① COL. /08 +P. 531.F

1. CONCENTRICALLY LOADED SHORT COLUMNS (JACKSON MORELAND)

$$P_u \le \phi P_n \qquad \text{— (I)}$$

FACTORED NOMINAL

$\phi = 0.70$ for SPIRALLY reinforced column] —(II)
$\phi = 0.65$ " TIED "

For CONCENTRICALLY loaded column (SPIRALLY REINFORCED):

$$P_{n,max} = 0.85[0.85 f_c'(A_g - A_{st}) + f_y A_{st}] \qquad \text{III}$$

For CONCENTRICALLY loaded TIED column:

$$P_{n,max} = 0.80[0.85 f_c'(A_g - A_{st}) + f_y A_{st}] \qquad \text{IV}$$

ACI code requirements for SPIRALLY reinf. columns:

1. spiral size (diameter) $\ge$ 10 mm — V
2. 75 mm $\ge$ ℄ to ℄ spiral spacing $\ge$ 25 mm — VI
   CLEAR distance between spirals $\ge \frac{4}{3}$ Aggregate MAX SIZE — VII
3. Spiral ANCHORAGE $\ge$ 1.5 extra turns @ end — VIII
4. Spiral LAP splice $\ge 48\ d_b$, $\ge$ 300 mm — IX
5. $\rho_s \ge 0.45\left(\frac{A_g}{A_c} - 1\right) f_c'/f_y$ — X

ρ_s = ratio of spiral reinforcement volume to total core volume (out-to-out of spirals)

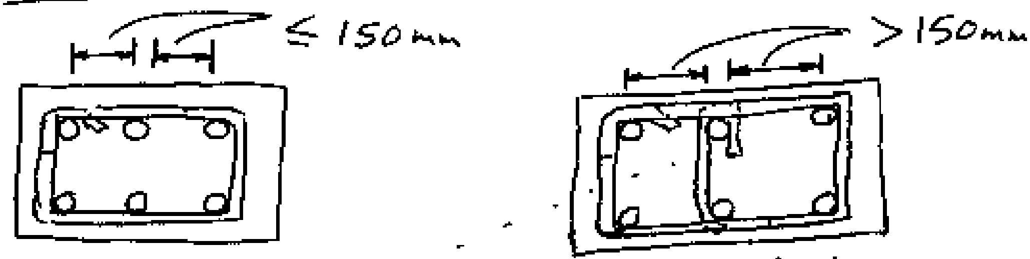
A_c = core area of spirally reinforced member measured to outside diameter of spiral.

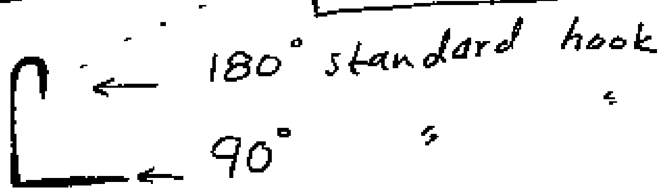
ACI code requirements for TIED columns:

Tie bar size (diameter) $\geq$ 12 mm — XI

Vertical Tie spacing $\left\{\begin{array}{l} \leq 16 \times \text{ longitudinal bar diameter} \\ \leq 48 \times \text{ tie bar diameter} \\ \leq \text{ least column dimension} \end{array}\right.$ — XII

TIE BAR ARRANGEMENT — XIII





Reinforcement (ρ_g) LIMITS: — XIV

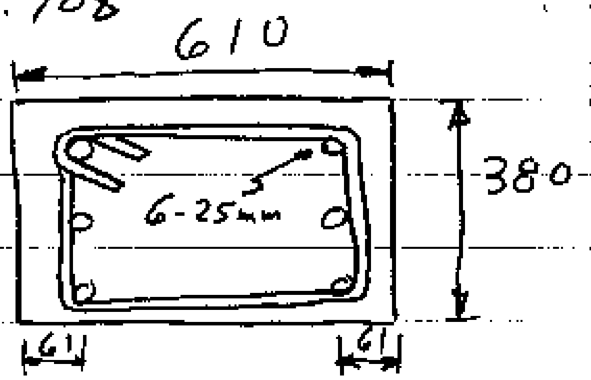
$$0.01 \leq \frac{A_{st}}{A_g} \leq 0.08$$ — XV

$\rho_g = A_{st}/A_g$

For LAP spliced longitudinal bars: — XVI

$A_{st}/A_g \leq 0.04$

No. of LONGITUDINAL BARS $\left\{\begin{array}{l} \geq 4 \text{ for bars in TIED cols. or} \\ \geq 6 \text{ for bars in SPIRALLY reinforced cols.} \\ \geq 3 \text{ for bars in TIED cols.} \end{array}\right.$



Ex.1 Calculate P_u

Concentric loading:

e = eccentricity = 0

$f'_c = 20.7\ N/mm^2$

$f_y = 345$ "

Solution:

$A_{st} = 6 \times 490 = 2940\ mm^2$

$\rho_g = \dfrac{A_{st}}{A_g} = \dfrac{2940}{610 \times 380} = 0.0127 \left.\begin{matrix} > .01\ (min) \\ < .08\ (max) \end{matrix}\right\} \therefore OK$

"TIED" column

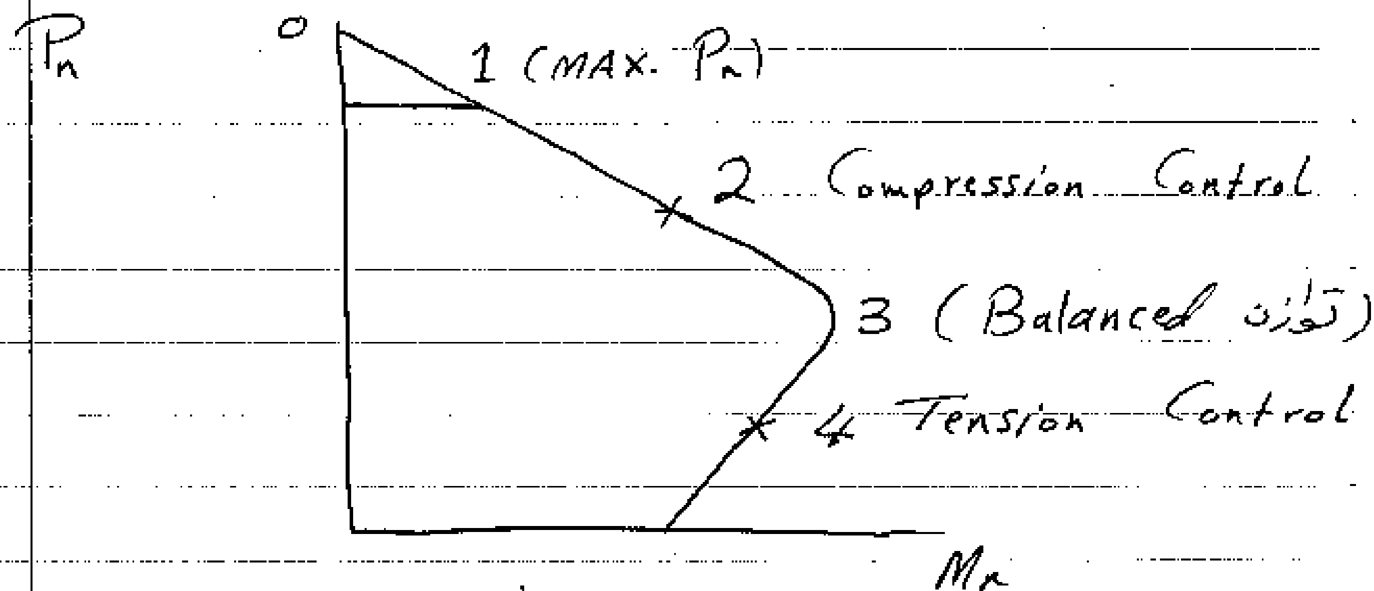
$P_{n,max} = 0.80\,[.85 f'_c (A_g - A_{st}) + f_y A_{st}]$

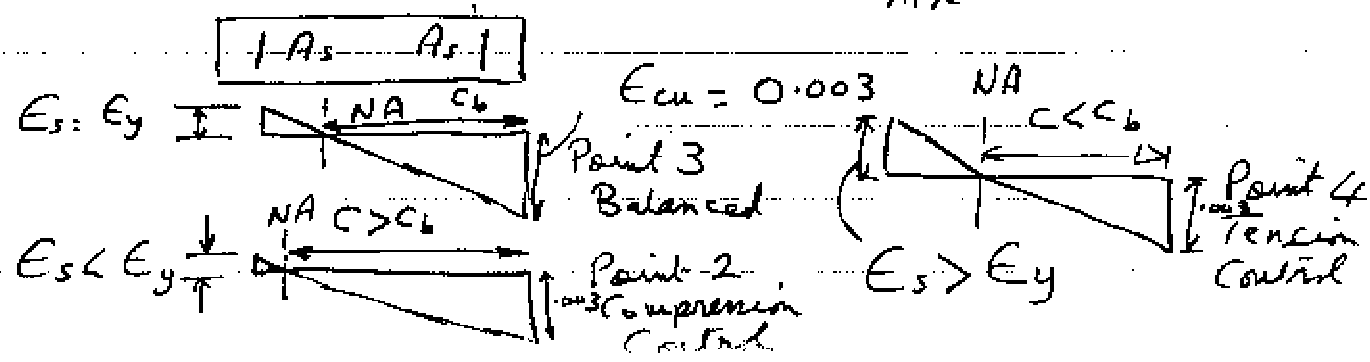
$= .8\,[.85 \times .0207\,(610 \times 380 - 2940) + .345 \times 2940]$

