$$

Solve for As_{max}.

$$a = \beta_1 \frac{600\ d}{600 + fy}$$

$$As_{max} = 0.75\ A_{sb}$$

$$As_{max} = 0.75 \frac{0.85\ fc'\ (\ b_f\ t + (a\text{-}t)\ b_w}{fy}$$

If As < As_{max}, value is OK

If As > As_{max}, the beam needs compression steel (seldom happens)

B. Steps in Determining Mu of a T-Beam with given As.

I. Assume steel yields (fs = fy) and compute the area of compression concrete, Ac

C = T

0.85 fc' Ac = As fy

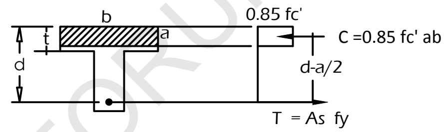
Ac = _____

Area of compression flange, Af = b_f t

If Ac < Af, a < t, proceed to Step II

If Ac > Af, a > t, proceed to Step III

II. a < t



Solve for a:

$$Ac = b_f \times a$$

a = _____

$$Mu = \phi\ As\ fy\ (d\text{-}a/2)$$

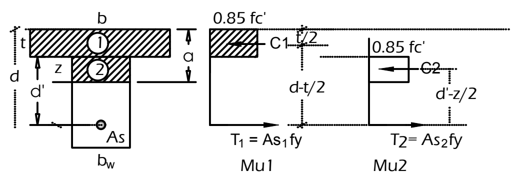
Verify if steel yields(this may not be necessary)

$c = a / \beta_1$ fs = 600 (d-c) / c

If fs > fy, steel yields (correct assumption)

If fs < fy, steel does not yield (seldom happen)

III. a > t



Solve for z:

$$Ac = A_f + b_w\ z$$

(see Steps I for values of Ac and Af)

Verify if steel yields:

a = t + z = _____

$c = a / \beta_1$ = _____ fs = 600(d-c)/c = _____

If fs > fy, steel yields (correct assumption)

If fs < fy, steel does not yield (seldom happen)

$$Mu1 = \phi\ C_1\ (d - t / 2)$$

$$Mu1 = \phi\ 0.85\ fc'\ A_f\ (d - t / 2)$$

$$Mu2 = \phi\ C_2\ (d' - z / 2)$$

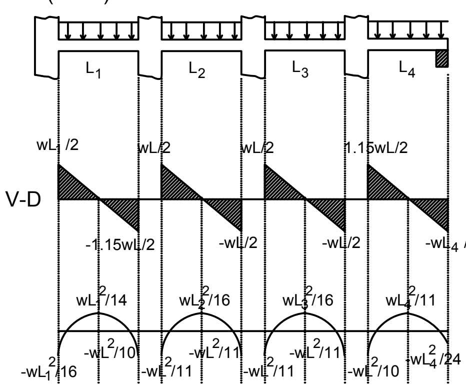
$$Mu2 = \phi\ 0.85\ fc'\ b_w\ z\ \ (d' - z / 2)$$

$$\boxed{Mu = Mu1 + Mu2}$$

ACI/NSCP Coefficients for Continuous Beams and Slabs

Requirements:

1. Two or more spans
2. Loads are uniformly distributed
3. Beams or slabs are prismatic
4. L - S ≤ 20%S
5. $\frac{1.7\ wll}{1.4\ wdl} \leq 3.0$



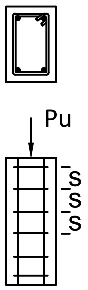
Note:

L = the average span between adjacent spans in shear and negative moment

COLUMNS

Classification of column as to:

A. REINFORCEMENT



1. TIED COLUMNS

Applied Axial Load:

$$Pu = 1.4\ DL + 1.7\ LL$$

Resisting Axial Load:

$$Pu = \phi\, 0.80\ Ag\ [0.85fc'(1-\rho_g) + \rho_g\ fy)]$$

ϕ = 0.70 for tied column

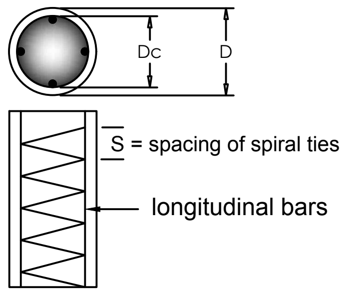
To be safe, Pu act. $\leq$ Pu res.

