$$\boldsymbol{\alpha} = Hb\sqrt{\frac{N}{\sum E_c\, Ig}} \quad \begin{array}{ll} \leq 0.6 & \text{for} \quad n \geq 4 \\ \leq 0.2 + 0.1n & \text{for} \quad n < 4 \end{array}$$

$$\boldsymbol{\alpha}_x = Hb\sqrt{\frac{N}{\sum E_c\, Iy}}$$

$$\boldsymbol{\alpha}_y = Hb\sqrt{\frac{N}{\sum E_c\, Ix}}$$

$$\boxed{H_e = K\, H_o}$$

TABLE 1:Ratio of H_e/H_o for braced column

Condition of upper end	Condition of lower end		
	1	2	3
1	0.75	0.8	0.9
2	0.8	0.85	0.95
3	0.9	0.95	1.0

$\boxed{K \leq 1}$

TABLE 2:Ratio of H_e/H_o for unbraced column

Condition of upper end	Condition of lower end		
	1	2	3
1	1.20	1.30	1.60
2	1.30	1.50	1.80
3	1.60	1.80	…….
4	2.20	…….	…….

$\boxed{K > 1}$

Slenderness Ratio (λ)

For rectangular columns

Calculate **λ** in both directions:

$$\lambda_b = \frac{H_e}{b} \quad , \quad \lambda_t = \frac{H_e}{t}$$

For Circular columns

$$\lambda_D = \frac{H_e}{D}$$

For all shapes

$$\lambda_i = \frac{H_e}{i} \qquad i = \sqrt{\frac{I_g}{A_g}}$$

Column type	Unbraced column		Braced column	
	Short column	Long column	Short column	Long column
Rectangular	$10 \leq \lambda_{b,t}$	$10 < \lambda_{b,t} \leq 23$	$15 \leq \lambda_{b,t}$	$15 < \lambda_{b,t} \leq 30$
Circular	$8 \leq \lambda_D$	$8 < \lambda_D \leq 18$	$12 \leq \lambda_D$	$12 < \lambda_D \leq 25$
General shapes	$35 \leq \lambda_i$	$35 < \lambda_i \leq 70$	$50 \leq \lambda_i$	$50 < \lambda_I \leq 100$

Design moments in long braced column

1- Calculation of additional moments due to buckling

$M_{add} = P\,\delta$

For rectangular columns

- If the column is slender in t direction (y-direction)

$$\delta_t = \frac{\lambda_t^{\,2}\,.\,t}{2000}$$

$$M_{add(x)} = P_u\,.\,\delta_t$$

- If the column is slender in b direction (x-direction)

$$\delta_b = \frac{\lambda_b^{\,2}\,.\,t}{2000}$$

$$M_{add(y)} = P_u\,.\,\delta_b$$

For Circular columns

$$\delta = \frac{\lambda_D^{\,2}\,.\,D}{2000}$$

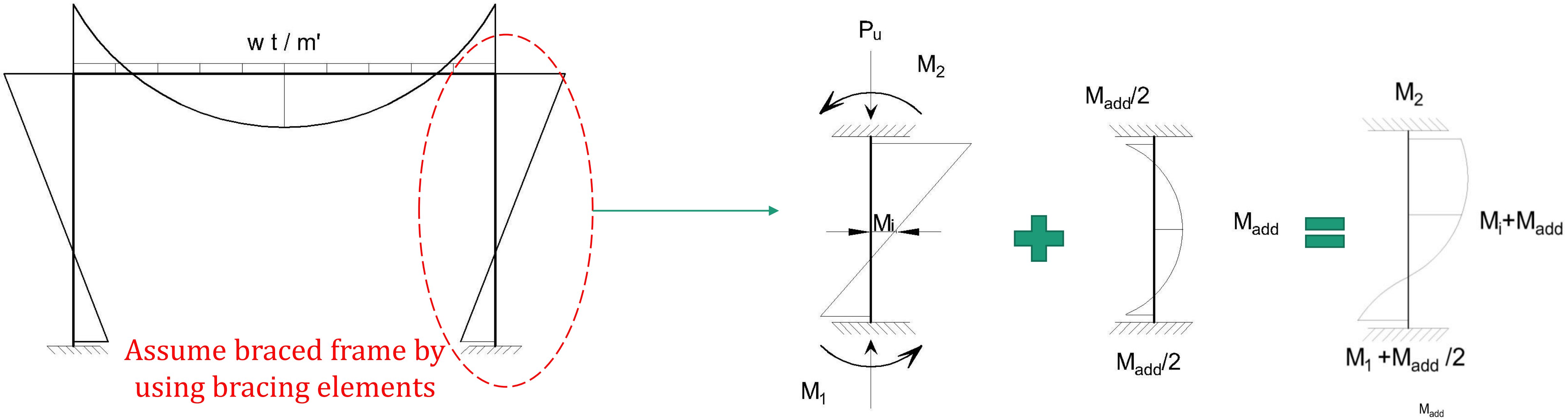
For columns with general shape

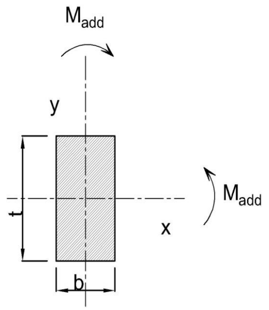
$$\delta = \frac{\lambda_i^{\,2}\,.\,t'}{30000}$$

$\mu_{\,min} = 0.25 + 0.052\lambda_b$ (for rectangular column)

$\mu_{\,min} = 0.25 + 0.015\lambda_i$ (for other column)

Design moments in long braced column





The design moment for braced column is the largest value of:

M_2

$M_1 + M_{add}/2$

$M_i + M_{add}$

$P\ e_{min}$

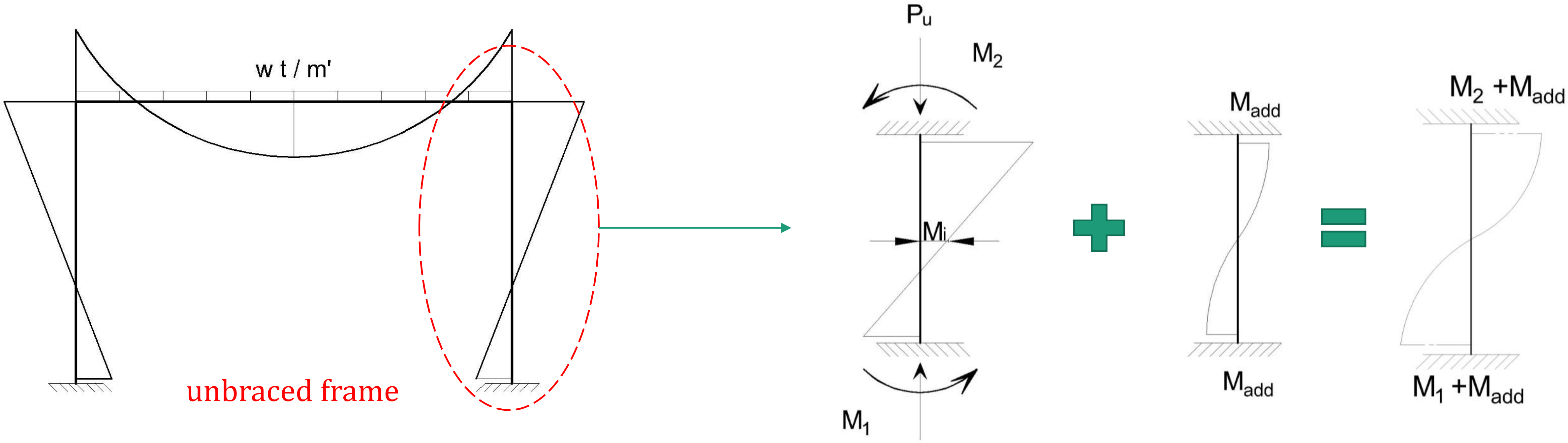
$$\boxed{M_i = 0.4M_1 + 0.6\ M_2\ \geq 0.4M_2}$$

$$\boxed{e_{min} = \text{bigger of}} \begin{cases} \boxed{0.05t \text{ or } 0.05b} \\ \boxed{20\text{ mm}} \end{cases}$$

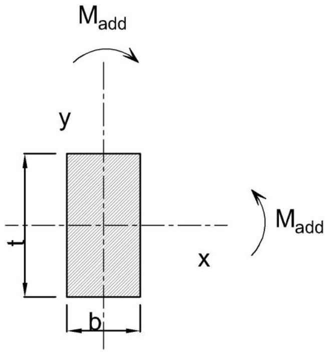
M_{add}

M_2 & M_1 equal zero if the column is hinged at the two ends so the only existing design moment is M_{add}.

Design moments in long unbraced column



The design moment for unbraced column is the largest value of:



$$\delta_{avg} = \frac{\sum \delta}{n}$$

$M_2 + M_{add}$

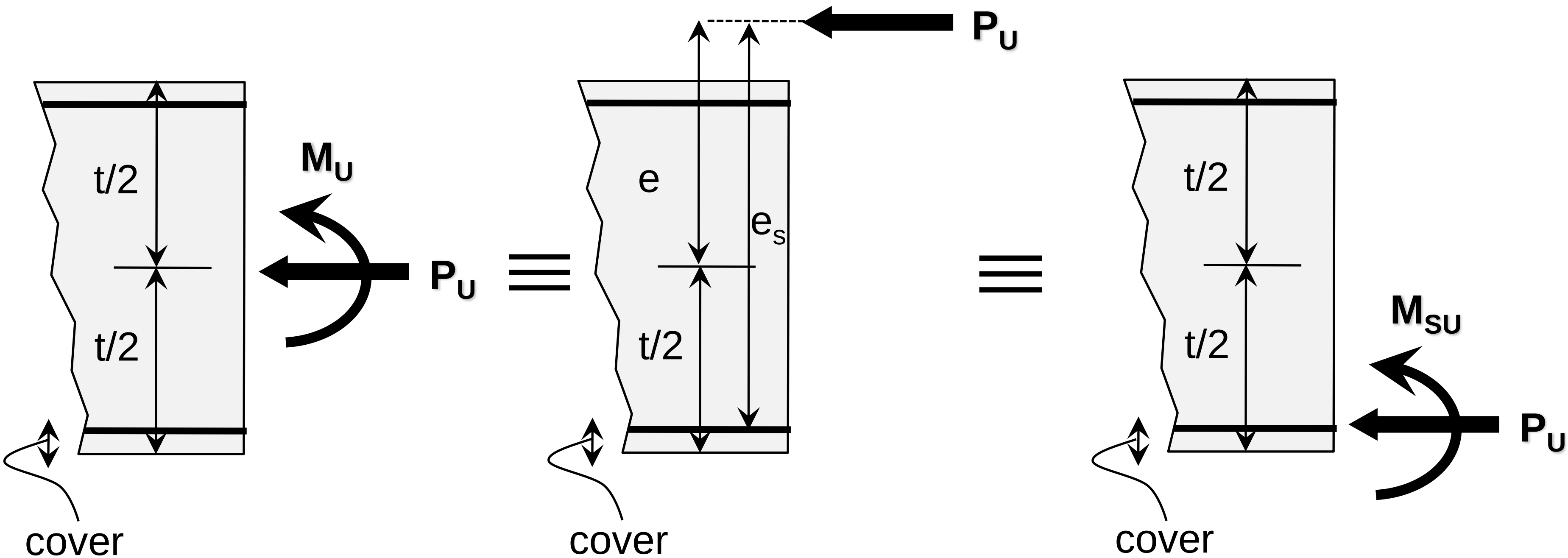
$P\ e_{min}$

$e_{min} =$ bigger of $\begin{cases} \boxed{0.05t \text{ or } 0.05b} \\ \boxed{20 \text{ mm}} \end{cases}$

M_{add}

M_2 & M_1 equal zero if the column is hinged at the two ends so the only existing design moment is M_{add}.