$= 4033$ kN "NOMINAL" concentric load capacity

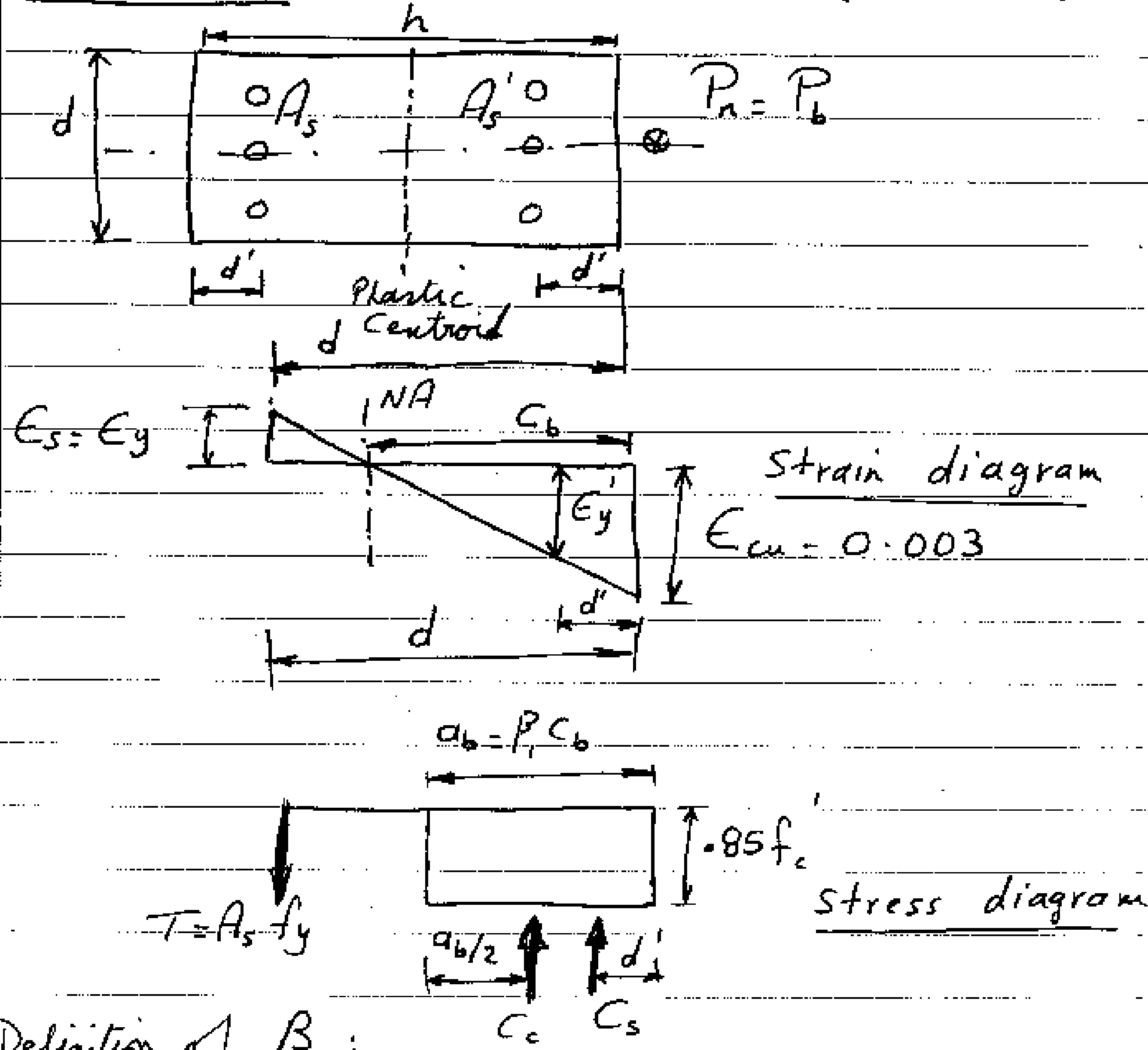
$P_u = \phi P_n = \overset{\text{TIED}}{.65} \times 4033 = 2621$ kN the max. allowable factored load for a "CONCENTRICALLY" ($e = 0$) loaded column.

2. "ECCENTRICALLY" loaded columns (SHORT قصيرة)





First case: Balanced Condition (Point 3)



Definition of β_1:

$\beta_1 = 0.85$ for $f_c' \leq 28\ N/mm^2$

$\beta_1 = 0.65$ " $f_c' \geq 56$ "

$\beta_1 = 0.85 - (.05/7)(f_c' - 28)$ for $28 < f_c' \leq 56\ N/mm^2$

Per ACI code: "Balanced Condition" occurs when simultaneously the tension steel A_s reaches ϵ_y ($\epsilon_y = f_y/E_s$), as the concrete fibre on the maximum compression side reaches a strain $\epsilon_{cu} = 0.003$

Similar Δ's:

$c_b/d = 0.003/(f_y/E_s + 0.003)$

ACI code gives $E_s = 200 \times 10^3$ N/mm²

$$\therefore C_b = d \frac{600}{f_y + 600}$$

Two possibilities: (a) or (b)

(a) If compression steel yields: $(\epsilon_s' \geq \epsilon_y)$

$$C_s = A_s'(f_y - 0.85 f_c')$$

or (b) If compression steel does not yield: $(\epsilon_s' < \epsilon_y)$

$$C_s = A_s'(\epsilon_s' E_s - 0.85 f_c')$$

$$T = A_s f_y$$

$$C_c = 0.85 f_c' a_b = 0.85 f_c' \beta_1 C_b b$$

Summing forces:

$$P_b \text{ [NOMINAL]} = C_c + C_s - T$$

Summing moments about PC (Plastic Centroid)

$$P_b e_b = C_s (h/2 - d') + C_c (h/2 - a/2) + T(d - h/2)$$

ϕ values for symmetrical sections ($A_s' = A_s = A_{st}/2$ & P.C is @ mid-depth) with $f_y \leq 420$ N/mm² and $(h - 2d')/h \geq .7$; then the following equations apply for "TENSION CONTROL":

For a SPIRALLY reinforced column:

$$\phi = \frac{0.9}{1 + 2P_n/(f_c' A_g)} \geq 0.7 \leftarrow \text{ضد اذن}$$

For a TIED column:

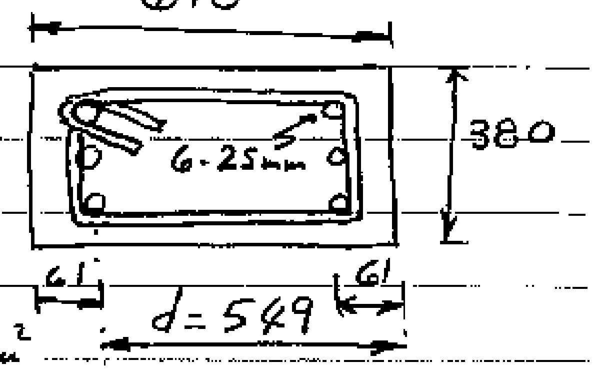
$$\phi = \frac{0.9}{1 + 2.5 P_n/(f_c' A_g)} \geq 0.65 \leftarrow \text{ضد اذن}$$

⑥ COL. /08

Ex2 Calculate the BALANCED load capacity of the tied column. $f'_c = 20.7\ N/mm^2$; $f_y = 345\ N/mm^2$

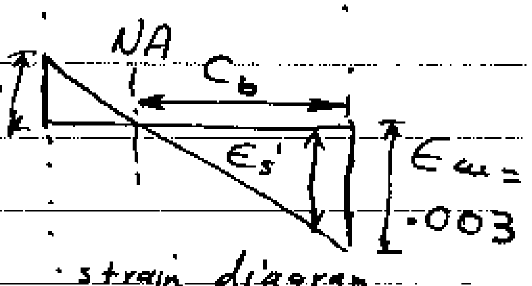
$A_s = A'_s = A_{st}/2 = 1470\ mm^2$

$[A_{st} = 2940\ mm^2]$



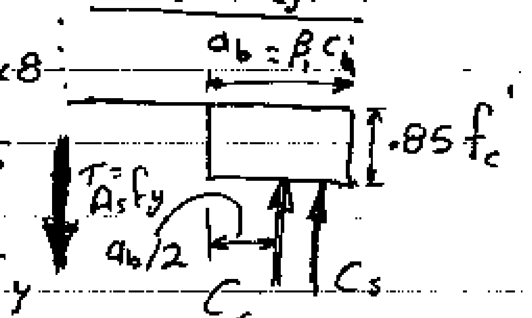
Solution: $d = 610 - 61 = 549\ mm$

$c_b = d \dfrac{600}{f_y + 600} = 549 \dfrac{600}{945} = 348.6\ mm$



$\epsilon'_s = 0.003 \dfrac{348.6 - 61}{348.6} = .00248$

$\epsilon_s = \epsilon_y = f_y/E_s = \dfrac{345}{200 \times 10^3} = .001725$



$\epsilon'_s = .00248 > f_y/E_s = .001725 = \epsilon_y$

∴ COMPRESSION steel yields; $f'_c = 20.7\ N/mm^2$; ∴ $\beta_1 = .85$

$C_c = 0.85 \times .0207 \times .85 \times 348.6 \times 380 = \underline{\underline{1981}}\ kN$ ∴ $a = 296.3$

$T = .345 \times 1470 = \underline{\underline{507}}\ kN$

$C_s = 1470(.345 - .85 \times .0207) = \underline{\underline{481}}\ kN$

من هذه القوى الثلاث (C_c, T, C_s) يمكن ايجاد الكل :

$P_b = P_n = C_s + C_c - T = 481 + 1981 - 507 = 1955\ kN$

BALANCED, NOMINAL

Taking moment about the PLASTIC CENTROID (الوسط):

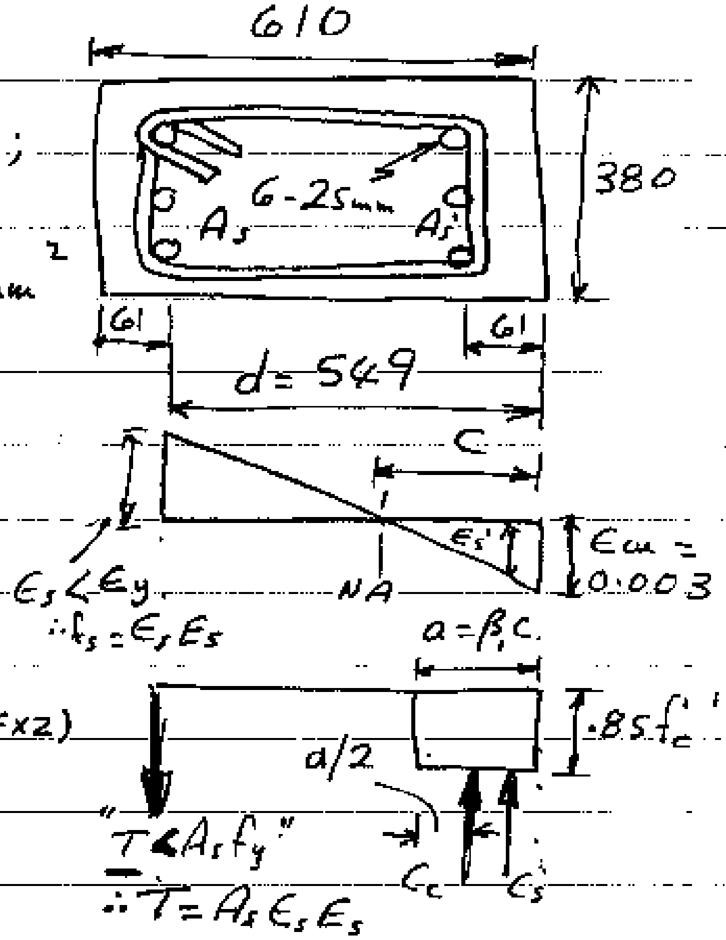
$P_b e_b = \overset{481}{C_s}(305 - 61) + \overset{1981}{C_c}(305 - 296.3/2) + \overset{507}{T}(549 - 305)$; $P_b = 1955$ نعوض ∴ $e_b = 282$