Note:
ρ_g = gross steel area = As/Ag
As = total steel area
db = bar diameter

ACI Code specs:

1. ρ_g = 0.01 - 0.08
2. Minimum side cover = 40 mm
3. Minimum vertical bars
   - 4 - 16mm dia. - for rec. section
   - 6 - 16mm dia. - for round section
4. Minimum lateral tie bar dia.
   - 10mm dia.- for $\leq$ 32 db main bar
   - 12mm dia.- for > 32 db main bar
5. Spacing of lateral ties (use the smallest)
   - a. 16 vert. bar diameter
   - b. 48 lateral tie bar diameter
   - c. least column dimension
6. Minimum side dimension of column = 200 mm
7. Clear distance between longitudinal bars
   a) 1.5 times bar diameter
   b) 1.5 times max. size of coarse aggregate
8. Minimum covering of ties
   a) 40 mm for interior columns
   b) 50 mm for exterior columns
   c) 1.5 times max. size of coarse aggregate
9. When there are more than four vertical bars, additional ties shall be provided so that every longitudinal bar will be held firmly in position. No bar can be located at a greater distance than 150 mm clear in either side from a laterally supported bar.

2. SPIRAL COLUMNS



Applied Axial Load:

$$Pu = 1.4\ DL + 1.7\ LL$$

Resisting Axial Load:

$$Pu = \phi\, 0.85\ Ag\ [0.85fc'(1-\rho_g) + \rho_g\ fy)]$$

ϕ = 0.75 for spiral column

To be safe, Pu act. $\leq$ Pu res.

ACI Code specs:

1. ρ_g = 0.01 - 0.06
2. minimum diameter = 250 mm
3. min. vertical bars = 6-16 mm
4. minimum spiral = 10 mm
5. clear distance between vertical bars
   a) 1.5 times bar diameter
   b) 1.5 times max. size of coarse aggregate
6. spacing of spirals
   a) not more that 75 mm
   b)not less than 25 mm
   c) not less than 1.5 times coarse aggregate
   d) not more than one-sixth
7. Spacing of spiral tie:

$$s = \frac{4A_{sp}}{\rho_s\, Dc}$$

Minimum spiral steel ratio : ρ_s

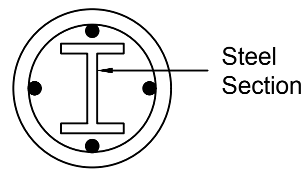
$$\rho_s = 0.45\ [\ (D/Dc)^2 - 1\ ]\ fc'/fy$$

$$\rho_s = \frac{4\ A_{sp}(D_c - d_b)}{s\ D_c^2}$$

where:

Asp = area of the spiral reinforcement
ρ_s = spiral steel ratio
Dc = core diameter (mm)

3. COMPOSITE COLUMN



B. SLENDERNESS

1. Short Column
   Klu/r $\leq$ 34 - 12 M1/M2
2. Slender Column
   Klu/r > 34 - 12 M1/M2

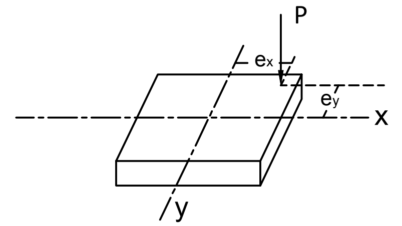
C. SECTION

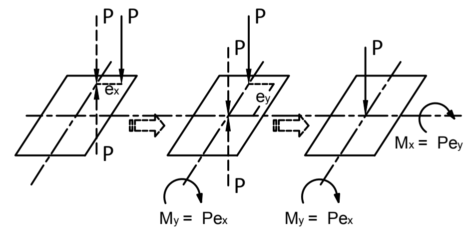
1. Square/rectangular
2. Round/Circular

D. LOAD

1. Axially Loaded
2. Eccentrically loaded
   a. Uniaxial bending
   b. Biaxial bending

ECCENTRICALLY LOADED COLUMN





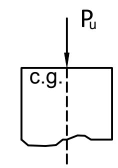
A. Compression plus Uniaxial Bending

e_{min} = 0.10 h for rectangular section
e_{min} = 0.05 D for circular section

where:

h = column dimension parallel to eccentricity (mm)
D = column diameter (mm)

1. e = 0

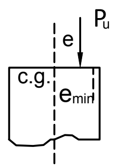


Axially Load:

$$\boxed{Pu = \phi\, 0.80\ Ag\ [0.85fc'(1-\rho_g) + \rho_g\ fy)]}$$

$$\boxed{Pu = \phi\, 0.85\ Ag\ [0.85fc'(1-\rho_g) + \rho_g\ fy)]}$$

2. $e = e_{min}$

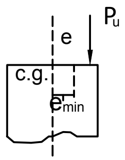


Axially loaded (Neglect the effect of moment)