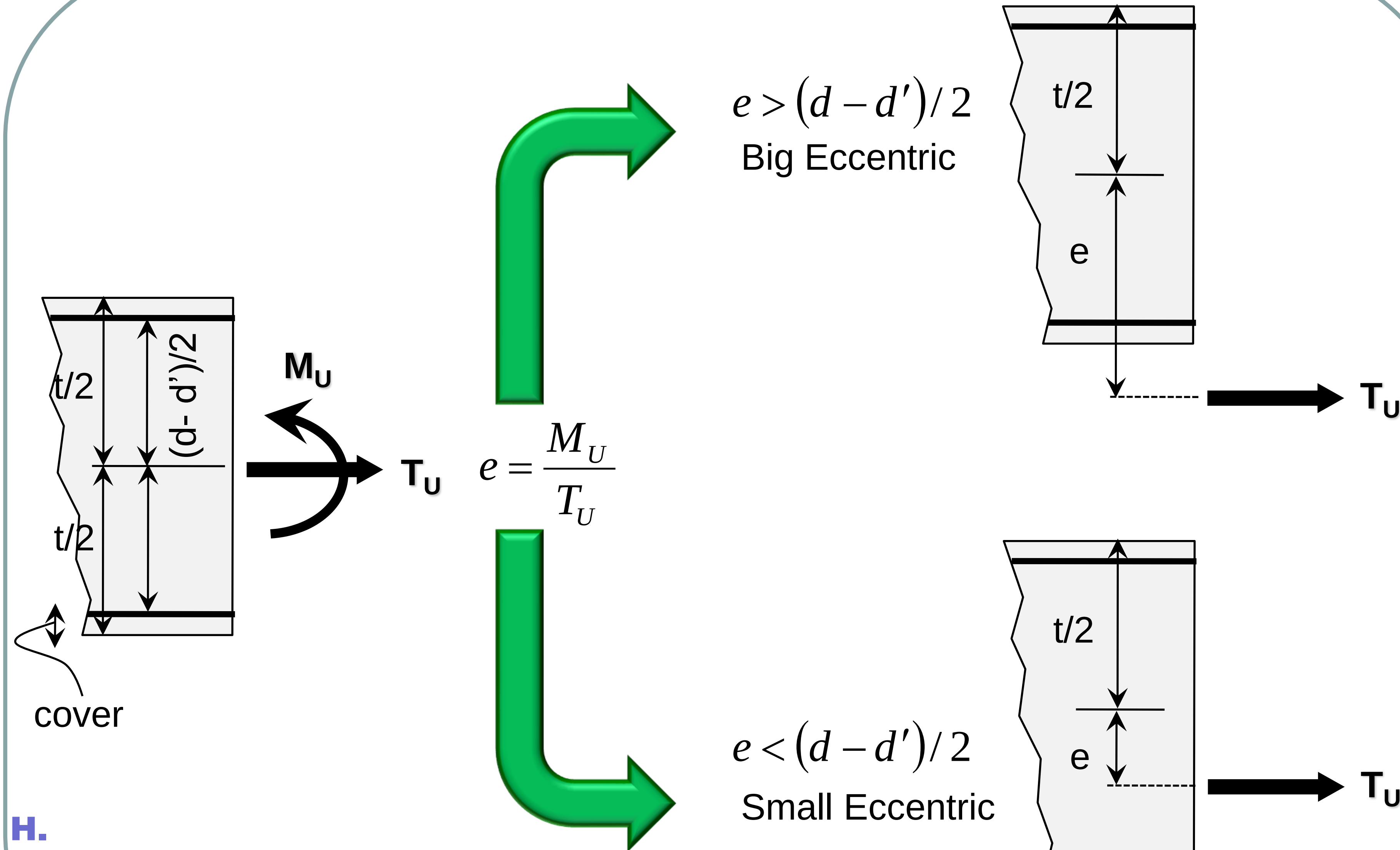
Approximate Method (M_{su} Method)



$$e = \frac{M_U}{P_U}$$

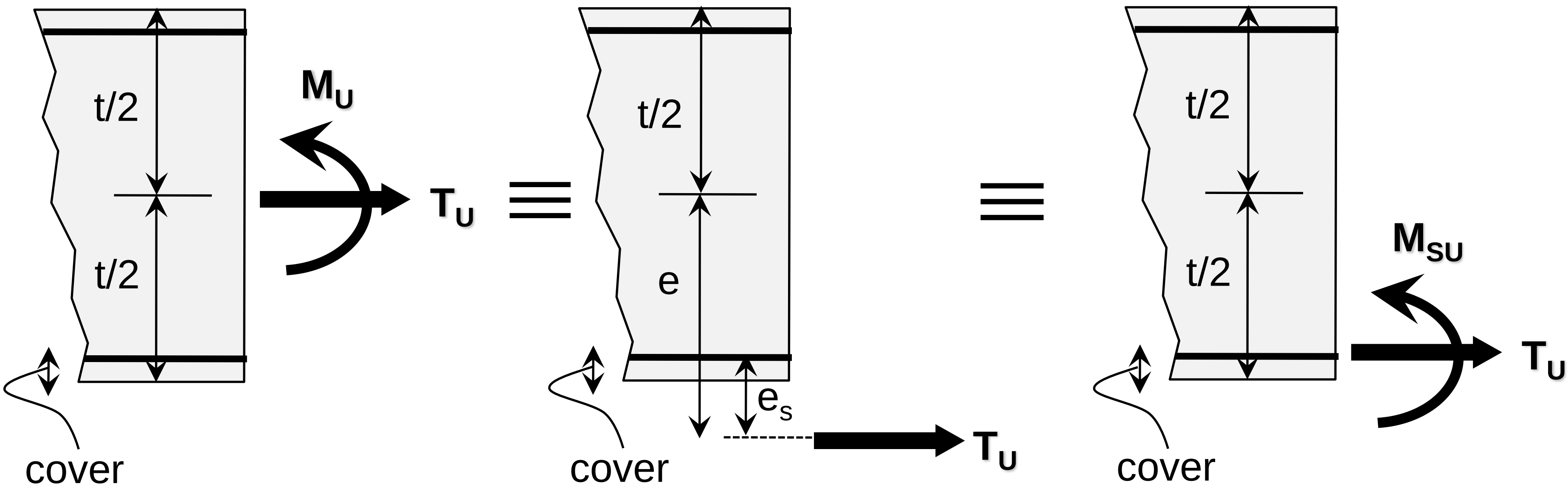
$$e_s = e + \frac{t}{2} - \text{cover}$$

$$M_{SU} = P_U \, e_S$$



t/2
(d- d')/2
t/2
cover
M_U
T_U
$e = \frac{M_U}{T_U}$
$e > (d - d')/2$
Big Eccentric
t/2
e
T_U
$e < (d - d')/2$
Small Eccentric
t/2
e
T_U

Big Eccentric Tension



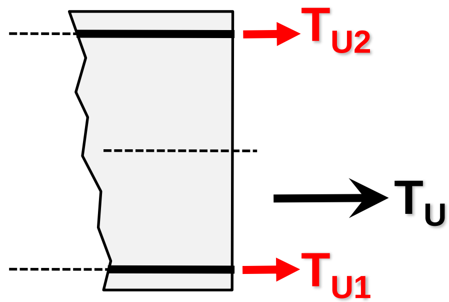
$$e = \frac{M_U}{T_U}$$

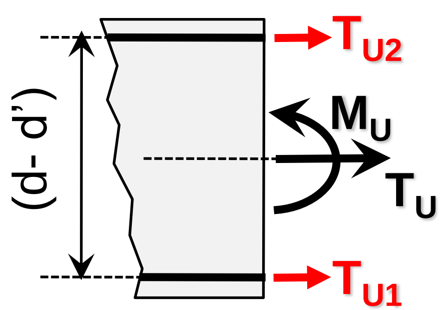
$$M_{SU} = T_U \, e_S$$

$$e_s = e - \frac{t}{2} + \text{cover}$$

Small Eccentric Tension

According to ECP section 4.2.1.4 (a) section capacity is carried only by reinforcing bars in this case



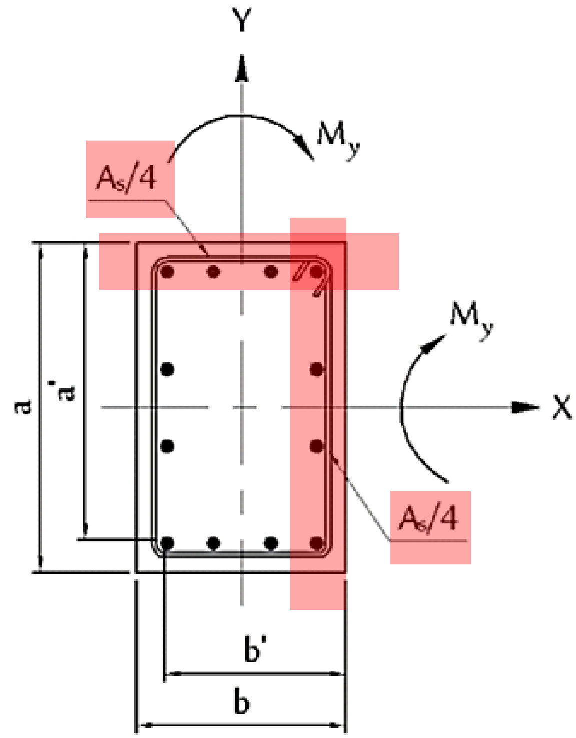


$$T_{U1} = \frac{T_U}{2} + \frac{M_U}{(d-d')} \qquad A_{S1} = \frac{T_{U1}}{\left(f_y/\gamma_S\right)}$$

$$T_{U2} = \frac{T_U}{2} - \frac{M_U}{(d-d')} \qquad A_{S2} = \frac{T_{U2}}{\left(f_y/\gamma_S\right)}$$

في حالة $\left(\frac{M_x}{a'} \le \frac{M_y}{b'}\right)$

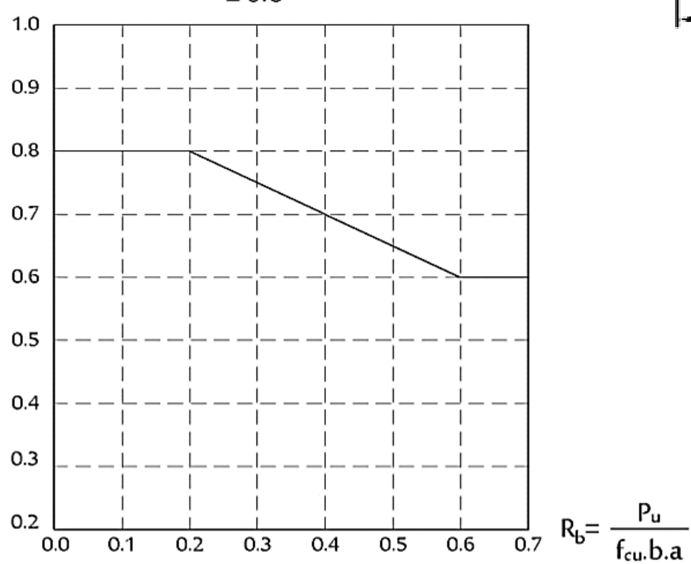
$$M'_y = M_y + \beta\left(\frac{b'}{a'}\right)M_x$$



في حالة $\left(\frac{M_x}{a'} > \frac{M_y}{b'}\right)$

$$M'_x = M_x + \beta\left(\frac{a'}{b'}\right)M_y$$

$$\beta = 0.9 - \frac{R_b}{2} \quad \begin{matrix} \ge 0.6 \\ \le 0.8 \end{matrix}$$



شكل (٦-٢٥-ب) قيمة المعامل β

جدول (٦-١٢-أ) قيم المعامل β

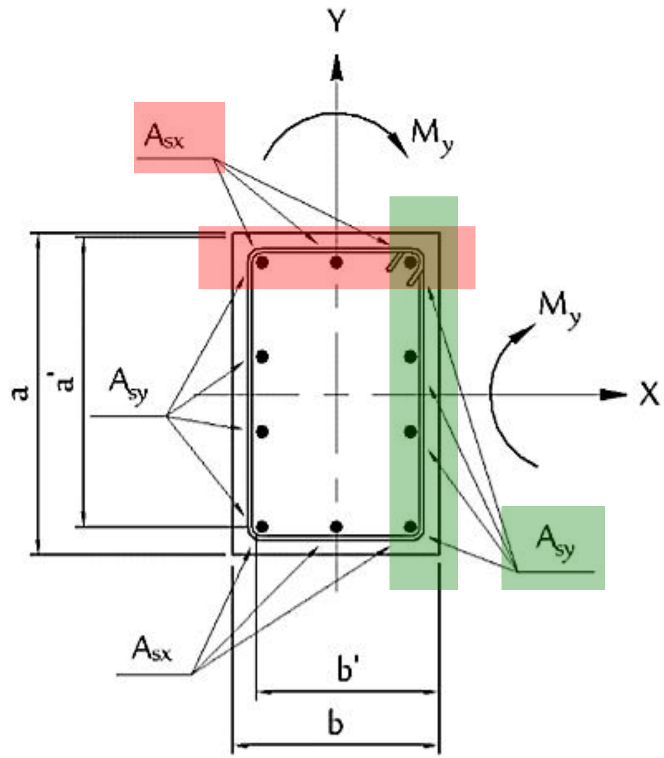
$R_b = \frac{P_u}{f_{cu}.b.a}$	≤ 0.2	0.3	0.4	0.5	≥ 0.6
β	0.80	0.75	0.70	0.65	0.60