∴ At "Balanced" conditions:

NOMINAL			FACTORED	
P_b, kN	e_b, mm	M_b, kNm	P_{ub}, kN	M_{ub}, kNm
1955	282.2	551.8	1271	358.7

Note $P_{ub} = .65\ P_b$ — ϕ اكمل الحل :

Ex 3 Find the column capacity when $e = 203$ mm; $f_c' = 20.7\ N/mm^2$, $f_y = 345\ N/mm^2$; $A_s = A_s' = 1470\ mm^2$



Solution:

Based on EX2 and since $e = 203$ mm $< e_b = 282.2$ mm, then COMPRESSION controls

$\therefore c > c_b = 348.6$ mm (EX2)

$\therefore \epsilon_s' > \epsilon_y$ (see EX2)

& $\epsilon_s < \epsilon_y$

$C_s = A_s'(f_y - .85 f_c')$

$= 1470(.345 - .85 \times .0207) = 481.3$ kN, as in EX 2

Put $c = x$, $\therefore C_c = .85 f_c' b (.85x)$ $[\beta_1 = .85]$

$\therefore C_c = .85 \times .0207 \times 380 \times .85x = 5.683x$ kN

$T = A_s f_s = 1470\left[\frac{.003(549 - x)}{x} \times 200\right]$ kN

$= \frac{484218 - 882x}{x}$ kN

Taking moments about the "NOMINAL" load P_n:

$$0 = 481.3(305 - 61 - 203) - 5.683x(.85x/2 - 305 + 203) + \frac{484218 - 882x}{x}(549 - 305 + 203)$$

Simplifying, we get:

$$0 = x^3 - 240x^2 + 155063x - 89615054 = 0$$

Solution to the nearest mm: $x = 4.04$ mm

$\therefore C_c = 2295.9$ kN & $T = 316.6$ kN

$P_n = \Sigma$ forces $= 481.3 + 2295.9 - 316.6 = 2460.6$ kN

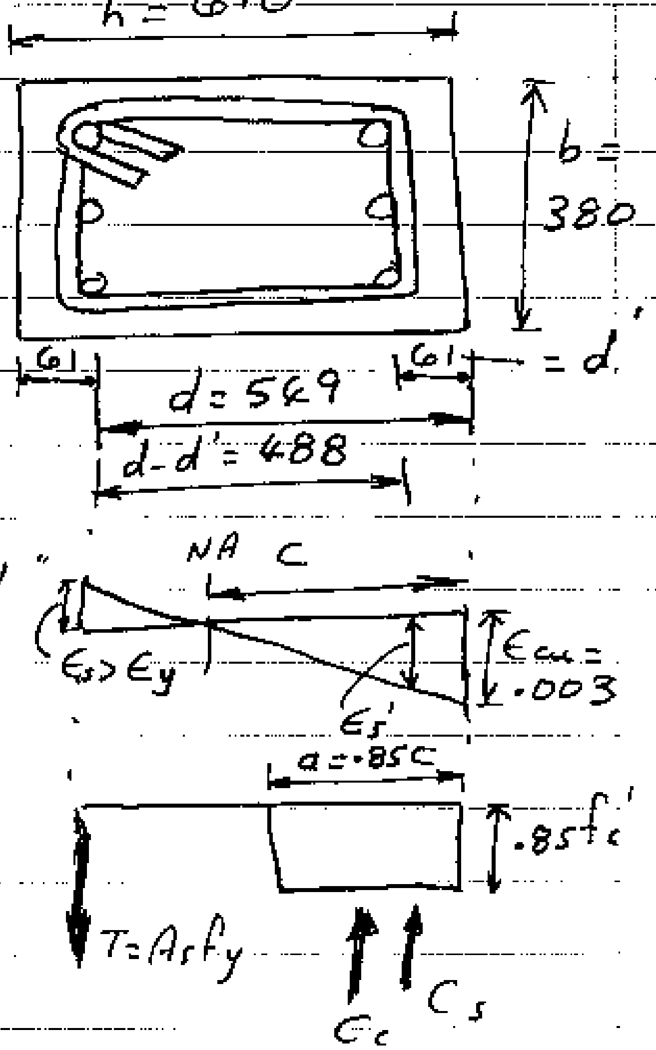
$M_n = 2460.6 \times .203 = 499.5$ kNm

Summary for $e = 203$ mm :

NOMINAL			FACTORED	
P_n, kN	e, mm	M_n, kNm	P_u, kN	M_u, kNm
2460.6	203	499.5	1599	324.7

using $\phi = 0.65$ (TIED COL.)

Ex. 4 $A_s = A_s' = A_{st}/2 = 1470$ mm^2, $f_c' = 20.7$ N/mm^2; $f_y = 345$ N/mm^2. With $e = 508$ m, calculate column capacity.



Solution :

Based on EX2 and since $e > e_b = 282.2$ mm, "TENSION" controls : $c > c_b$ & $\epsilon_s > \epsilon_y$

$\Rightarrow$ Start calculations by assuming that $\epsilon_s' \geq \epsilon_y$, and then check this assumption before the end of solution.

$T = A_s f_y = 1470 \times .345 = 507.2$ kN   put $c = x$

$C_c = .85 f_c' ab = .85 \times .0207 \times 380 \times .85x$

$= 5.683\, x$

$C_s = A_s'(f_y - .85 f_c') = 1470\,(.345 - .85 \times .0207)$

$= 481.3$ kN

P_n[NOMINAL] $= \Sigma$Forces $= C_c + C_s - T$

$= 5.683\,x + 481.3 - 507.2 = 5.683x - 25.9$

Taking moments about ℄ tension steel:

$$P_n\left(e + \frac{d-d'}{2}\right) = C_c\,(d - a/2) + C_s\,(d - d')$$

$$(5.683x - 25.9)(508 + 244) = 5.683x\,(549 - .425x)$$
$$+ 481.3 \times 488$$; Simplifying, we get:

$$x^2 + 477.6x - 105309.4 = 0$$; solve: $x = 164.1$ mm

$\therefore C_c = 5.683x = 932.6$ kN; $P_n = 932.6 - 25.9 = 906.7$ kN

Check the assumption that $\epsilon_s' \geq \epsilon_y$ [i.e. $f_s' = f_y$] :-

$$\epsilon_s' = \frac{164.1 - 61}{61} \times .003 = .00188$$

$$\epsilon_y = f_y / E_s = 345/(200 \times 10^3) = .001725 < .00188$$ $\therefore$ COMP STEEL YIELDS AS ASSUMED

NOW FIND ϕ FOR TIED COLUMN:

$$\phi \leq \frac{.9}{1 + 2.5 P_n / (f_c' A_g)} \geq 0.65$$ (الغرض تكبير ϕ وليس تقليلها)

$$\therefore \phi = \frac{.9}{1 + 2.5 \times 906.7 / (.0207 \times 610 \times 380} = .611 \geq 0.65$$

Using $\phi = 0.65$ FOR "TIED" COL. GOVERNS (الأكبر)

P_n, kN	e, mm	M_n, kNm	P_u, kN	M_u, kNm
906.7	508	460.6	589.4	299.4
← NOMINAL →			← FACTORED →	

ملاحظة ان الحل بطريقة رياضية (Mathematical) يستغرق وقت طويل لا داعي له حيث ان ذلك يحتاج الى معادلات قد تصل الى الدرجة الثالثة. لذلك يفضل ايجاد طريقة اسرع واكثر كفاءة لمعرفة "مقاومة الاعمدة القصيرة ذات العزم احادي المحور"

(SHORT COLUMNS UNDER UNIAXIAL BENDING)

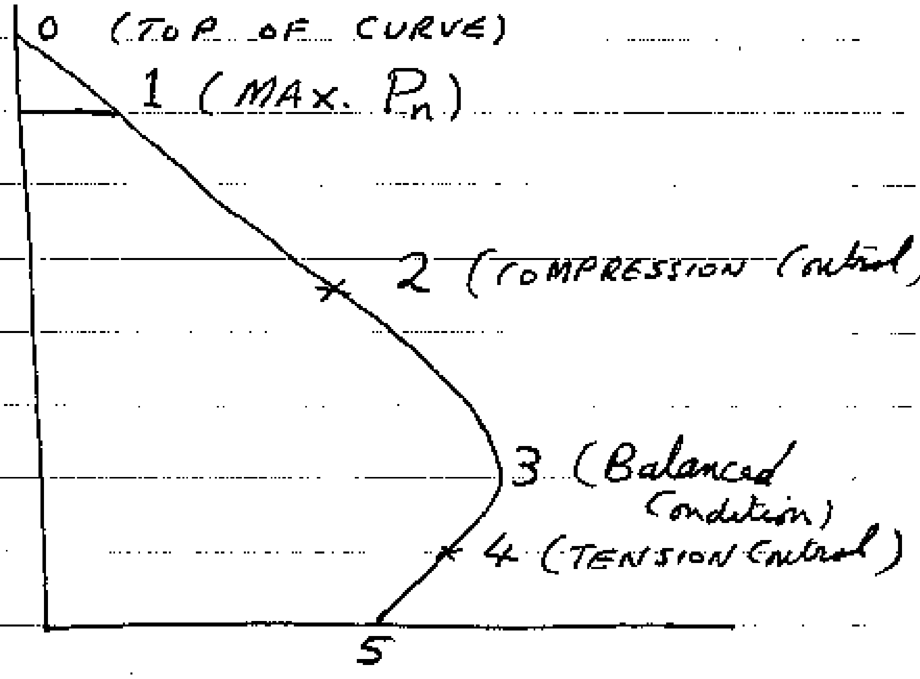
هذه الطريقة هي:

COLUMN INTERACTION DIAGRAM

(See pp 493-496 FERGUSON)

COLUMN INTERACTION DIAGRAM (CID)