$$P_u = \phi\, 0.80\, A_g\, [0.85f'c(1-\rho_g) + \rho_g\, f_y)]$$

$$P_u = \phi\, 0.85\, A_g\, [0.85f'c(1-\rho_g) + \rho_g\, f_y)]$$

3. $e_{min} < e < e_b$

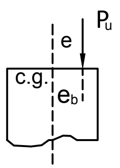


Eccentrically loaded
Consider effect of moment

Failure by crushing of concrete

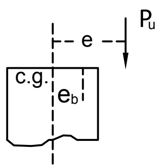
$f_s' = f_y$
$f_s < f_y$

4. $e = e_b$



Eccentrically loaded
Consider effect of moment
$f_s' = f_y$
$f_s = f_y$

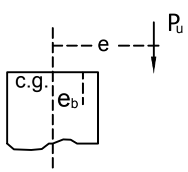
5. $e_b < e$



Eccentrically loaded
Consider effect of moment

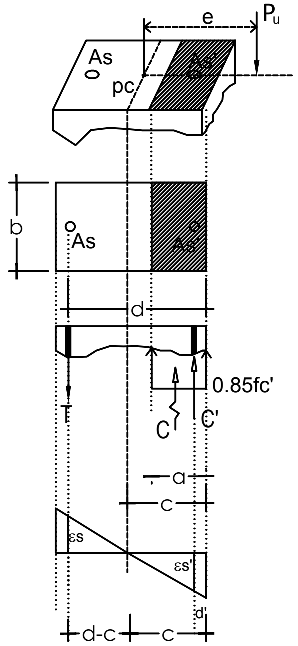
Failure initiated by yielding of tension steel
$f_s = f_y$

6. $e_b <<< e$



Very large moment and negligible axial load
Column behaves like a beam

Compression plus Uniaxial Bending:



Gross Steel Ratio:

$\rho_g = (A_s + A_s') / A_g$
$A_g = bh$
$M_n = P_n (e)$
$M_u = \phi M_n$

Mn = nominal moment
Mu = ultimate moment

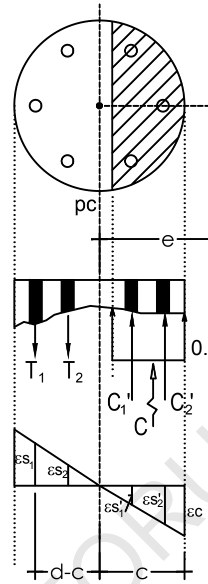
SHORT ECCENTRICALLY LOADED ROUND COLUMNS

Column Interaction Eqtn: (Homogenous Mat'l.)

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

Bresler's Eqtn: (Reinf. Conc.-Composite Mat'l.)

$$\frac{P_n}{P_{nx}} + \frac{P_n}{P_{ny}} + \frac{P_n}{P_{no}} \leq 1.0$$



where:
Pn - nominal load cap.of column at e_x and e_y
Pno - nominal load cap.of column at e = 0
Pnx - nominal load cap.of e_y and e_x
Pny - nominal load cap.of column at e & e = 0

SLENDER COLUMNS

A. Columns braced against sidesway

1. When $Klu/r \leq 34 - 12\, M_1/M_2$, column is short.
2. When $Klu/r > 34 - 12\, M_1/M_2$, column is slender.

B. Unbraced Columns

1. When $Klu/r \leq 22$, column is short.
2. When $Klu/r > 22$, column is slender.

Effective length factor, k

Condition	Value of k
pinned at both ends	1.0
fixed at both ends	0.5
fixed at one end, pinned at the other	0.7
fixed at one end, free at the other	2.0

k 1.0 for braced frames, no sidesway
k > 1.0 for unbraced frames, with sidesway
k = 1.0 for compression members in frames braced against sidesway unless a theoretical analysis shows that a lesser value can be used.

For slender columns (to consider $P\Delta$ - effect or secondary moment)

1. When $Mu(A) \leq Pu(15 + 0.03h)$, use $Mu = Pu\,(15 + 0.03\,h)$
2. When $Mu(A) > Pu(15 + 0.03h)$, use $Mu = Mu(A)$

ACI Moment Magnifier Method

Factored Design Moment:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s}$$

where:

b = bending
s = sidesway
δ = moment magnification factor

Moment Magnifiers

$$\delta_b = \frac{C_m}{1 - \frac{P_u}{\phi P_c}} \geq 1.0$$

$$\delta_s = \frac{C_m}{1 - \frac{\Sigma P_u}{\phi \Sigma P_c}} \geq 1.0$$

$$C_m = 0.60 + 0.40\, M_1/M_2 \geq 0.40$$

(for braced without transversed loads)

$C_m = 1.0$ (for all other cases)

$M_1/M_2 = \dfrac{\text{smaller end moment}}{\text{bigger end moment}}$

where: = + for single curvature
= - for double curvature

$$P_c = \frac{\pi^2 EI}{(K l_u)^2} \qquad EI = \frac{E_c I_g / 2.5}{1 + \beta d}$$

where:

$E_c = 4700\sqrt{f'c}$ (MPa)
$I_g = bh^3/12$
$\beta d = \dfrac{\text{factored axial dead load}}{\text{factored axial total load}}$