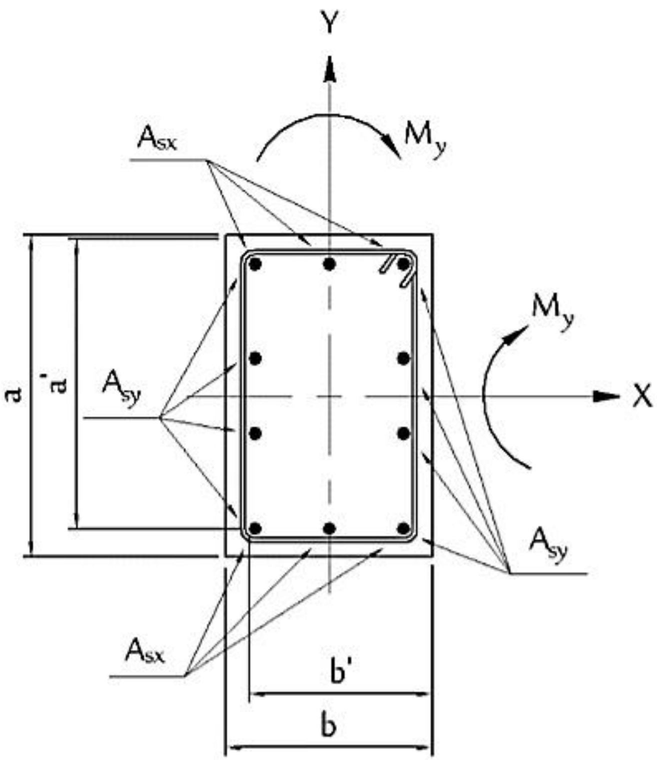
3- ECP Simplified Method (Symmetrical RFT)

في حالة القطاعات المستطيلة المتساوية التسليح على كل وجهين متقابلين في قطاع العمود (شكل ٦-٢٦) وبشرط أن تكون القيمة $\frac{P_u}{f_{cu}.b.a}$ أقل من أو تساوى ٠,٤٠، يمكن تصميم العمود بطريقة مبسطة لمقاومة القوة المحورية P_u وكل من عزمى الانحناء المعدلين التاليين M'_x ، M'_y كل على حدة كالتالي:

$$M'_x = M_x \,.\, \alpha_b \qquad \text{Eq. [6-46a]}$$

$$M'_y = M_y \,.\, \alpha_b \qquad \text{Eq. [6-46b]}$$

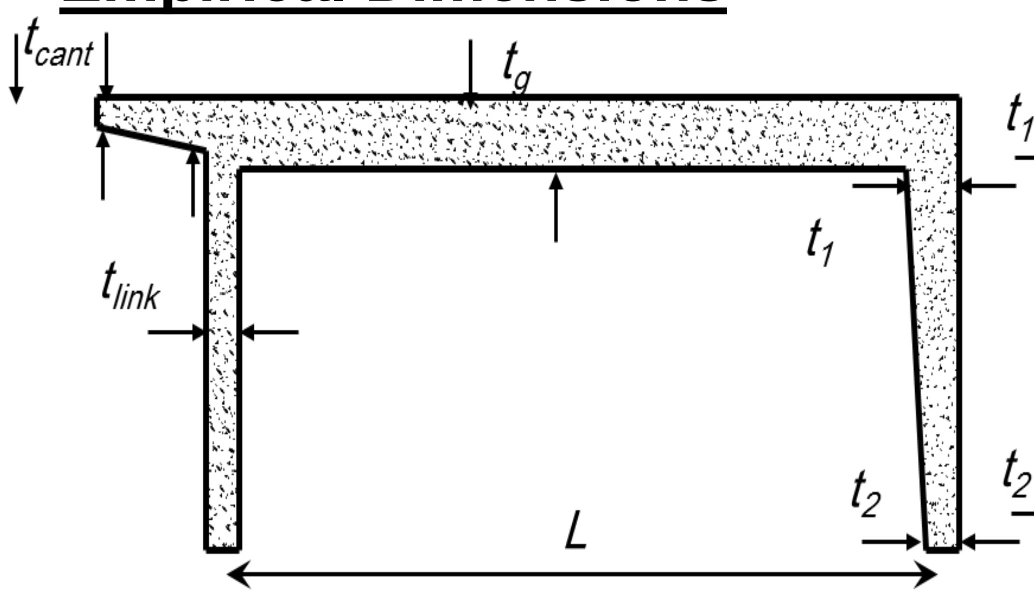


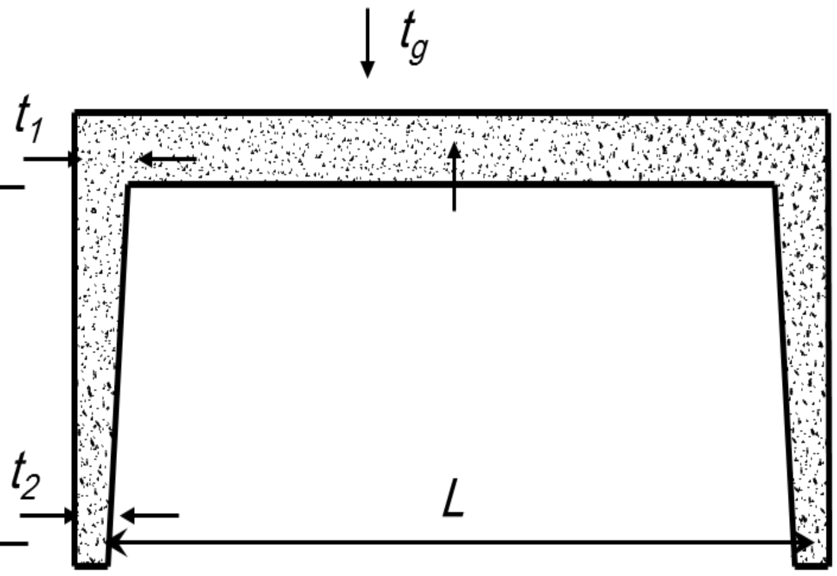


جدول (٦-١٢-ب) قيمة المعامل α_b

Smaller of (Mx/a′)/(My/b′) (My/b′)/Mx/a′) / Rb = Pu/(fcu b.a)	0	0.33	0.5	1
$R_b = 0$	1	0.95	0.9	0.9
$R_b = 0.1$	1	1.20	1.25	1.30
$R_b = 0.2$	1	1.35	1.50	1.75
$R_b = 0.3$	1	1.25	1.35	1.40
$R_b = 0.4$	1	0.95	0.95	0.95

Empirical Dimensions





$$t_g = \frac{L}{12 \sim 16}$$

$$t_1 = (0.8 \sim 1.0)\, t_g$$

$$t_2 = 0.6\, t_g$$

$$b = 30, 35, 40\, cm$$

$$t_{link} = \frac{L}{20}$$

$$t_{cant} = \textit{from design}$$

$$t_3 = 0.67\, t_{cant}$$

Notes

- Main Supporting Element Must be in The Short Directions
- Use Expansion Joints to divide long halls into smaller ones with maximum length = 45.0m

Start frame design by assuming preliminary Dimensions;

Frame girder thickness t_g = Span/(14 – 16) = 24.0/16 = 1.50m

Column top thickness t_1=(0.8 – 1.0) t_g = 1.50m

Column bottom thickness t_2 = (0.6-0.8)t_g = 1.0m

Frame breadth b = 30 – 40 cm → Choose b=40cm

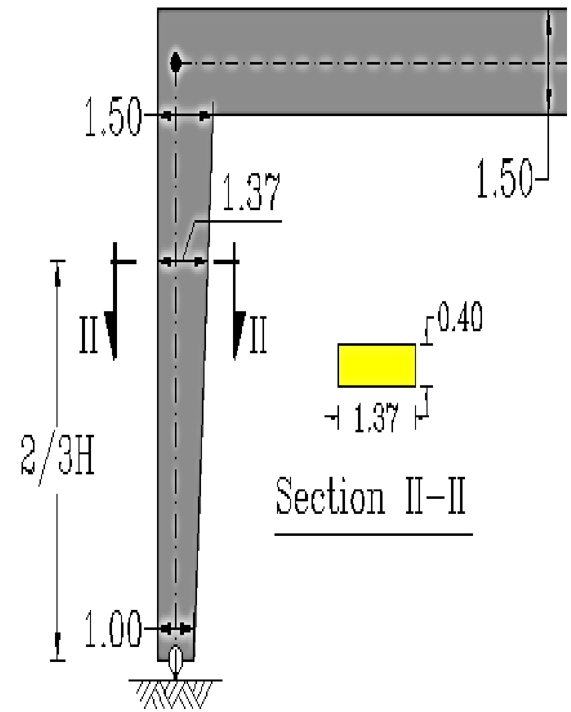
Frame spacing (4.0-6.0m) = 6.0m

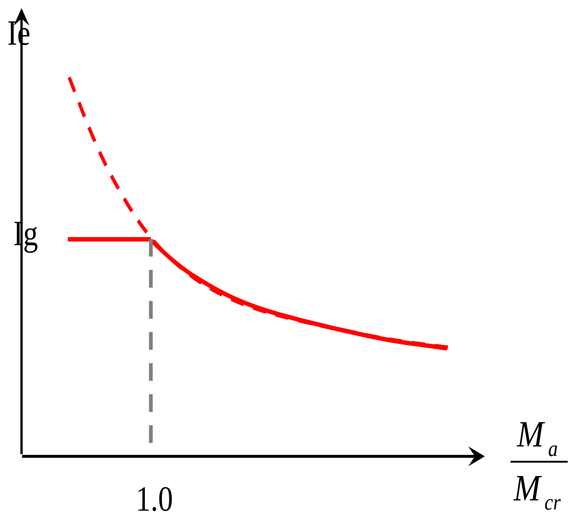
Secondary beam dimensions = 20×60 (span 6.0m)

Edge beam dimensions = 25×60 (span 6.0m)

Wall beam dimensions = 25×60 (span 6.0m)

Slab thickness = 12.0cm





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

$$M_{cr} = \frac{f_{ctr}\, I_g}{y_t}$$

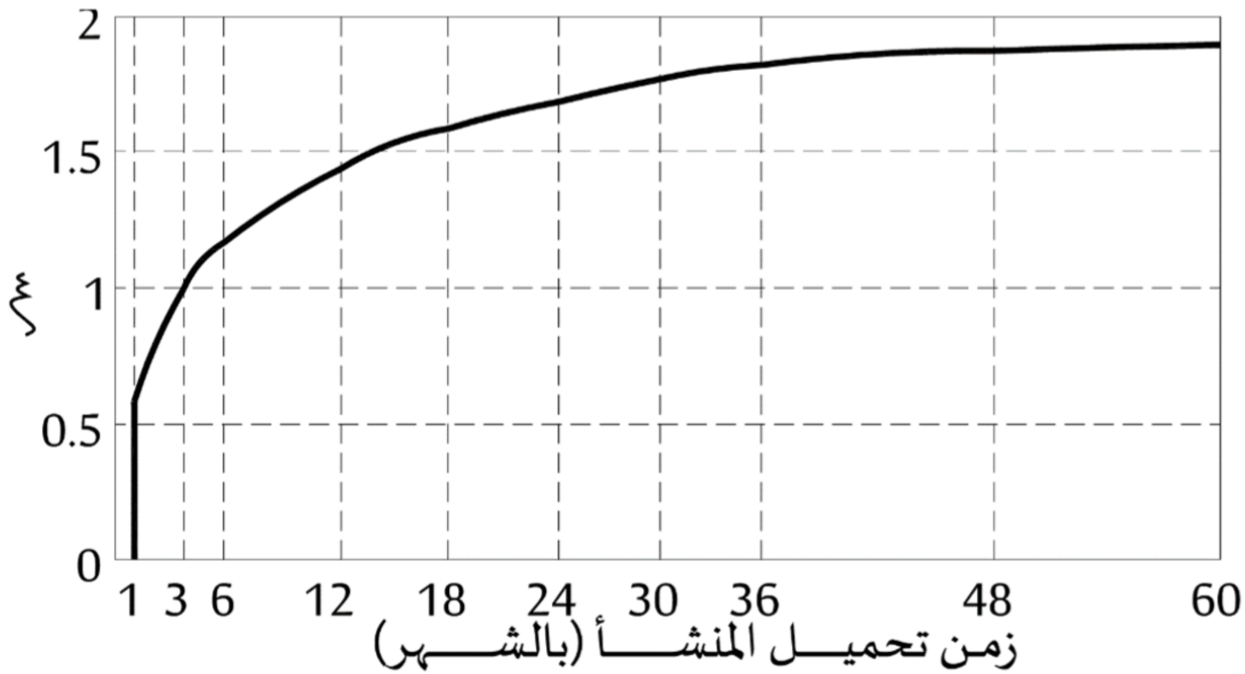
$$f_{ctr} = 0.6\sqrt{f_{cu}} \quad N/mm^2$$

$$\frac{bz^2}{2} - n\,As\left(d-z\right) = 0 \quad \Rightarrow \quad \mathbf{Z}$$

$$I_{cr} = \frac{bz^3}{3} + nAs\left(d-z\right)^2$$

$$n = \frac{E_s}{E} = 15$$

Long term deflection



شكل رقم (٤-٣١) علاقة زمن تحميل المنشأ بمعامل الترخيم للأحمال الدائمة

- بعد خمس سنوات أو أكثر ξ=٢,٠٠
- بعد ١٢ شهر ξ=١,٤٠
- بعد ٦ شهور ξ=١,٢٠
- بعد ٣ شهور ξ=١,٠٠

$$\alpha = \frac{\xi}{1+50\mu'}$$

$$\mu' = \frac{A'_s}{b\,d}$$

Deflection Limits

ECP Clause 4-3-1-2

(1) Total deflection;

$$\left(\Delta_{DL+LL} + \Delta_{long\,term}\right) \leq \begin{cases} \dfrac{L}{250} \Rightarrow \text{for beams and slabs} \\[2ex] \dfrac{L}{450} \Rightarrow \text{for cantilevers} \end{cases}$$

$$\Delta_{long\ term} = \alpha(\Delta_{DL+sustained\ LL})$$

α is calculated at 5 years (ζ=2)

If unsafe, either increase cross section or make cambering

(٩-٥-٣-ز). تحديب فرم بطنيات الكمرات والبلاطات طبقا للبيانات الواردة بمستندات المشروع. وفى حالة عدم توافر هذه البيانات تُحدب الفرم للبحور التي تصل أو تزيد على ثمانية أمتار للكمرات أو سته أمتار للبلاطات بقيمة من (١/٣٠٠) إلى (١/٥٠٠) من طول البحر. وفى حالة الكوابيل التي يزيد بروزها على متر ونصف يكون التحديب في حدود (١/١٥٠) للكمرات و (١/١٠٠) للبلاطات من طول الكابولي.