Following is the CID for RC columns based on the latest ACI 318 Code:



Important Points in "CID"

0: Top of curve لا يحتاجها

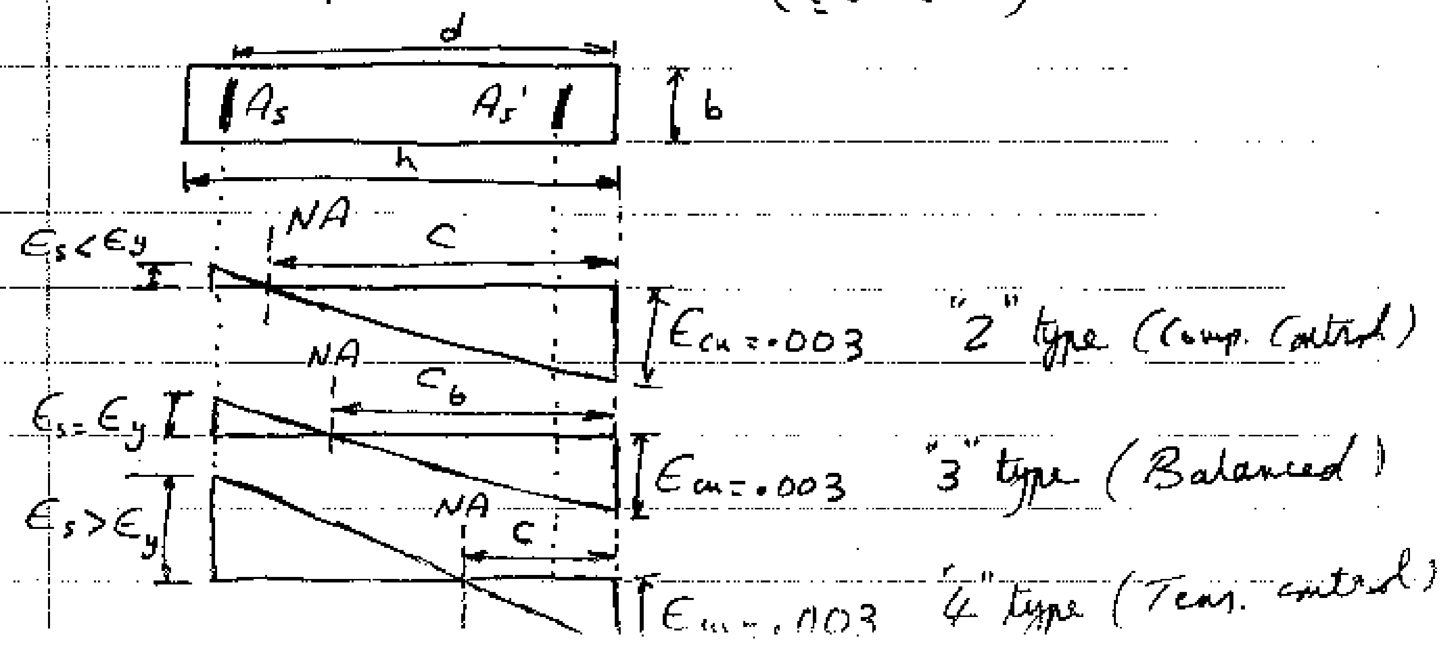
1: MAX P_n (Factor of 0.80 → TIED Columns)

2: COMPRESSION Control → $\epsilon_s < \epsilon_y$

3: Balanced Condition → $\epsilon_s = \epsilon_y$

4: TENSION Control → $\epsilon_s > \epsilon_y$

5: NO axial load (نقطة نقية)



Use FERGUSON pp 493 - 496 interaction diagrams depending on the value of $\underline{\underline{\gamma}}$.

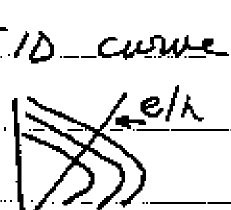
Equations to be used:

$\rho_t = A_{st}/(bh)$ (I): $A_{st} = A_s + A_s'$ (مجموع حديد الطرفين)

$\mu = f_y/(0.85 f_c')$ (II)

$\rho_t \mu$ (III) (Use to find C/D curve)

e/h (IV)



Correction Factor:

$$P_u = P_{uF} \times \frac{0.65}{0.70} \quad (V)$$

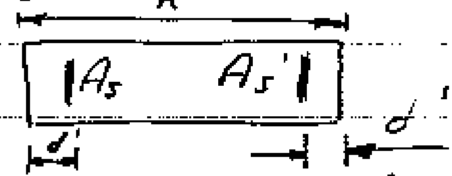
P_u: Factored (مطلوب ايجاده)

P_{uF}: من FERGUSON

معامل التصحيح بسبب تغير φ من (0.7) في (ACI) القديم الى (0.65) في (ACI) الجديد

يقرر $\underline{\underline{\gamma}}$ اي صفحة نستعمل من 493-496

$$\gamma = \frac{h - 2d'}{h} \quad (VI)$$



Note: $A_s = A_s' = \frac{A_{st}}{2}$

ملاحظة: الصفحات التالية (493-496) المرقمة (12)، (13)، (14)، (15) نستعمل لكل امثلة لايجاد مقاومة العمود FACTORED ← "SHORT TIED COLUMN UNDER UNIAXIAL BENDING"

Note: To obtain P_{uF} (see pp 493-496 Ferguson):

$f_c' \le 28$ N/mm² (VII)

$f_y = 400$ " (VIII)

; then apply the correction of Eq. (V)

(12) Col./08

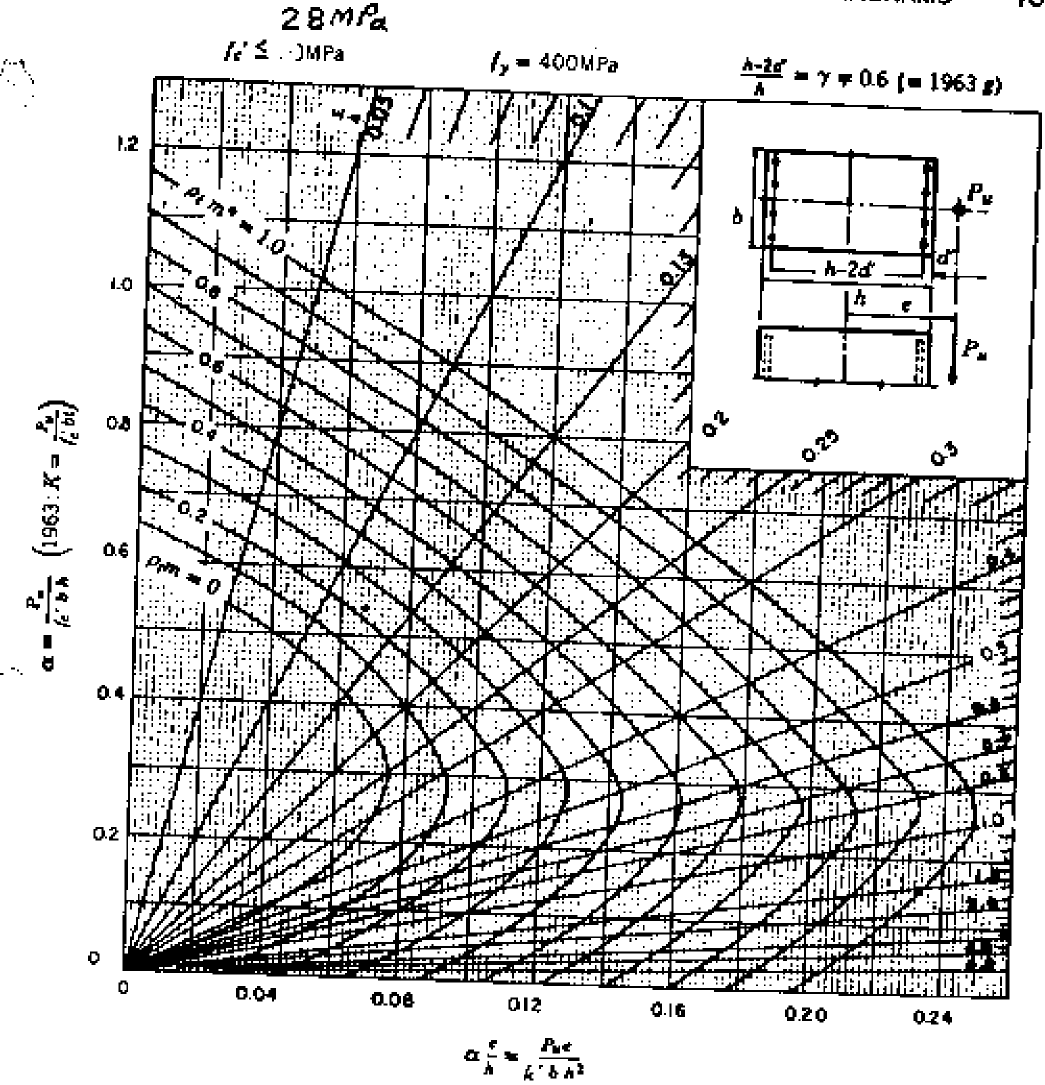


FIGURE 18.12a Column Interaction Chart. $\gamma = 0.6$. $m = f_y/0.85 f_c'$ is 1963 notation. m now should become a lower-case Greek letter. Chart readings include the effect of ϕ, but not correctly below ordinate of 0.10.