Klu/r = slenderness ratio

r = 0.30h for rectangular
= 0.25D for round column

$P_u = P_{dl} + P_{ll}$

FOOTINGS

Types of Footing:

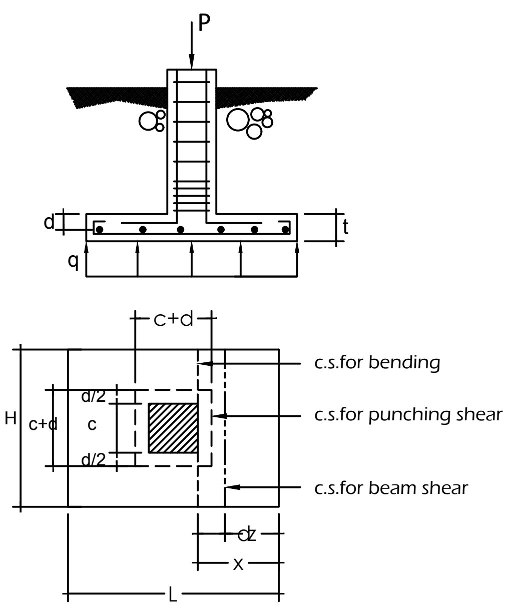
1. Spread Footing (Isolated Footing)
2. Wall Footing
3. Combined Footing
4. Mat and Raft Foundation
5. Footing on Piles

SPREAD FOOTING

Modes of failure:

1. Bearing of soil
2. Bending or Flexure
3. One-way Shear or Beam Shear
4. Two-way Shear of Punching Shear

SPREAD FOOTING (ISOLATED FOOTING)



A. BEARING ON SOIL

$$q = P / Af$$

To be safe, $q \leq q\ all$

where:

q = bearing stress on soil (MPa)
$q\ all$ = allow. bearing stress on soil (MPa)
P = column load
Af = area of soil in contact with bearing stress of soil (mm^2)

B. BENDING OR FLEXURE

Applied Moment:

$$Mu = qu\ (Lx^2) / 2$$

Resisting Moment of steel:

$$Mu = \phi As\ fs\ (d - a/2)$$

Resisting Moment of Concrete:

$$Mu = \phi\ \rho\ fy\ bd^2 [\ 1 - 0.59 \rho fy / fc']$$

To be safe, Mu act $\leq$ Mu resist.

C. ONE -WAY OR BEAM SHEAR

Applied Ultimate Shear:

$$Vu\ act = qu\ (\ H\ z\ )$$

Vu act - critical shear force 'd" from the face of support

Resisting Ultimate Shear Force of concrete

$$\phi Vc = \phi\ 1/6 \sqrt{fc'}\ bd$$

where:

ϕ = capacity or strength reduction factor
= 0.85 for shear and torsion

Vc = nominal shear force capacity of concrete

b,d = beam dimensions (mm)

D. TWO -WAY OR PUNCHING SHEAR

Applied Punching Shear Force:

$$V_p = qu\ [\ L^2 - (c + d)^2\ ]$$

Resisting Shear force of Concrete:

$$V_p = v_{pc}(Ap)$$

Resisting Shear stress of Concrete in Punching:

$$v_{pc} = \phi[\ 1 + 2 / \beta_c\ ]\ 1/6 \sqrt{fc'} \leq \phi 1/3 \sqrt{fc'}$$

To be safe, $V_p \leq v_{pc}$

where:

L = side dimension of footing (m)
c = column dimension (mm)
qu = net upward soil bearing stress or pressure (MPa)

$$qu = \frac{1.4\ P_{DL} + 1.7\ P_{LL}}{Af}$$

$$Ap = 4\ (c + d)\ d$$

CHECK DEVELOPMENT LENGTH

$$Ld_{reqd} = \frac{0.02\ A_b\ fy}{\sqrt{fc'}}$$

DEVELOPMENT LENGTHS

A. STEEL IN TENSION

$$Ld = \frac{0.02\ A_b\ fy}{\sqrt{fc'}}$$

Minimum Ld = 0.06 $d_b fy$ or 300 mm

For top bars:

Ld is multiplied by a factor 1.4

For 35 mmØ and smaller bars

$$Ld = \frac{0.02\ A_b\ fy}{\sqrt{fc'}}$$

Minimum Ld = 300 mm

For 45 mmØ bars

$$Ld = \frac{25\ A_b\ fy}{\sqrt{fc'}}$$

Minimum Ld = 300 mm

For 55 mmØ bars

$$Ld = \frac{40\ fy}{\sqrt{fc'}}$$

Minimum Ld = 300 mm

B. STEEL IN COMPRESSION

$$Ld = \frac{0.02\ A_b\ fy}{\sqrt{fc'}}$$

Minimum Ld = 0.04 $d_b fy$ or 300 mm