H. Hadhoud

Deflection Limits

ECP Clause 4-3-1-2

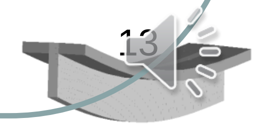
(2) For beams and slabs supporting non-structural elements that are not likely to be damaged by large deflections;

$$\Delta_{LL} \leq \frac{L}{360}$$

$$\Delta_{LL} = \Delta_{DL+LL} - \Delta_{DL}$$

Δ_{DL+LL} is calculated using $I_{e(DL+LL)}$

Δ_{DL} is calculated using $I_{e(DL)}$



Deflection Limits

ECP Clause 4-3-1-2

(3) For beams and slabs supporting non-structural elements that are likely to be damaged by large deflections (like glass facades);

$$(\Delta_{LL-sustained\ LL}) + \Delta_{Long\ term,finishes} \leq \frac{L}{480}$$

For beams and slabs supporting non-structural elements that are not likely to be damaged by large deflections;

$$(\Delta_{LL-sustained\ LL}) + \Delta_{Long\ term,finishes} \leq \frac{L}{240}$$

$\Delta_{Long\ term}$ is the total additional deflection occur after construction of nonstructural elements

$$\Delta_{Long\ term,finishes} = (\alpha_1 - \alpha_2)(\Delta_{DL+sustained\ LL})$$

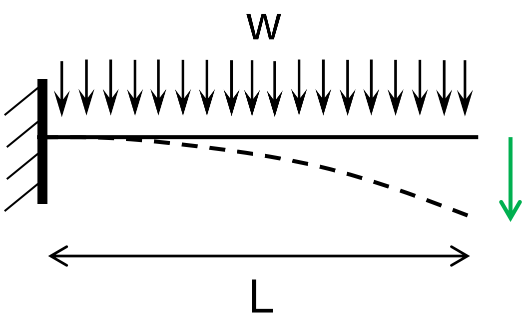
(α_1) is calculated at 5 years (ζ=2)

(α_2) is calculated at the time of adding the nonstructural elements

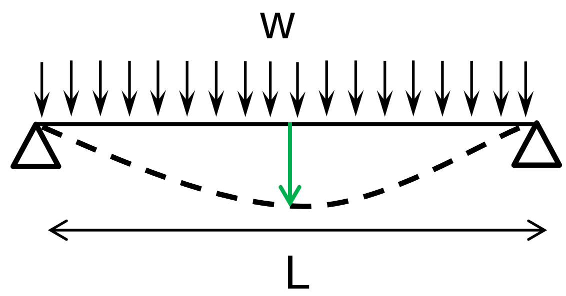
It may be assumed at 12 months (ζ=1.4)

Deflection Calculation

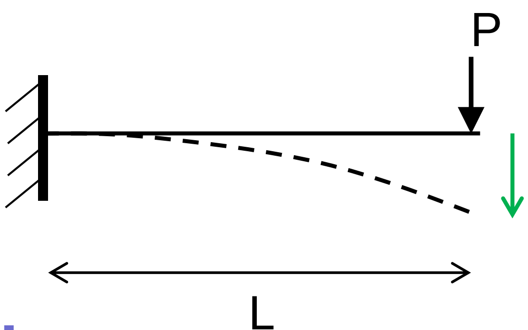
ECP Clause 4-3-1-1



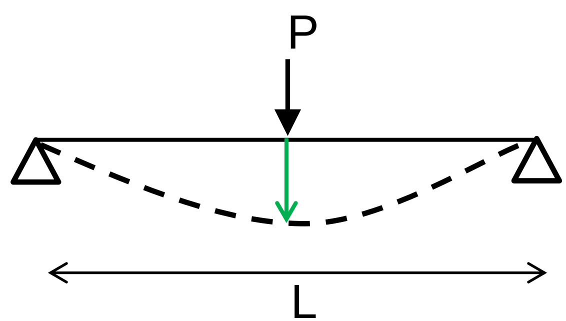
$$\Delta = \frac{w L^4}{8 E I_e}$$



$$\Delta = \frac{5 w L^4}{384 E I_e}$$

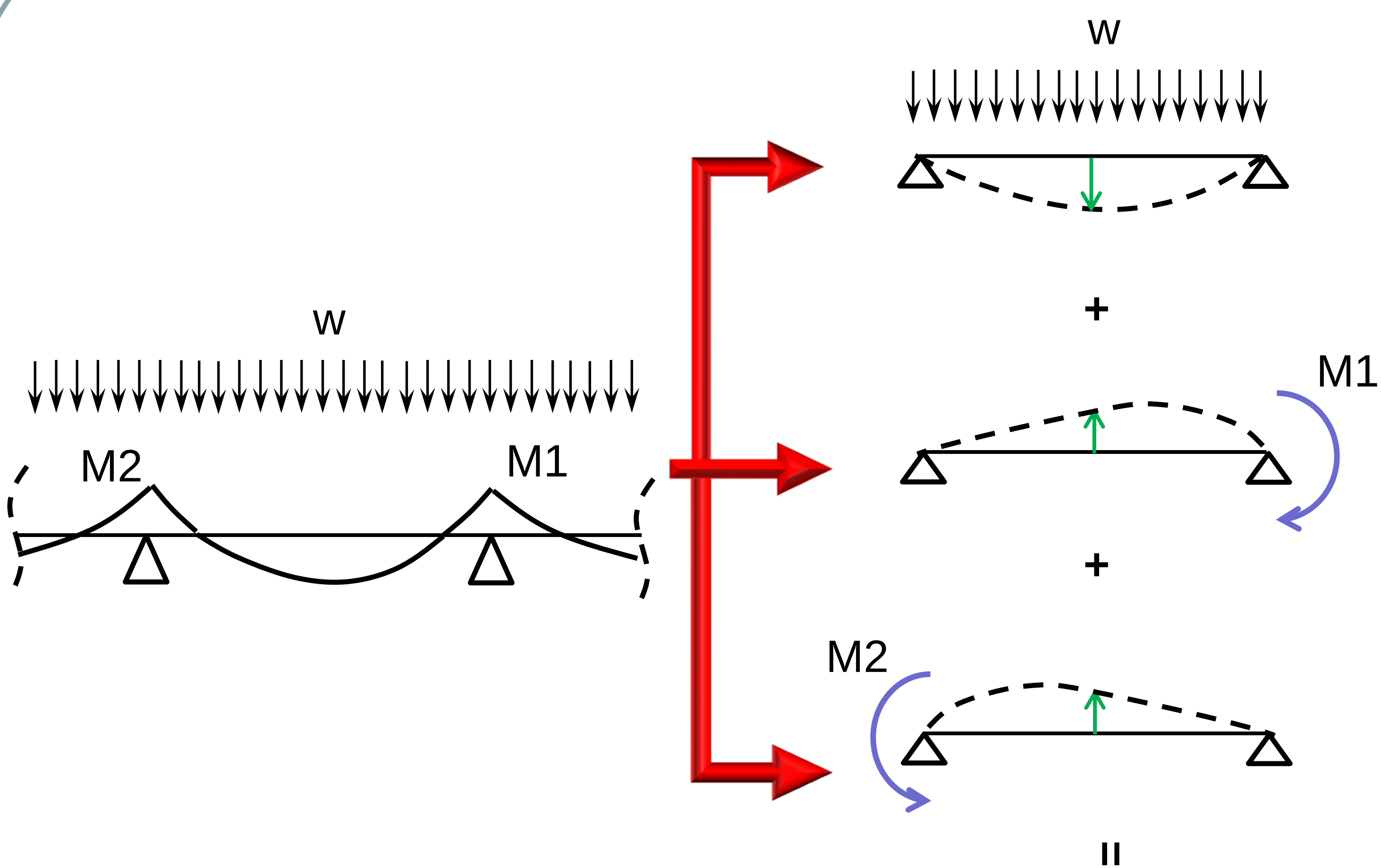


$$\Delta = \frac{P L^3}{3 E I_e}$$



$$\Delta = \frac{P L^3}{48 E I_e}$$

$$E_c = 4400 \sqrt{f_{cu}} \;\; N/mm^2$$



$$\Delta_1 = \frac{5wL^4}{384EI_e}$$

$$\Delta_2 = \frac{M_1 L^2}{16EI_e}$$

$$\Delta_3 = \frac{M_2 L^2}{16EI_e}$$

$$\Delta = \Delta_1 - \Delta_2 - \Delta_3$$

$$E_c = 4400\sqrt{f_{cu}} \; N/mm^2$$

1- Plain concrete footing

$$P_w \cong \frac{P_u}{1.5}$$

Effect of soil weight above footing

$$A_1\, B_1 \cong \frac{1.1\, P_w}{q_{all)gross}} \quad \text{OR} \quad A_1\, B_1 \cong \frac{P_w}{q_{all)net}} \qquad (1)$$

$$A_1 - B_1 = a - b \qquad (2)$$

From (1) and (2) get A_1 and B_1

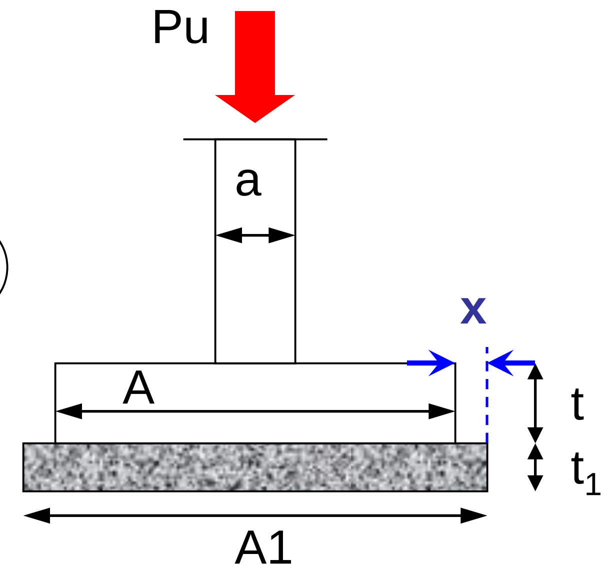
t_1= 300 ~ 500 mm

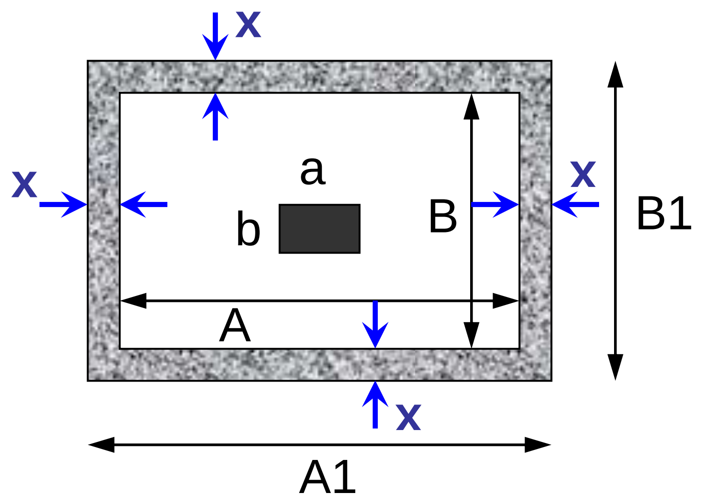
x =(0.8 ~ 1) t_1

$$q_n = \frac{P_w}{A_1\, B_1}$$

$$M = q_n \times \frac{x^2}{2} \qquad f_t = \frac{6\,M}{1000\, t_1^2}$$

$$check \;\; f_t \leq \frac{f_{ctr}}{\eta} = \frac{0.6\sqrt{f_{cu}}}{1.6} \;\; N/mm^2$$