lution

(13) Col. /08

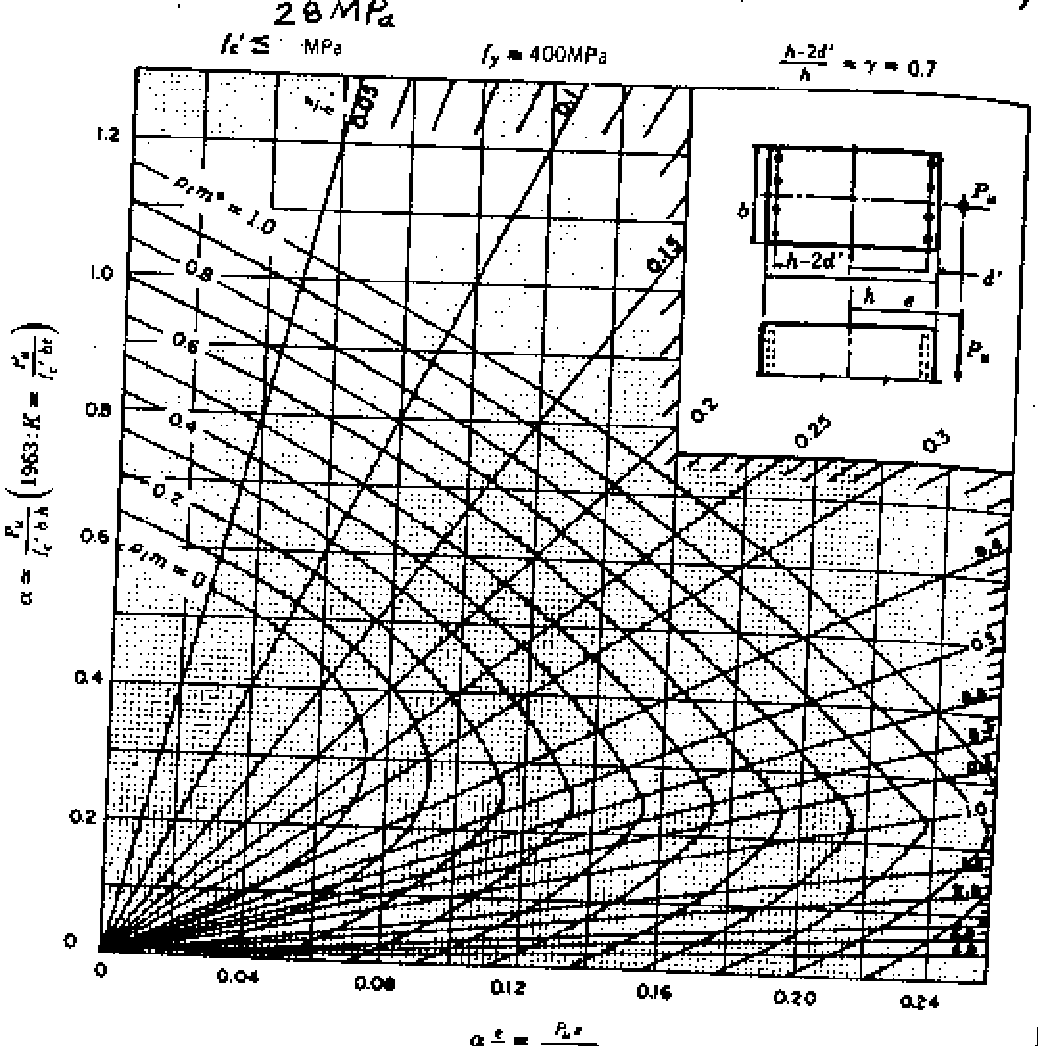


FIGURE 18.12b Column Interaction Chart. $\gamma = 0.7$. Chart readings include the effect of ϕ, but not correctly below ordinate ...

(14) Col./08

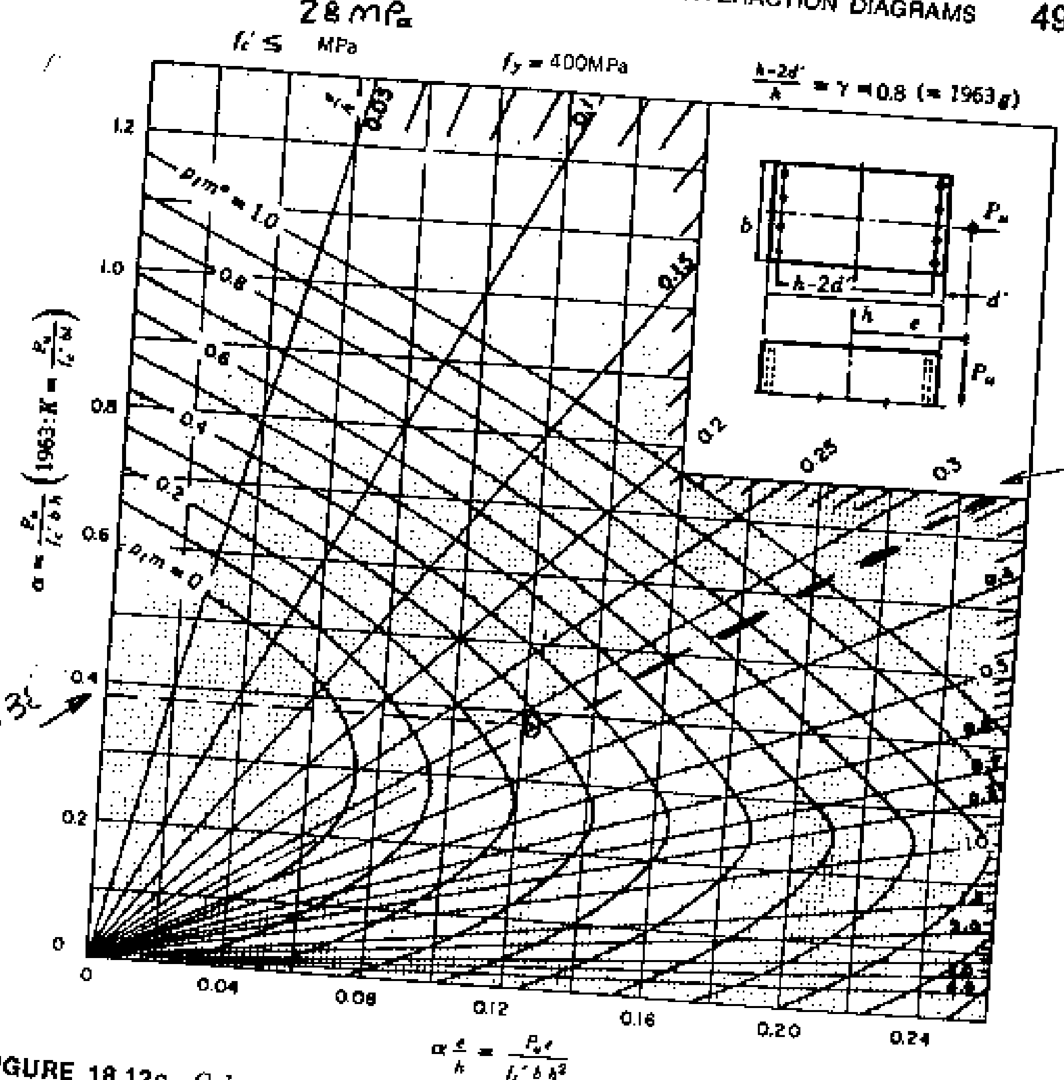


FIGURE 18.12c Column Interaction Chart. $\gamma = 0.8$. Chart readings include the effect of ϕ, but not correctly below ordinate of 0.10.

EX 5 solution:

$$\alpha_F = 0.38$$

$\therefore P_{uF} = .38 \times .0207 \times 380 \times 610 = 1823$ kN

P_u (وضع1) $= \frac{.65}{.7} \times \leftarrow = 1693$ kN

M_u (s) $= .203 \times 1693 = 343.6$ kNm

(15) COL 03

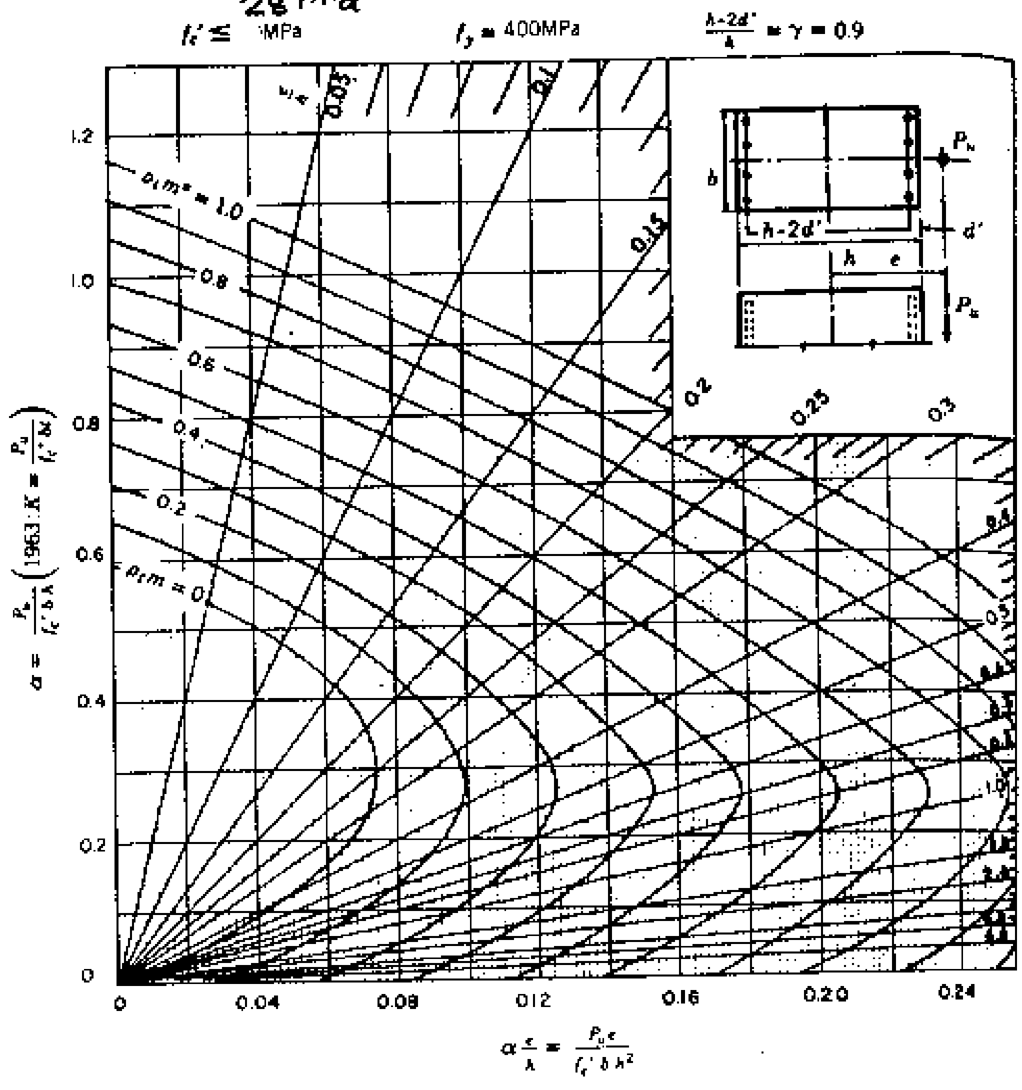


FIGURE 18.12d Column Interaction Chart. $\gamma = 0.9$. Chart readings include the effect of ϕ, but not correctly below ordinate of 0.10.

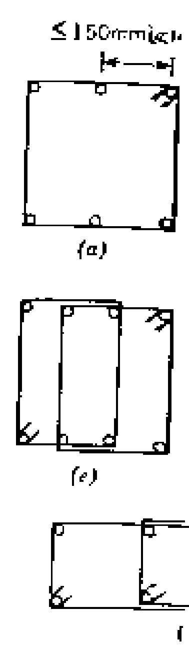


FIGURE 18.13

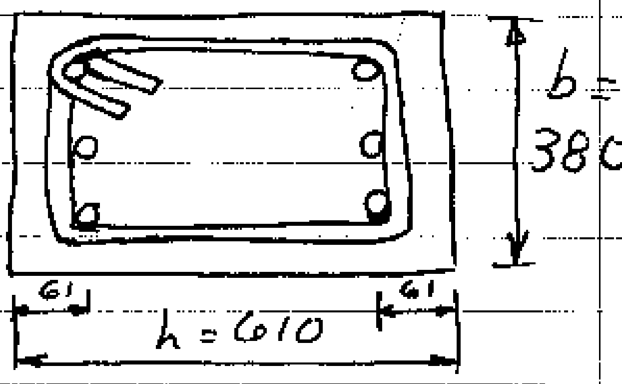
18.11 COLUMN T

Column ties n
are rather sim
no bar more th
smallest of:

1. 16 d_b for t
2. 48 d_b for t
3. The minin

A few typic

EX5 Find the section capacity (tied column) when $e = 203$ mm ; $f_c' = 20.7$ N/mm^2; $f_y = 400\ N/mm^2$; $A_s = A_s' = A_{st}/2 = 1470\ mm^2$.



Solution :

$\rho_t = A_{st}/(bh)$ (I)

$= 2940/(610 \times 380)$

$= 0.01268 \geq .01$ (OK) & $\leq .08$ (OK)

$\mu = f_y/(0.85 f_c')$ (II)

$= 400/(.85 \times 20.7)$

$\therefore \rho_t \mu = .01268 \times 400/(.85 \times 20.7) = 0.288$

$e/h = 203/610 = 0.333$

$\gamma = (h - 2d')/h$ (VI)

$= (610 - 2 \times 61)/610 = 0.8$; $\therefore$ GOTO p. 495 F

with $e/h = 0.333$ & $\rho_t \mu = 0.288$:- $\alpha_F = 0.38$

$\therefore 0.38 = \dfrac{P_{uF}}{f_c' b h} = \dfrac{P_{uF}}{.0207 \times 380 \times 610}$

$\therefore P_{uF} = 1823$ kN (Based on OLD $\phi = 0.7$)

$\therefore P_u = P_{uF} \times \dfrac{.65 \text{ (جديد)}}{.7 \text{ (قديم)}} = \underline{\underline{1693}}$ kN

الجواب الخطأ

الجواب النهائي

$\therefore M_u = .203 \times 1693 = \underline{\underline{343.6}}$ kNm

EX6: Find the column capacity with $e = 508$ mm.

Solution: $\rho_t \mu = 0.288$ (Ex 5)

$e/h = 508/610 = 0.833$; goto p. 495 F :-

$\alpha_F = 0.16$; $\therefore .16 = \dfrac{P_{uF}}{.0207 \times 380 \times 610}$

$\therefore P_{uF} = 768$ kN

$P_u = (.65/.7)\ 768 = \underline{713}$ kN

$M_u = .508 \times 713 = \underline{\underline{362}}$ kNm

(17) COL. /08

ϕ is assumed to be 0.65 ; check ϕ:

$\therefore P_n = 713/.65 = 1097$

$$\phi = \frac{.9}{1 + 2.5 P_n/(f_c' A_g)} \geq 0.65$$

$$= \frac{.9}{1 + 2.5 \times 1097/(.0207 \times 610 \times 380)} \geq .65$$

$= 0.573 \geq 0.65$

GOVERNS (GREATER)

$\therefore$ Final answer:

$P_u = 713$ kN

$M_u = 362$ kNm

COLUMN CAPACITY

(18) COL. /08

BIAXIAL MOMENTS IN COLUMNS

USE BRESLER RECIPROCAL method:

$$\frac{1}{P_u} = \frac{1}{P_{ux}} + \frac{1}{P_{uy}} \quad \frac{1}{P_{uo}}$$

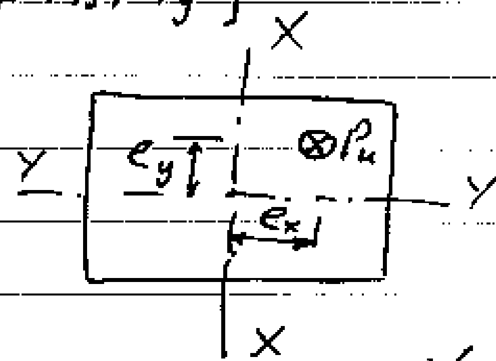
P_u = Factored moment load capacity with e_x & e_y

P_{ux} = " " " " with e_x only ($e_y = 0$)

P_{uy} = " " " " " " e_y " ($e_x = 0$)

$P_{uo} = \phi\left[0.85 f_c'(A_g - A_{st}) + A_{st} f_y\right]$

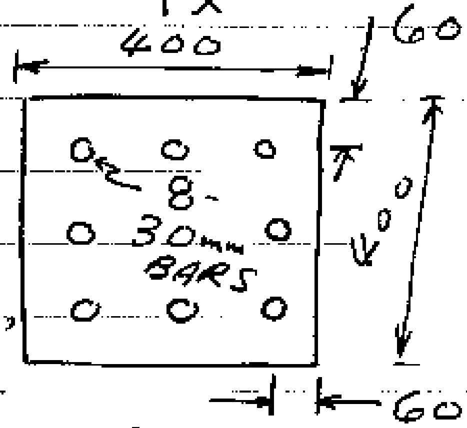
See diagram:



$e_x = \frac{M_{ux}}{P_u}$

$e_y = \frac{M_{uy}}{P_u}$

Ex: Calculate P_u with $M_{ux} = 151.4$ kNm & $M_{uy} = 67.8$ kNm. The tied column is 400 x 400 mm, $f_y = 400\ N/mm^2$, $f_c' = 21 N/mm^2$.



Start ITERATION with P_{uF} = 640 kN.

Solution: Since the process involves ITERATION, it is better to TRANSFORM to P_{uF} & M_{uF}. At the end with the ratio of (.65/.7), P_u and M_u will go back to their required values:

$M_{uF} = (0.7/0.65) M_u$ & $P_{uF} = (.7/.65) P_u$

$\therefore M_{uxF} = (.7/.65) 151.4 = 163$ kNm

$M_{uyF} =$ " $67.8 = 73$ "

$\rightarrow$ Start with P_{uF} @ 640 kN

Ferguson INT. DIAGRAM :

$e_x = M_{uxF}/P_{uF} = 163\times10^6/(680\times10^3) = 255$ mm

$e_y = M_{uyF}/P_{uF} = 73\times10^6/(640\times10^3) = 114$ mm

If we neglect (on the safe side) the two bars in the centre of the section, we obtain:

$A_{st} = 706\times 6 = 4236$ mm^2

$\rho_t = A_{st}/(bh) = 0.0265$

$\mu = f_y/(.85 f_c') = 400/(.85\times21) = 22.41$

$\rho_t\mu = .0265\times 22.41 = 0.593$

$e_x/h = .638$; $e_y/h = .285$; $\gamma = 0.7$

GOTO p. 496 Ferguson:

$\alpha_x = .29$; $P_{uxF} = .29\times .021\times 400^2 = 974$ kN

$\alpha_y = .50$; $P_{uyF} = .5\times .021\times 400^2 = 1680$ "

$P_{no} = .85 f_c' (A_g - A_{st}) + A_{st} f_y$

$= .85\times .021 (400^2 - 5668) + 5668\times .4 = 5014$ kN

$P_{uoF} = .7\times 5014 = 3510$ kN Ferguson value.

$$\therefore \frac{1}{P_{uF}} = \frac{1}{P_{uyF}} + \frac{1}{P_{uxF}} - \frac{1}{P_{uoF}}$$

$$= \frac{1}{1680} + \frac{1}{974} - \frac{1}{3510} \;\; ; \therefore P_{uF} = 768 \text{ kN}$$

HOMEWORK: Try a few iterations, increasing P_{uF} in each case.

After a few cycles:

→ Try $P_{uF} = 1180$ kN (بداية الدورة)

$e_x = 163\times10^6/(1180\times10^3) = 138$ mm ; $e_x/h = .345$ $\Big\}$ $\rho_t\mu = .593$

$e_y = 73\times10^6/$ " $= 61.9$ " ; $e_y/h = .155$

$\therefore \alpha_x = 0.44$, $P_{uxF} = 1478$ kN $\Big]$ $P_{uoF} = 3510$ kN

$\alpha_y = .66$, $P_{uyF} = 2218$ "

(20) COL. /08

$$\frac{1}{P_{uF}} = \frac{1}{1478} + \frac{1}{2218} - \frac{1}{3510} \; ; \; P_{uF} = \underline{\underline{1187}}\ kN$$

قريب جداً البداية في هذه الدور = $\underline{\underline{1180}}$ kN

Use $P_{uF} = 1180$ kN with $M_{uxF} = 173$ kNm &

$M_{uyF} = 73$ kNm

الجواب العكسي

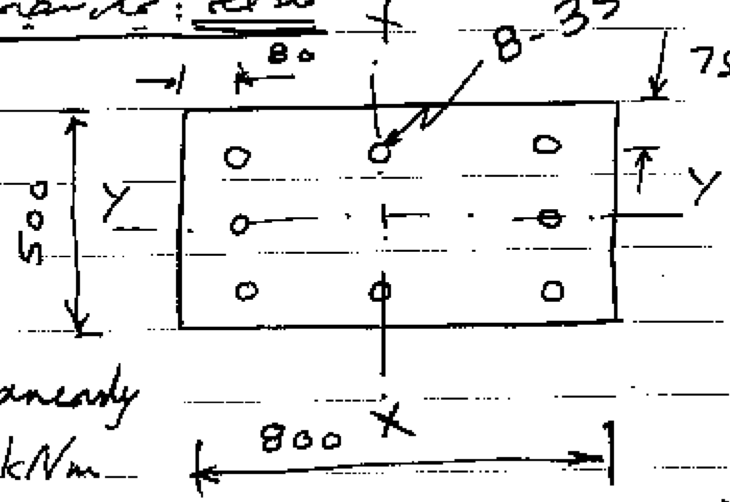
Final result $\boxed{\text{الجواب النهائي}}$ ∴ use (.65/.7) correction factor

$P_u = \underline{\underline{1096}}$ kN & $M_{ux} = \underline{\underline{151.4}}$ kNm &

$M_{uy} = 67.8$ kNm

ملاحظة: ممكن تنظيم جدول بنتائج علاقات

Ex (A) The short tied column is 500 x 800 mm. $P_u = 3990$ kN which acts simultaneously with $M_{ux} = 474$ kNm (about major axis XX), and M_{uy} about minor axis YY. Calculate M_{uy}. $f_c' = 28$ N/mm², $f_y = 400$ N/mm²



Solution: As before, it is easier to change to $\underline{\underline{F}}$ values using the factor (.7/.65)

$\therefore P_{uF} = 4297$ kN & $M_{uxF} = 510$ kNm

$$\frac{1}{P_{uF}} = \frac{1}{P_{uxF}} + \frac{1}{P_{uyF}} - \frac{1}{P_{uoF}}$$

$e_x/h = (510/4297)/.8 = .148$; For M_x: $\gamma_x = 640/800 = 0.8$; $\rho_t = 6 \times 962/(800 \times 500) = 0.01443$; $\rho_t \mu = 0.01443 \times 400/(.85 \times 28) = .243$

Goto p.495 $\underline{\underline{F}}$: $\alpha = .53$ & $\frac{\alpha e}{h} = .078$ (giving $e/h = .147$, ...