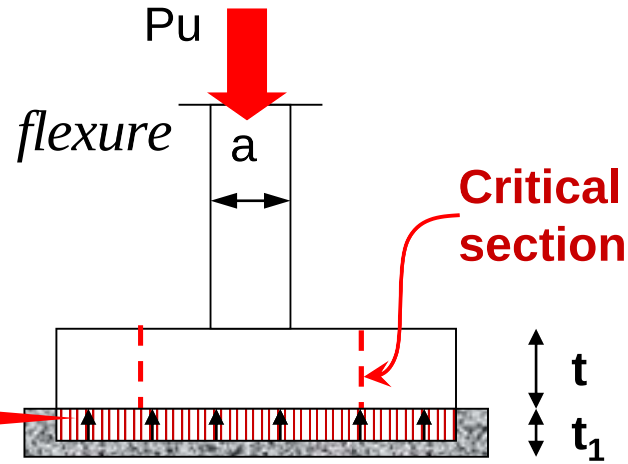


2- Reinforced concrete footing

$$A = A_1 - 2x \qquad B = B_1 - 2x$$

$$Assume\ t \ge 500\,mm \Rightarrow check\ punching,\ shear,\ flexure$$

$$d = t - 50\,mm$$



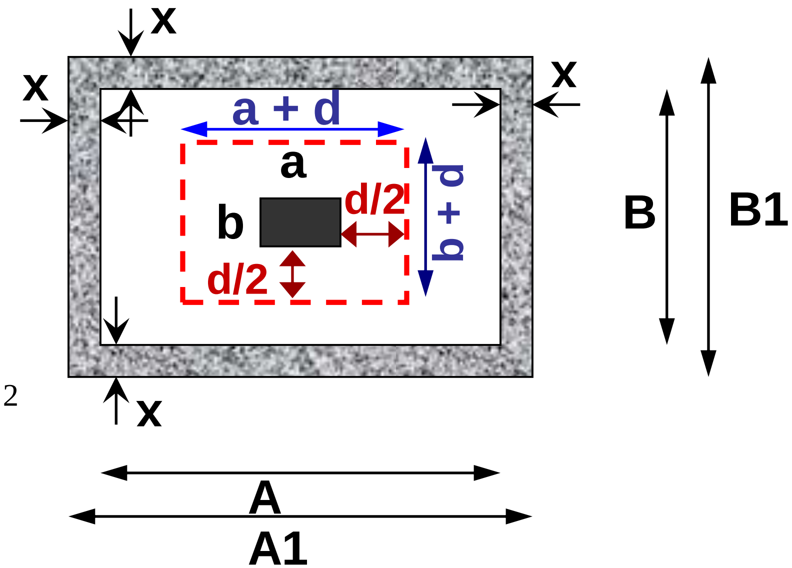
Punching (Two-way shear)

$$p_n = \frac{P_u}{AB}$$

$$punching\,area = b_o d = \left[2\left(a + d + b + d\right)\right]d$$

$$punching\ load = Q_{up} = P_u - p_n\left(a + d\right)\left(b + d\right)$$

$$q_{up} = \frac{Q_{up}}{A_p} = \frac{Q_{up}}{b_o d} \le q_{cup}$$



a is the smaller

$$q_{cup} = least\ of \begin{cases} 0.8\left(\alpha d / b_o + 0.2\right)\sqrt{f_{cu} / \gamma_c} & N/mm^2 \\ 0.316\left(0.5 + a / b\right)\sqrt{f_{cu} / \gamma_c} & N/mm^2 \\ 0.316\sqrt{f_{cu} / \gamma_c} & N/mm^2 \end{cases}$$

Eq. 4-32 & 4-33

$$\alpha \begin{cases} 4\ \text{(interior column)} \\ 3\ \text{(exterior column)} \\ 2\ \text{(corner column)} \end{cases}$$

4

Direct shear (one-way shear)

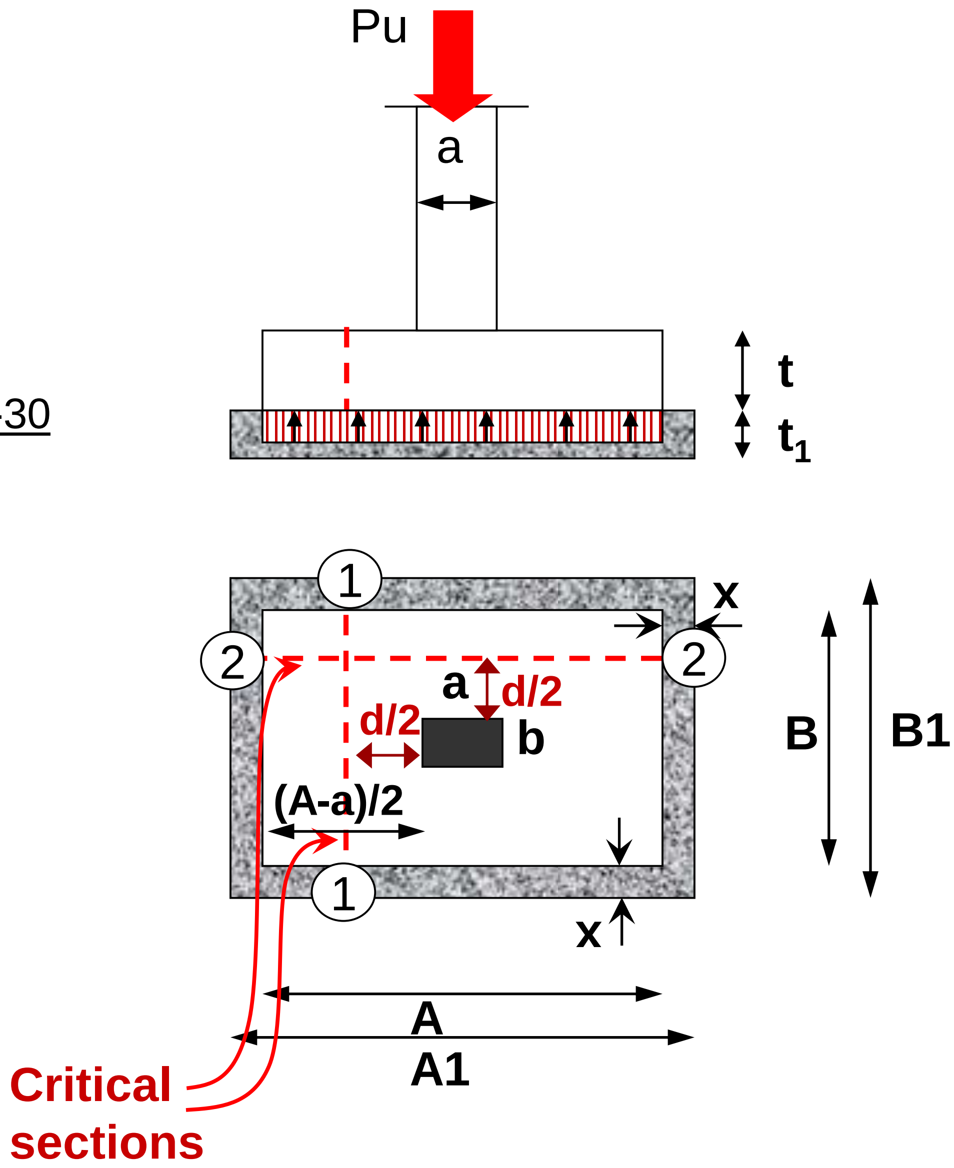
$$Q_{u1} = p_n\left(\frac{A-a}{2} - \frac{d}{2}\right)B$$

$$q_{u1} = \frac{Q_{u1}}{Bd} \leq 0.16\sqrt{\frac{f_{cu}}{\gamma_c}}\ N/mm^2$$ Eq. 4-30

$$Q_{u2} = p_n\left(\frac{B-b}{2} - \frac{d}{2}\right)A$$

$$q_{u2} = \frac{Q_{u2}}{Ad} \leq 0.16\sqrt{\frac{f_{cu}}{\gamma_c}}\ N/mm^2$$

If not increase d



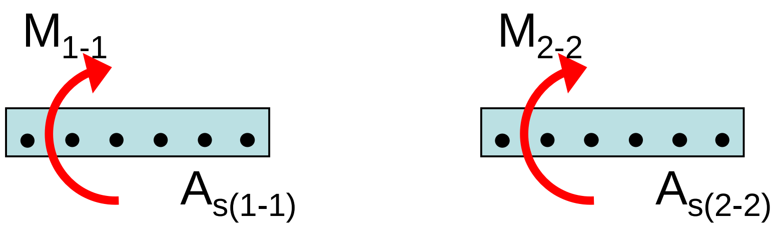
Flexure

$$M_{u)1-1} = \frac{p_n\left(\frac{A-a}{2}\right)^2}{2} = \frac{p_n(A-a)^2}{8}$$

$$M_{u)2-2} = \frac{p_n\left(\frac{B-b}{2}\right)^2}{2} = \frac{p_n(B-b)^2}{8}$$

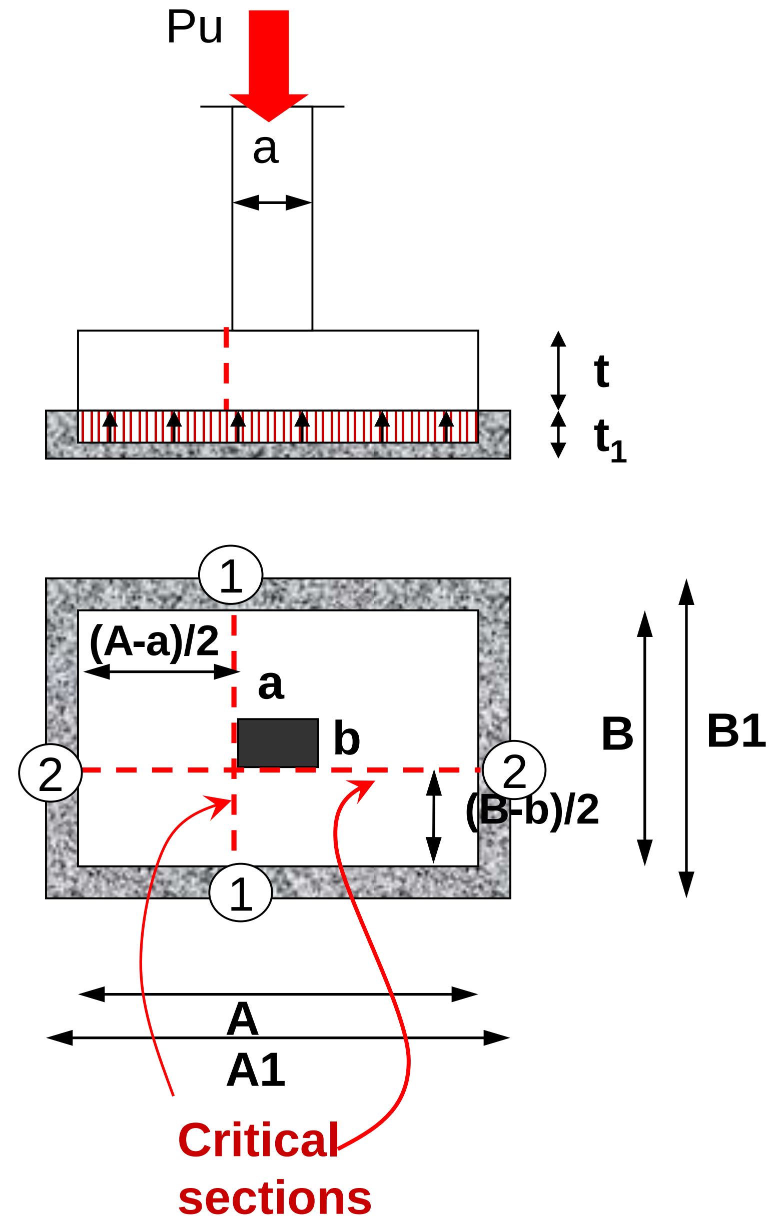
$$d_{1-1} = t - 50\ mm\ \&b = 1000mm$$

$$d_{2-2} = t - 70\ mm\ \&b = 1000mm$$



$$A_{S)\min} = \frac{0.15}{100}(1000\ d)\,mm^2$$

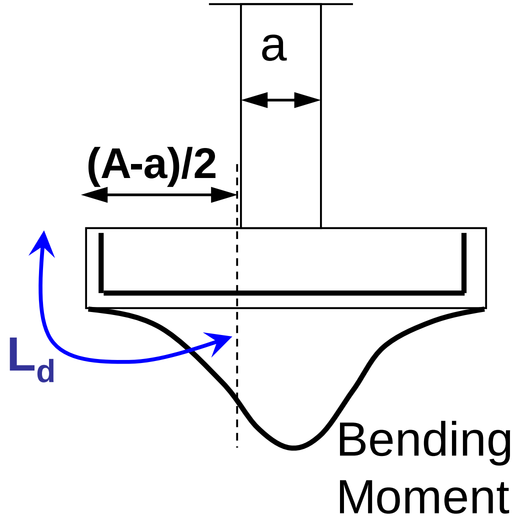
$$\Phi_{\min} = 12mm\ \&S_{\max} = 200\ mm$$



Bar development length requirements

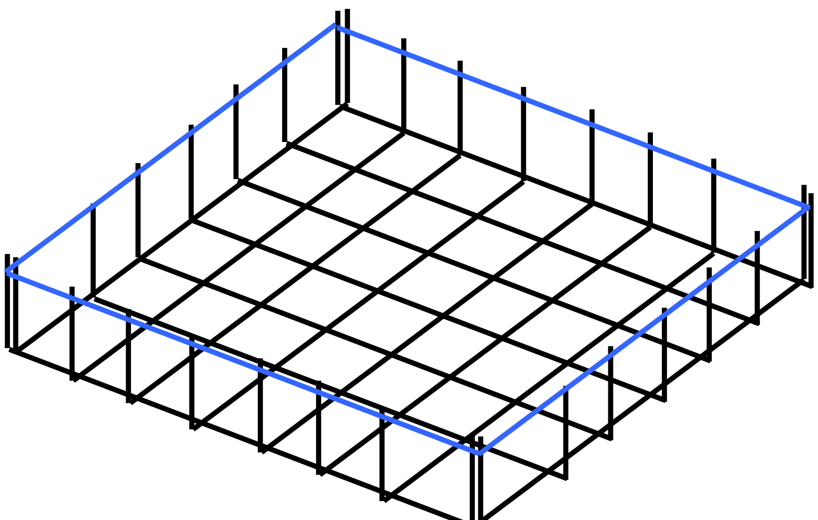
$$if\ \frac{A-a}{2} < 60\Phi$$ →  Use U-shaped bars

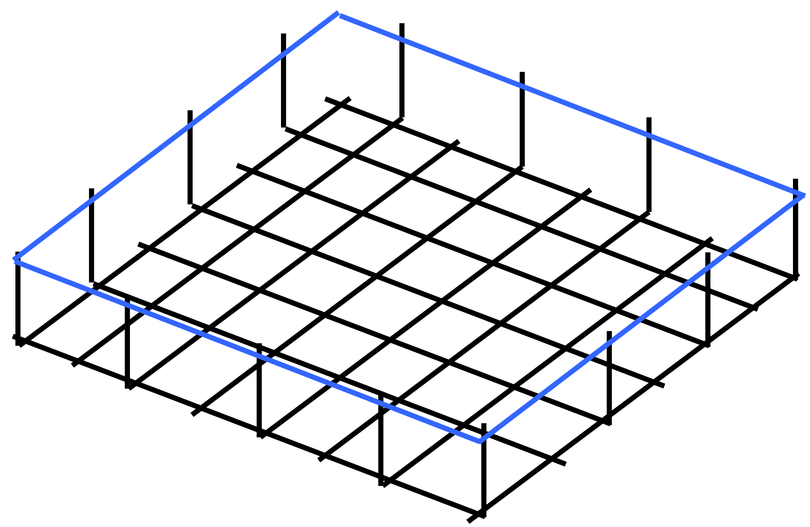
Else →  Use staggered L-shaped bars OR U-shaped bars



$$if\ \frac{B-b}{2} < 60\Phi$$ →  Use U-shaped bars

Else →  Use staggered L-shaped bars OR U-shaped bars





Design of Combined Footings

Prepared by
Prof. Dr. Hamed Hadhoud

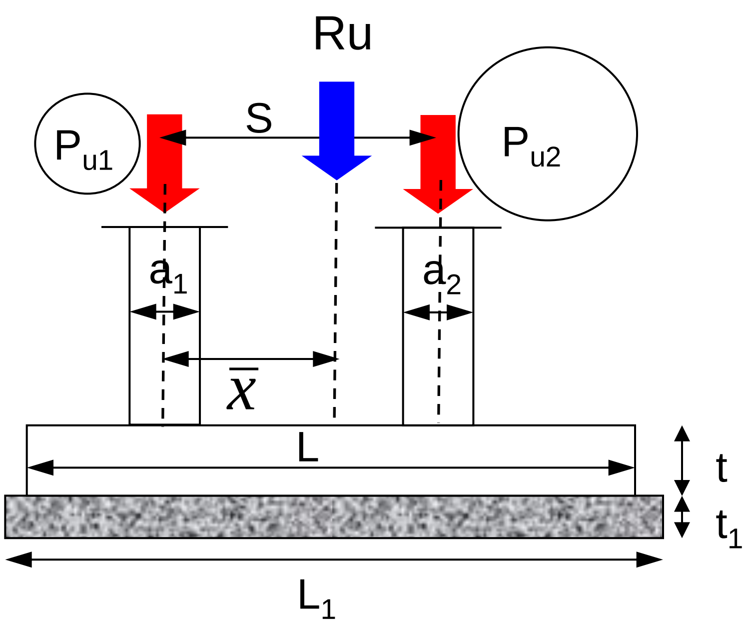
1- Plain concrete footing

$$P_{u2} > P_{u1}$$

$$R_u = P_{u1} + P_{u2}$$

$$\bar{x} = \frac{P_{u2}\,S}{R_u}$$

$$\frac{L_1}{2} \cong \bar{x} + \frac{a_1}{2} + 1m \Rightarrow get\ L_1$$



$$R_w \cong \frac{R_u}{1.5}$$

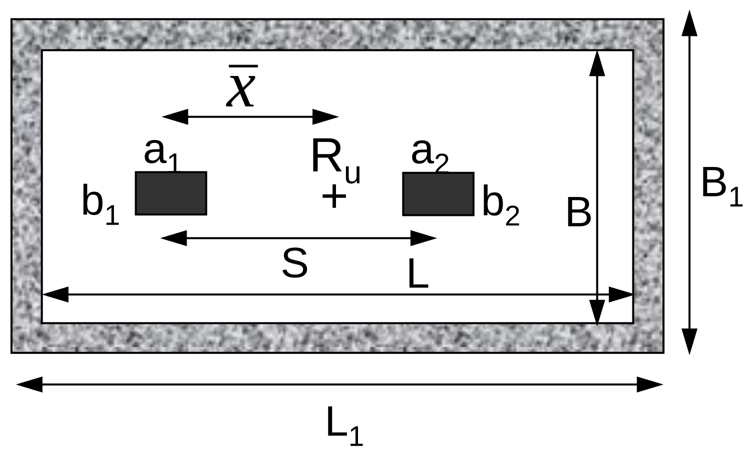
Effect of soil weight above footing

$$L_1 B_1 \cong \frac{1.1 R_w}{q_{all)gross}}$$

OR

➔ Get B_1

$$L_1 B_1 \cong \frac{R_w}{q_{all)net}}$$



1- Plain concrete footing

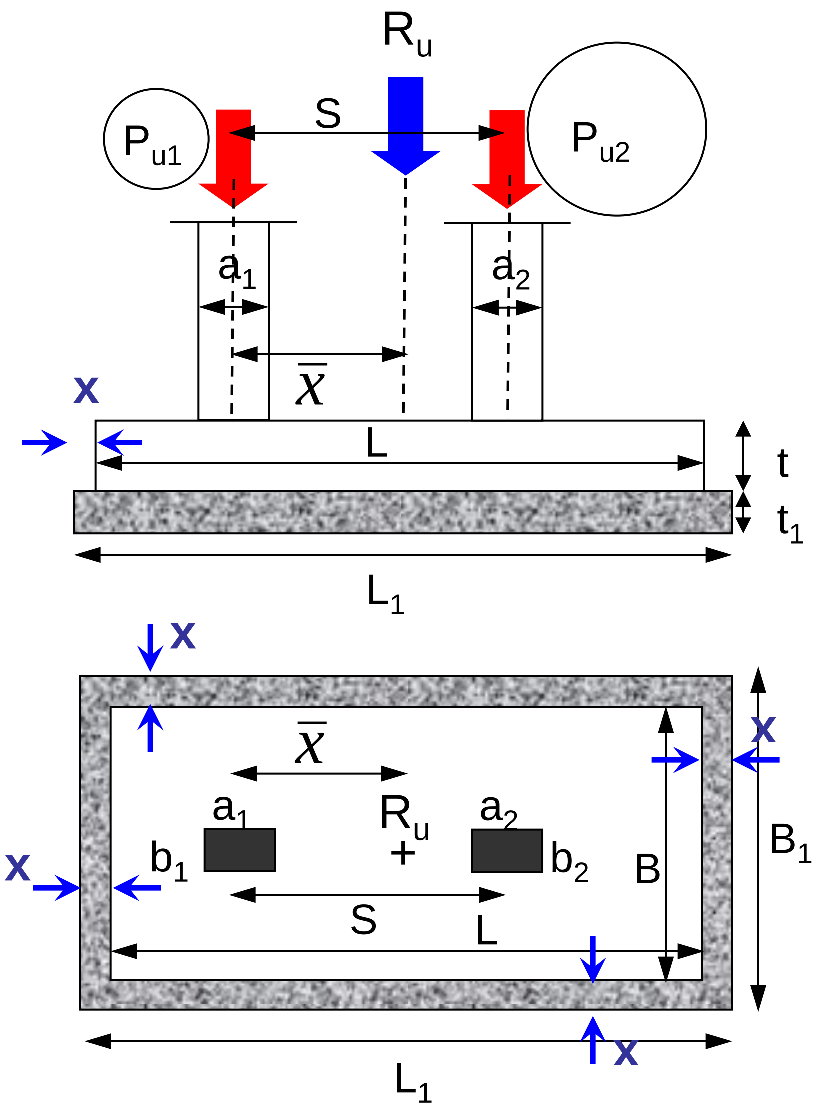
t_1= 300 ~ 500 mm

x =(0.8 ~ 1) t_1

$$q_n = \frac{R_w}{L_1\, B_1}$$

$$M = q_n \times \frac{x^2}{2} \qquad f_t = \frac{6\,M}{1000\, t_1^2}$$

$$check\ \ f_t \le \frac{f_{ctr}}{\eta} = \frac{0.6\sqrt{f_{cu}}}{1.6}\ \ N/mm^2$$



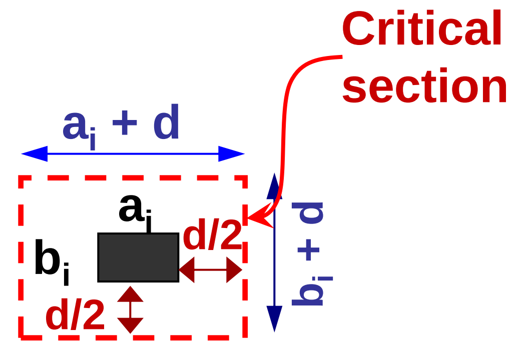
2- Reinforced concrete footing

$$L = L_1 - 2x \qquad B = B_1 - 2x$$

$$Assume\ t \geq 700\ mm \Rightarrow check\ punching,\ shear,\ flexure$$

$$d = t - 70\ mm$$

Punching (Two-way shear)



i denotes for 1 and 2

$$p_n = \frac{R_u}{LB}$$

$$punching\ area = b_o d = \left[2\left(a_i + d + b_i + d\right)\right] d$$

$$punching\ load = Q_{up} = P_{ui} - p_n\left(a_i + d\right)\left(b_i + d\right)$$

$$q_{up} = \frac{Q_{up}}{A_p} = \frac{Q_{up}}{b_o d} \leq q_{cup}$$

a is the smaller

$$q_{cup} = least\ of \begin{cases} 0.8\left(\alpha d / b_o + 0.2\right)\sqrt{f_{cu}/\gamma_c}\ \ N/mm^2 \\ 0.316\left(0.5 + a/b\right)\sqrt{f_{cu}/\gamma_c}\ \ N/mm^2 \\ 0.316\sqrt{f_{cu}/\gamma_c}\ \ N/mm^2 \end{cases}$$

Eq. 4-32 & 4-33

$$\alpha \begin{cases} 4\ \text{(interior column)} \\ 3\ \text{(exterior column)} \\ 2\ \text{(corner column)} \end{cases}$$