$\therefore .53 = P_{uxF} / (.028 \times 500 \times 800)$; $\therefore P_{uxF} = 5936$ kN

$P_{uoF} = .7 P_{no} = .7 [.85 f_c' (A_g - A_{st}) + f_y A_{st}]$

$= .7 [.85 \times .028 (800 \times 500 - 8 \times 962) + .4 \times 8 \times 962]$

$= 8961$ kN

$$\therefore \frac{1}{4297} = \frac{1}{5936} + \frac{1}{P_{uyF}} - \frac{1}{8961}$$

$\therefore P_{uyF} = 5687$ kN

$\alpha_y = P_{uyF} / (f_c' b h) = 5687 / (.028 \times 500 \times 800)$

$= 0.508$

$\gamma_y = 350 / 500 = 0.7$ $\therefore$ GOTO p. 494 $\underline{F}$:

$\alpha e/h = 0.084$

$\therefore e/h = .084 / .508 = .165$

$e_y = .165 \times 500 = 82.68$ mm

$\therefore M_{uyF} = 82.68 \times 4297 \times 10^{-3} = 355.3$ kN.m

$\therefore M_{uy} = (.65/.7) 355.3 = \underline{\underline{329.9 \text{ kNm}}}$

↑ الجواب المطلوب

Ex. (B) For the same column of Ex. (A), when $P_u = 3250$ kN, $M_{uy} = 464$ kNm, find M_{ux} capacity

Solution: As before, it is easier to change values, by multiplying with $.7/.65$ → $P_{uF} = 3500$ kN & $M_{uyF} = 500$ kN

$e_y/h = (500/3500)/.5 = .286$; $C_{\mu} = 0.243$ (Ex. (A))

& $\gamma_y = 0.7$; $\therefore \alpha_y = 0.37$ [p. 494 $\underline{F}$]

$\therefore P_{uyF} = .37 \times .028 \times 500 \times 800 = 4144$ kN

$$\therefore \frac{1}{3500} = \frac{1}{P_{ux}} + \frac{1}{4144} - \frac{1}{8961}$$

$\therefore P_{uxF} = 6410$ kN

$\therefore \alpha_x = 6410 / (.028 \times 500 \times 800) = .572$, $\gamma_x = .8$ & $C_{\mu} = .243$ (Ex. (A))

∴ p. 495 Ferguson : $\alpha_x e_x/h = \underline{0.07}$

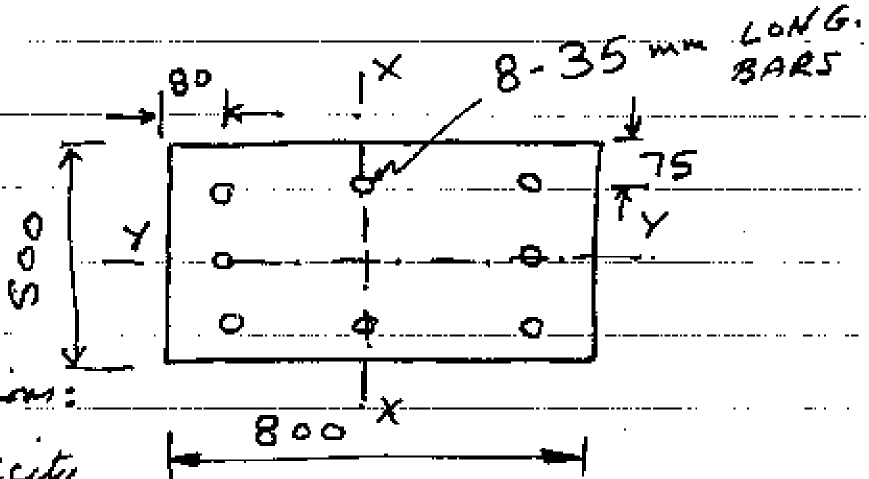
∴ $e_x/h = .07\ /.572 = .122$

$M_{uxF} = .122 \times .8 \times 3500 = 342.7\ kNm$

∴ $M_{ux} = (.65/.7)\,342.7 = 318.2\ kNm$ (الجواب المطلوب) the factored BM capacity w.r.t. major axis X-X.

مهم جداً Homework (BIAXIAL BENDING) Use the Bresler RECIPROCAL method for the following, based on :

The short tied column is 500x800 mm. $f_c' = 28\ N/mm^2$ & $f_y = 400$ "



4-HOMEWORK questions:

1. Calculate P_u capacity when $M_{ux} = 650\ kNm$ & $M_{uy} = 200\ kNm$

2. Calculate P_u capacity when $M_{ux} = 300$ & $M_{uy} = 600\ kNm$

3. With $P_u = 4640\ kN$ & $M_{ux} = 370\ kNm$ find M_{uy} capacity

4. " " 2800 " & $M_{uy} = 325$ " " M_{ux} "

ملاحظة: ان حل هذه الاسئلة مسبقا ← للطالب بان يحل الاسئلة في الامتحان بوقت قصير

SLENDER (LONG) RC COLUMNS

The basic BUCKLING (EULER) equation is:

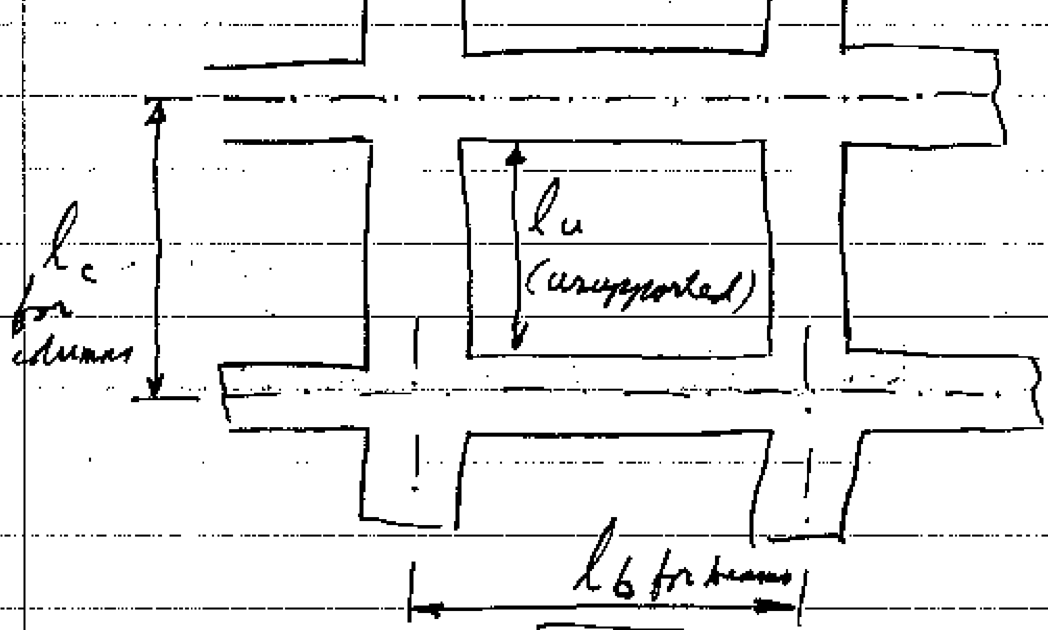
$$P_c = \frac{\pi^2 EI}{(k\,l_u)^2} \qquad (A)$$

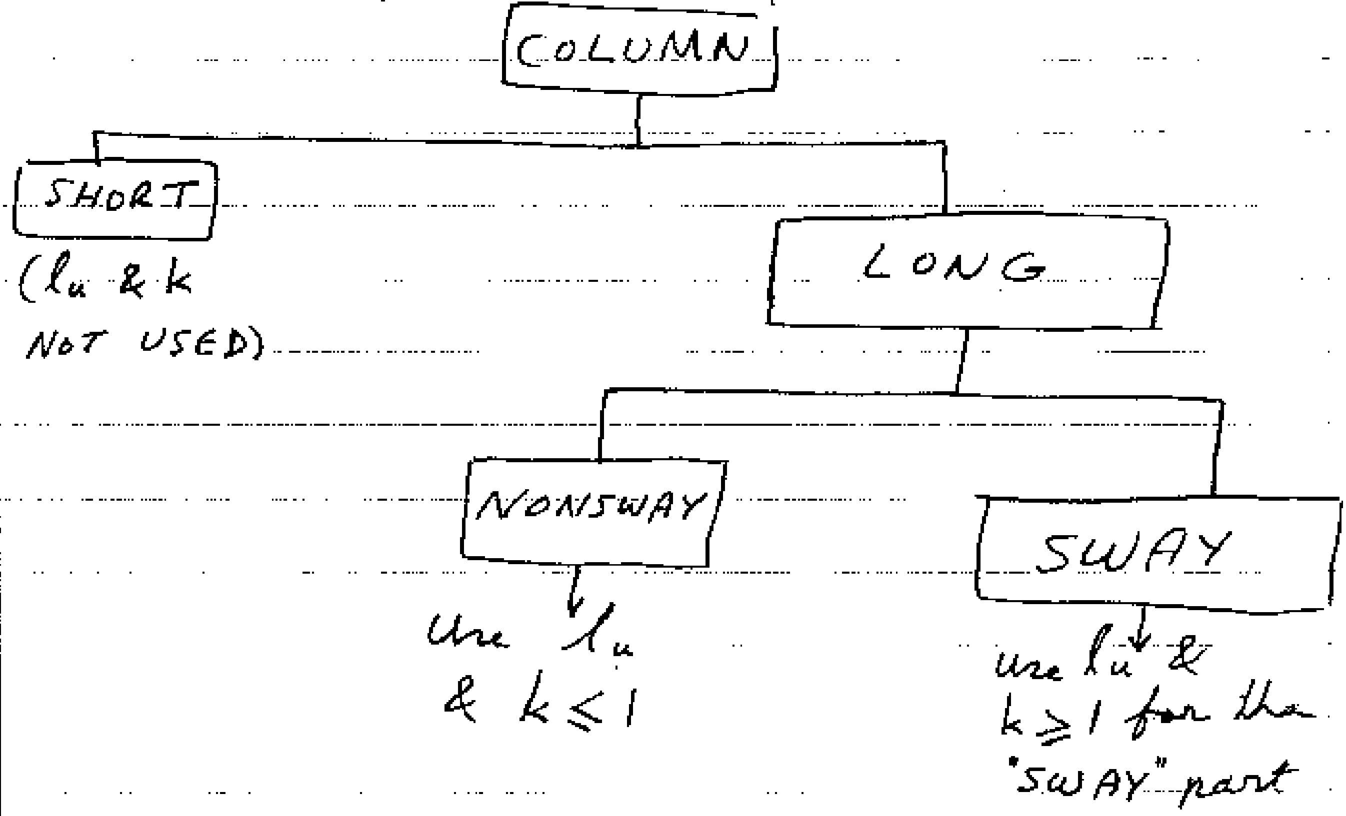
In Eq. (A): P_c = buckling load capacity