Direct shear (one-way shear)

$$w_n = \frac{R_u}{L}$$

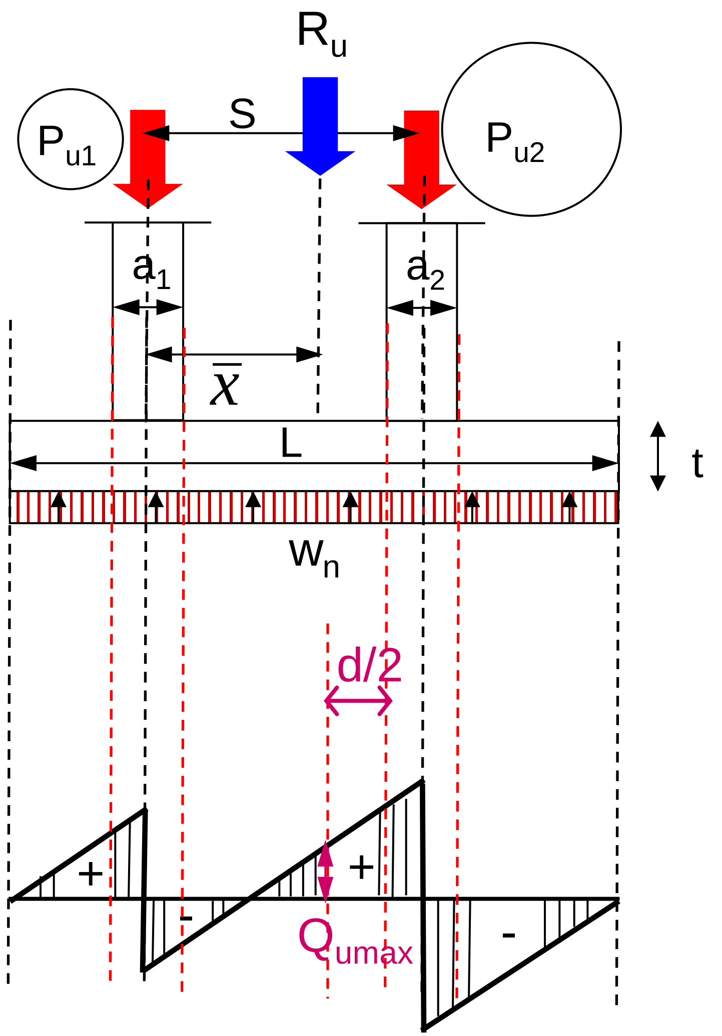
Draw shearing force diagram

Decide critical section for shear

(at distance d/2 from column face)

$$q_u = \frac{Q_{u\max}}{Bd} \le 0.16\sqrt{\frac{f_{cu}}{\gamma_c}}\ N/mm^2$$ <u>Eq. 4-30</u>

If not increase d



Flexure

Draw bending moment diagram

Decide critical section for negative moment
(at zero shear) ➔ *get* M_{umax-}

Calculate maximum positive moment
(at column face) ➔ M_{umax+}

$$d = t - 70\ mm$$

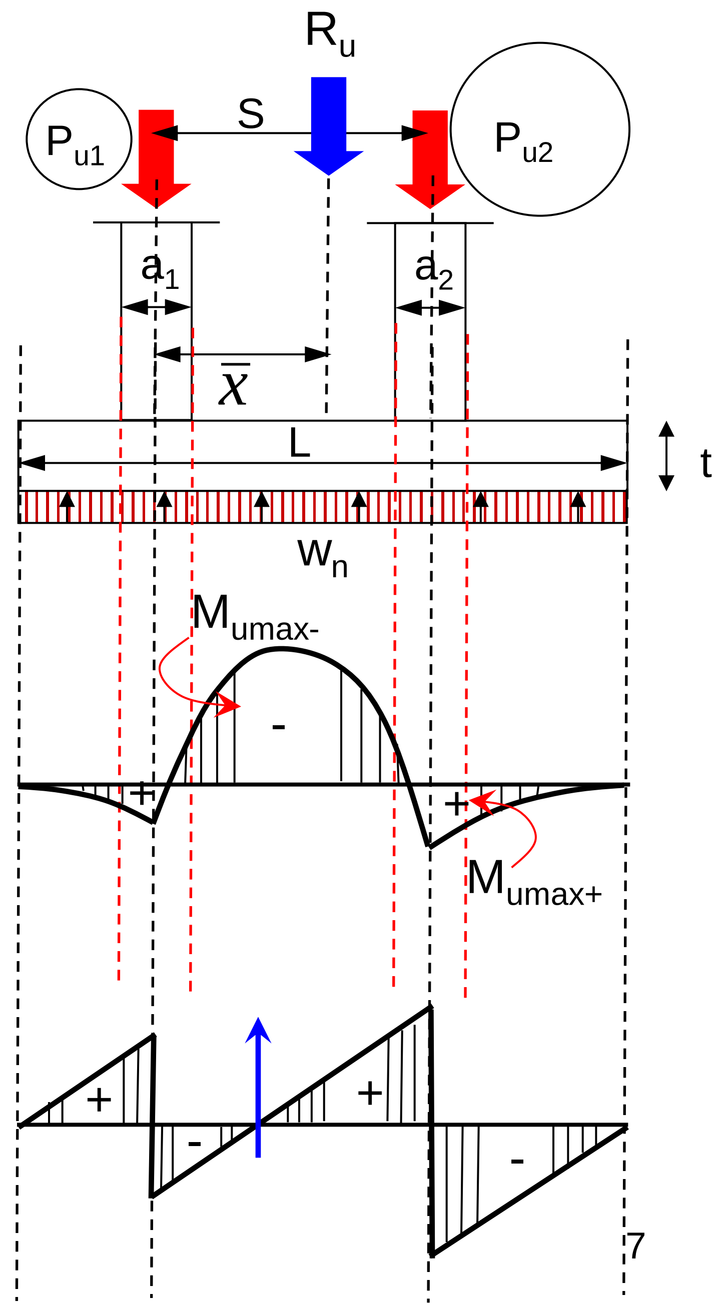
$$R = \frac{M_{u\max+}}{f_{cu}\,B\,d^2} \qquad R = \frac{M_{u\max-}}{f_{cu}\,B\,d^2}$$

$$A_{s+ve} = wBd\,\frac{f_{cu}}{f_y} \qquad A_{s-ve} = wBd\,\frac{f_{cu}}{f_y}$$

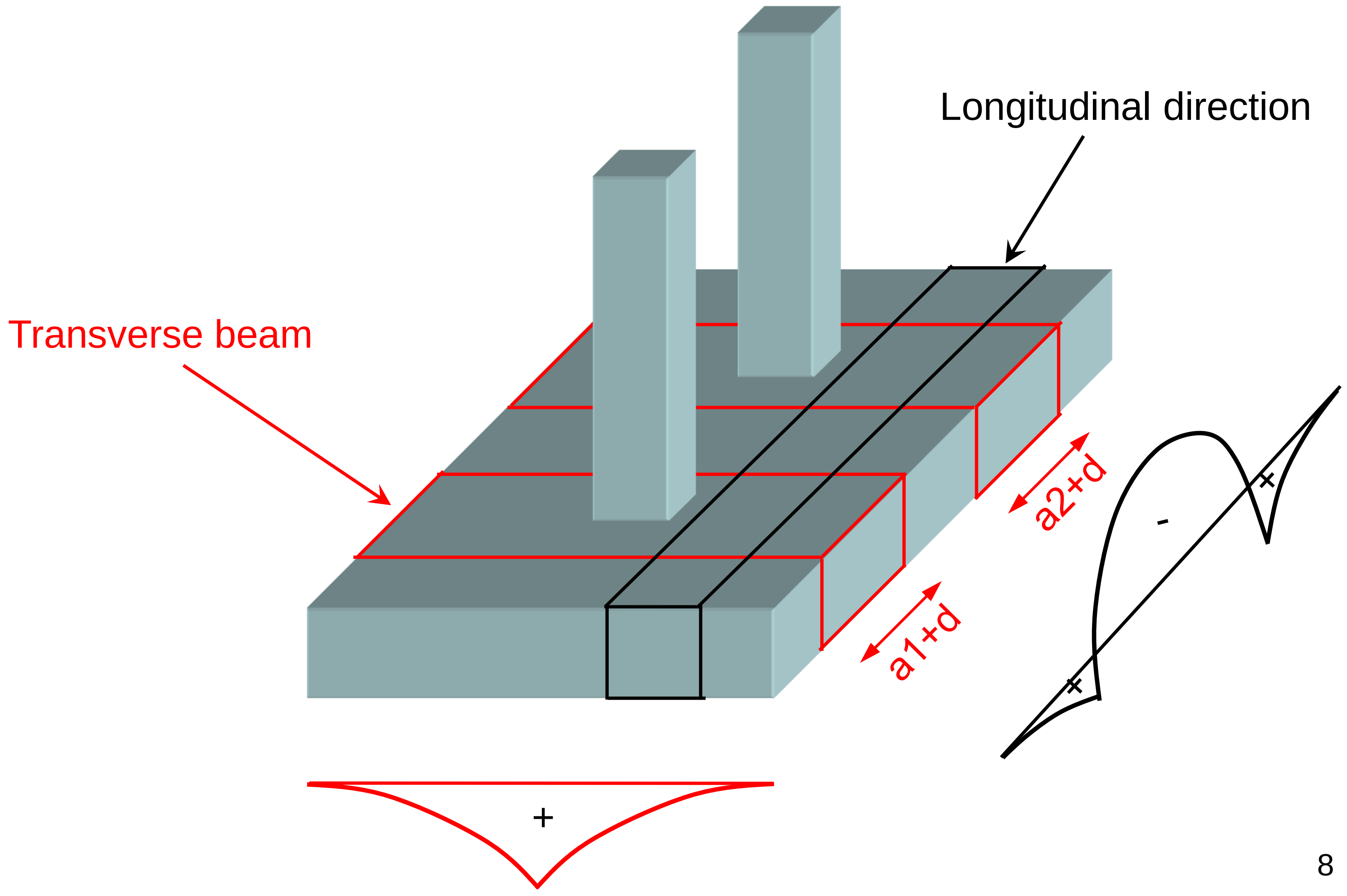
$$A_{s+ve}/m' = A_{s+ve}/B \qquad A_{s-ve}/m' = A_{s-ve}/B$$

$$A_{S)\min} = \frac{0.15}{100}(1000\ d)\,mm^2$$

$$\Phi_{\min} = 12\,mm\ \ \&\,S_{\max} = 200\ mm$$



Flexure in the transverse direction



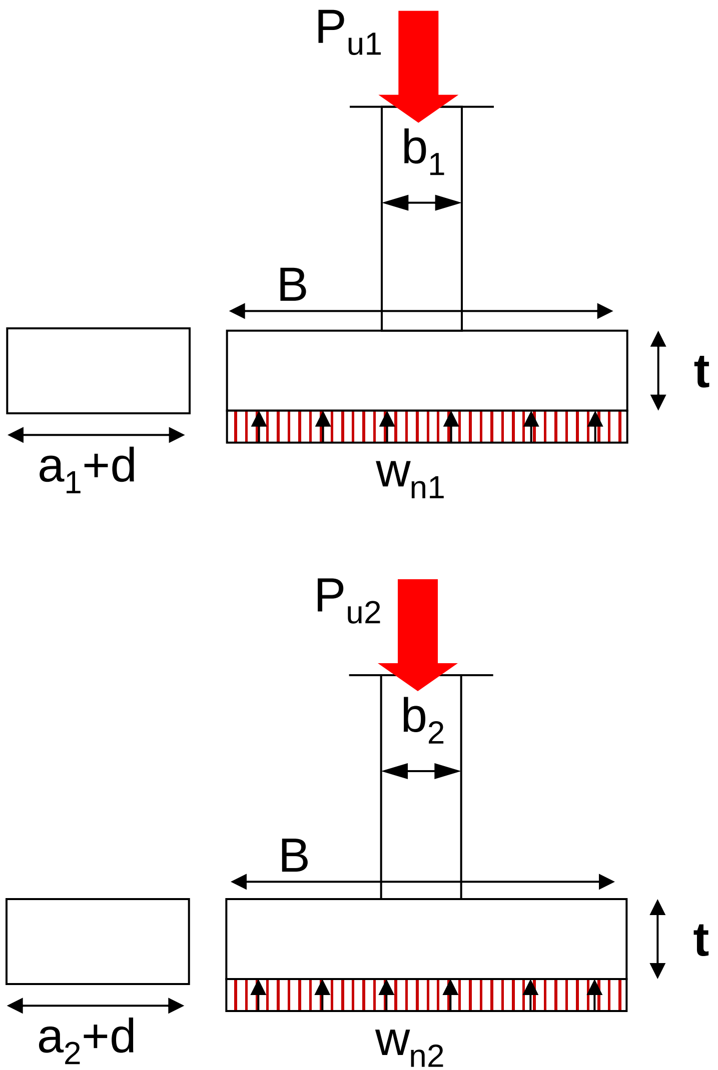
Flexure in the transverse direction

$$w_{n1} = \frac{P_{u1}}{B} \quad \& \quad w_{n2} = \frac{P_{u2}}{B}$$

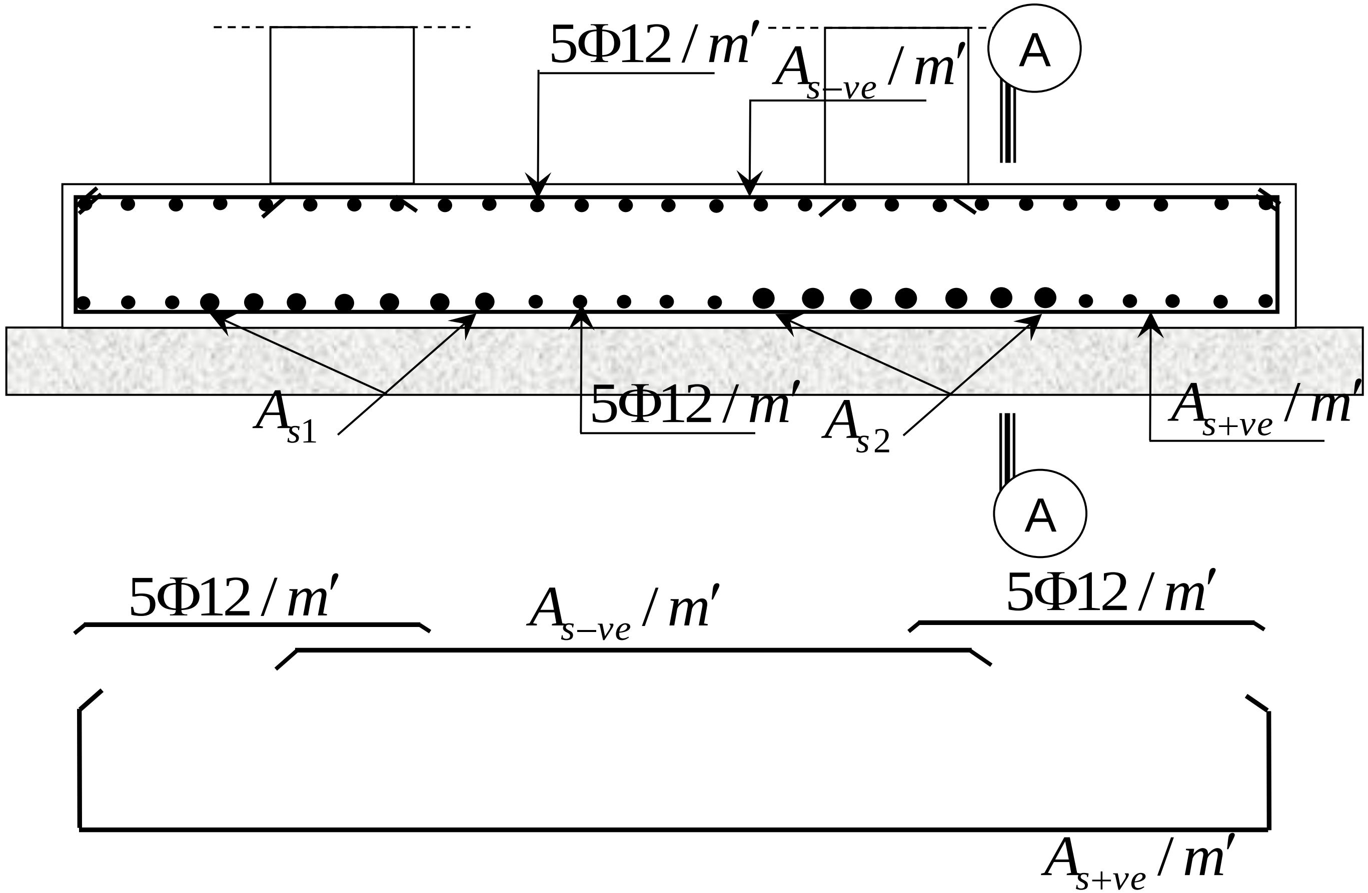
$$M_{u1} = \frac{w_{n1}(B-b_1)^2}{8} \quad \& \quad M_{u2} = \frac{w_{n2}(B-b_2)^2}{8}$$

d=t-90 mm d=t-90 mm

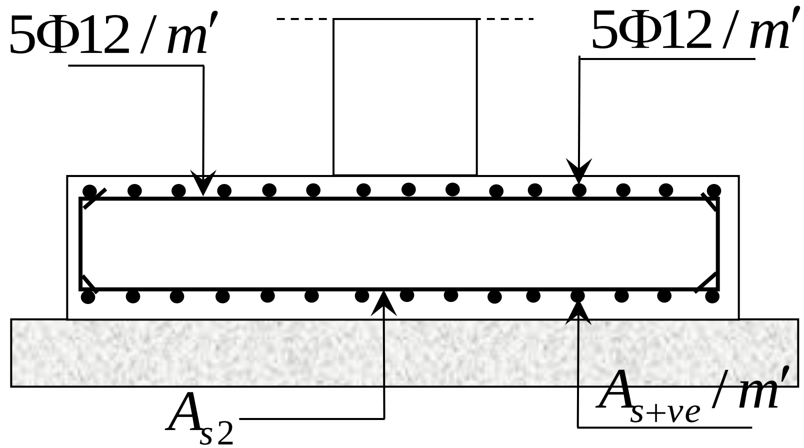
A_{s1} A_{s2}

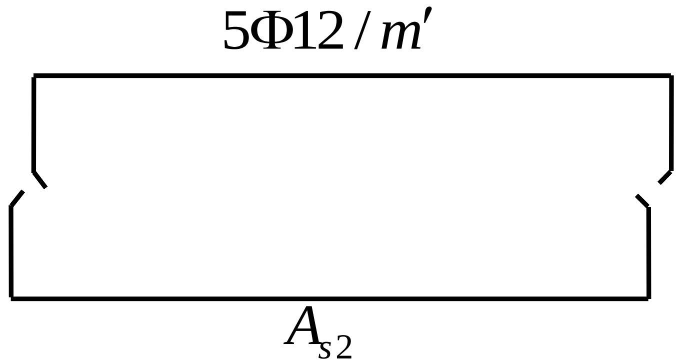


Details

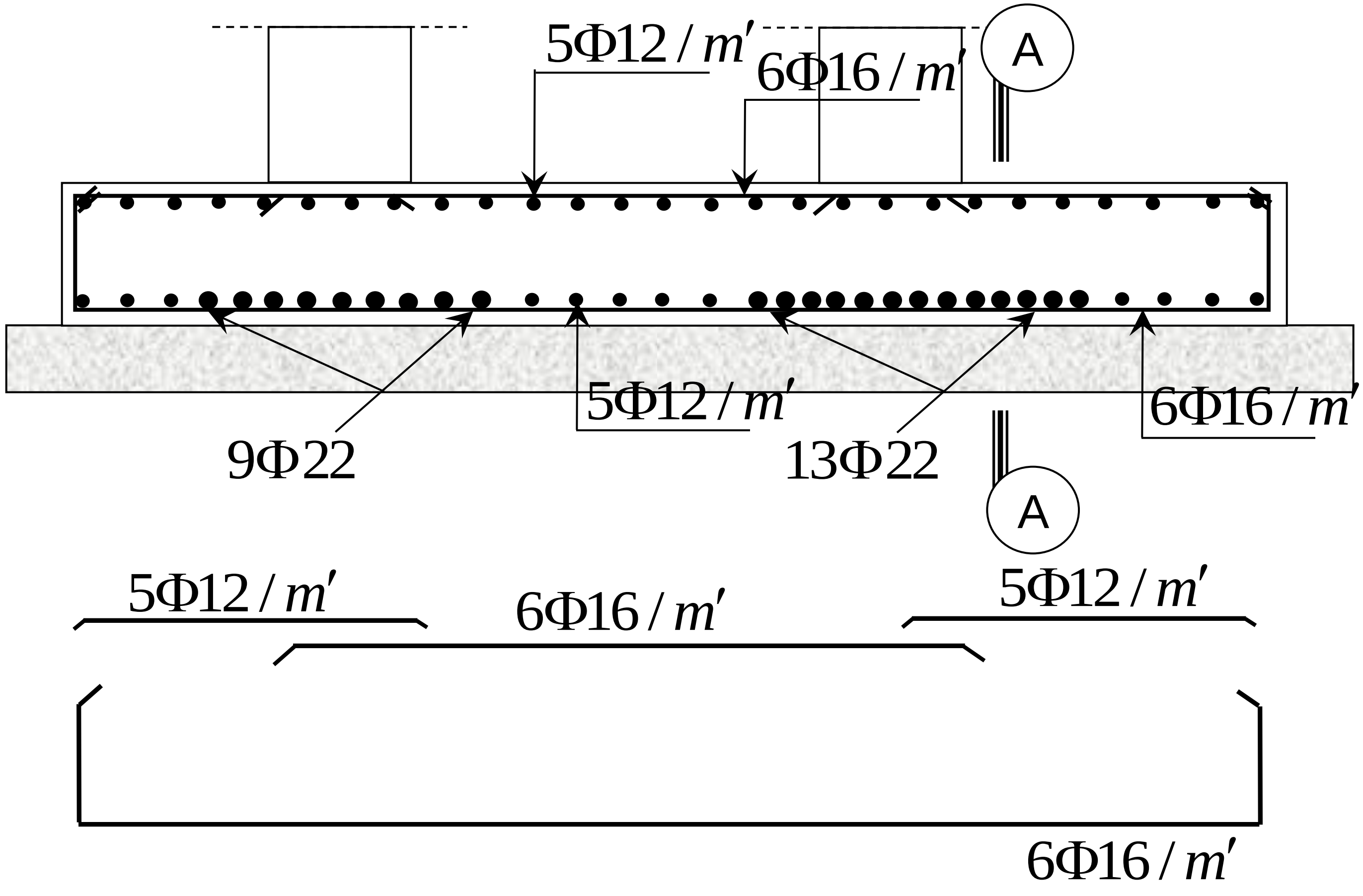

$5\Phi12/m'$
A_{s-ve}/m'
A
A_{s1}
$5\Phi12/m'$
A_{s2}
A_{s+ve}/m'
A
$5\Phi12/m'$
A_{s-ve}/m'
$5\Phi12/m'$
A_{s+ve}/m'

Details



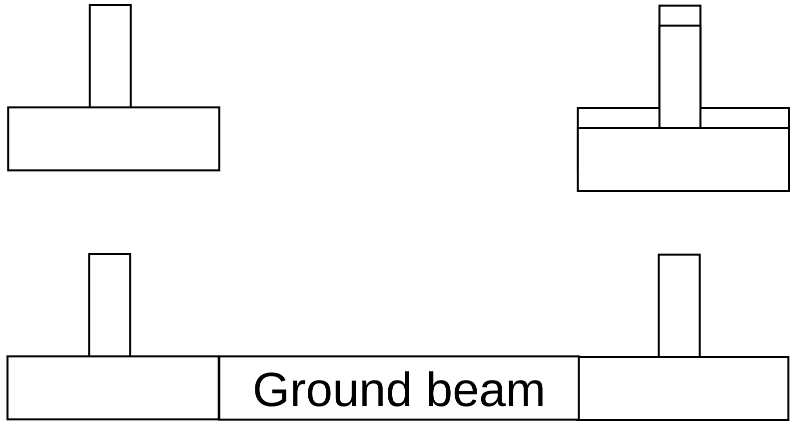


Details

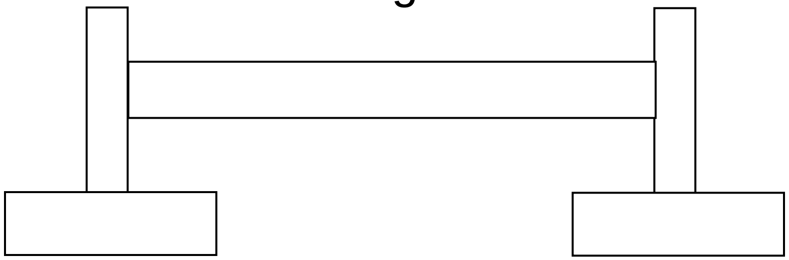

5Φ12 / m′
6Φ16 / m′
A
5Φ12 / m′
6Φ16 / m′
9Φ22
13Φ22
A
5Φ12 / m′
6Φ16 / m′
5Φ12 / m′
6Φ16 / m′

Ground Beams

Reduces differential settlement of footings



Strong soil



Weak soil