EI = value to be calculated for RC

l_u = column UNSUPPORTED length

k = factor to be calculated depending on end conditions.





LATEST ACI Column Design & Analysis

$$P_c = \pi^2 EI / (k l_u)^2 \quad \text{—— (1S)}$$

For columns with known LONGITUDINAL steel:

$$EI = (0.2 E_c I_g + E_s I_{se}) / (1 + \beta_d) \quad \text{(2S)}$$

For preliminary calculations – Long. steel unknown:

$$EI = (E_c I_g / 2.5) / (1 + \beta_d) \quad \text{—— (3S)}$$

ملاحظة: لا يجوز استعمال المعادلة (3S) عند معرفة
التسليح الطولي (Long. steel) يُركز الحساب
على المعادلة (2S)

$$E_c = 4700 \sqrt{f'_c} \quad \text{—— (4S)}$$

للاسطوانة

$$\left.\begin{aligned} E_s &= 200\,000 \ N/mm^2 \\ &= 200 \ kN/mm^2 \end{aligned}\right] \quad \text{—— (5S)}$$

k = value per "JACKSON-MORELAND" Chart —— (6S)

$$\left.\Psi = \frac{\Sigma(EI/l_c) \leftarrow \text{أعمدة}}{\Sigma(EI/l_b) \leftarrow \text{عتبات}}\right] \text{in a PLANE} \quad \text{—— (7S)}$$

To find Ψ, Eq. (8S) may be used:

For BEAMS, mom. of inertia $= 0.35 \, I_g$] (8S)
" COLUMNS " " $= 0.70 \, I_g$] note: for ▨ $I_g = bh^3/12$

l_c = ℄ to ℄ length of column] see diagram
l_b = " " " " beam] p. 23 COL./07

Eq. (9S) gives upper limit for "MOM. MAGN." method:

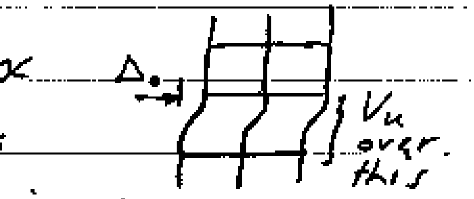
$k l_u / r \leq 100$
$r = 0.3 \times$ overall column dimension ▨ [rectangular] (9S)
$r = 0.25 \times$ diameter of ◍ [circular] column

A storey may be assumed as "NONSWAY" if :

$$Q = \frac{\sum P_u \Delta_o}{V_u l_c} \leq 0.05 \qquad (10S)$$

where Q = storey stability index

Δ_o = " lateral deflection :



V_u = factored horizontal shear in storey

With BIAXIAL moments, magnify each one separately. Then treat the column with the magnified moments as an equivalent "SHORT" column. (11S)

[1] Magnified Moments – NONSWAY Frames

Ignore slenderness [i.e. column is SHORT] if:

$$(kl_u/r) \leq 34 - 12\, M_1/M_2 \leq 40 \quad \text{— (12S)}$$

where $(M_1/M_2) \geq -0.5$ ↑ 08 — (13S)

$$M_c = \delta_{ns} M_2 \quad \text{— (14S)}$$

where M_c = Magnified moment to be used in DESIGN

δ_{ns} = NONSWAY moment magnification

M_2 = LARGER factored column end moment, always positive

M_1 = smaller column factored end moment, positive if member is bent in single curvature: M_2, M_1, negative if member is bent in double curvature M_2, M_1

$$\delta_{ns} = \frac{C_m}{1 - \dfrac{P_u}{0.75 P_c}} \geq 1 \quad \text{— (15S)}$$

where P_u = factored axial load ($\leq \phi P_n$)

P_c = critical load = $\dfrac{\pi^2 EI}{(k l_u)^2}$ (16 S)

$$C_m = (0.6 + 0.4\, M_1/M_2) \geq 0.4 \qquad (17S)$$

$$EI = (0.2\, E_c I_g + E_s I_{se})/(1+\beta_d) \qquad (18S)$$

ملاحظة: تستعمل هذه عند معرفة التسليح الطولي

$$E_s = 200 \text{ kN/mm}^2 \qquad (19S)$$

or $EI = (0.4\, E_c I_g)/(1+\beta_d)$ (20S)

ملاحظة: لا تستعمل هذه المعادلة عند معرفة التسليح الطولي

<u>Note</u>: When magnifying moment, use the greater of M_2 (اكبر عددي) or $M_{2,min}$:

$$M_{2,min} = P_u(15 + 0.03h) \qquad (21S)$$

(15 in mm)

IF $M_2 < M_{2,min}$; then:

either $C_m = 1$

or base C_m on actual M_1 & M_2 (22S)

[2] Magnified Moments — SWAY Frames

For SWAY case, NEGLECT slenderness (i.e. column is SHORT) if $\dfrac{kl_u}{r} < 22$ (23S)

SWAY frame design consists of 3 steps:

(i) Calculate $\delta_s M_s$ per Eq. (27) or Eq. (28)

(ii) Add $\delta_s M_s$ to M_{ns} at each end to obtain M_1 & M_2

(iii) If the column is still SLENDER [now $k_{ns} < 1$], and Eq. (29 S) applies, then magnify again with Eq. (14 S)

(24S)

End moments M_1 & M_2 shall be:

$M_1 = M_{1ns} + \delta_s M_{1s}$ — (25S)

$M_2 = M_{2ns} + \delta_s M_{2s}$ — (26S)

$$\delta_s M_s = \frac{M_s}{1 - Q} \geq M_s$$ — (27S)

This may be used only if $\delta_s \leq 1.5$

where δ_s = SWAY magnification factor

Alternatively (بديل):

$$\delta_s M_s = \frac{M_s}{1 - \dfrac{\Sigma P_u}{0.75 \Sigma P_c}} \geq M_s$$ (28S)

ΣP_u = Total factored loads of all storey columns

ΣP_c = " critical " " " " "

Note: For the SWAY case use $\beta_d = 0$ if the SWAY is caused by short term effects: WIND or EARTHQUAKE influence.

If a column has:

$$\frac{l_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f'_c A_g}}}$$ (29S)

Then apply Eq.(14) [$M_c = \delta_{ns} M_2$]

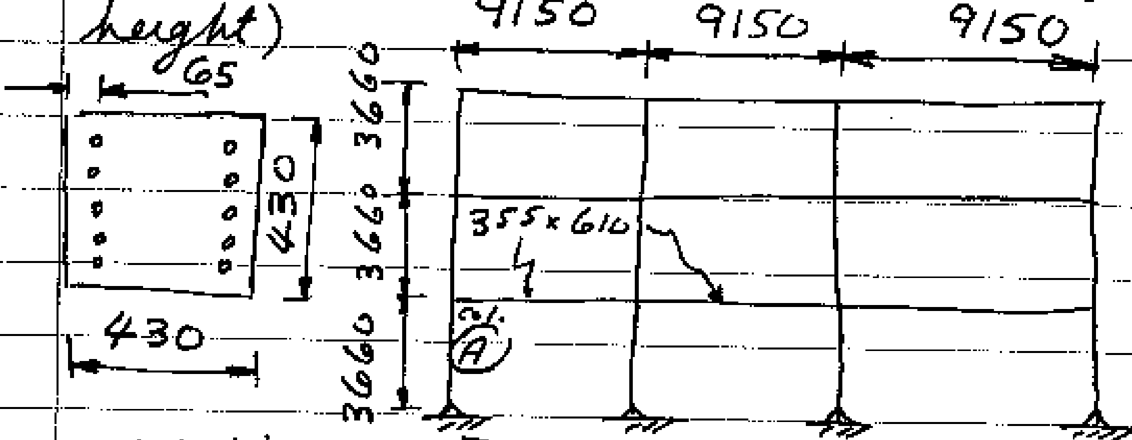
If $\delta_s > 2.5$, then STIFFEN column — (30S)

ملاحظة: (1) اذا كان العمود NONSWAY وثبت انه طويل بموجب [Eq. 12S] يستعمل δ_{ns} كما في Eq.[14S]

(2) اذا كان العمود SWAY وثبت انه طويل بموجب [Eq. 23S] يستعمل δ_s بموجب [Eq. (27S)] او [Eq. (28S)] وينتج عن ذلك M_1 [المعادلة (25S)] و M_2 [المعادلة (26S)]

(3) بعد ذلك اذا كانت NONSWAY طويلة ($k<1$) وبنفس الوقت [Eq.29] تنطبق يستعمل δ_{ns} بموجب Eq.[14S].

Ex: The frame is NONSWAY. Find if column (A) is safe. Column size is 430 x 430 mm. $f_c' = 21\ N/mm^2$, $f_y = 400\ N/mm^2$ for the 10-25 mm longitudinal bars. $P_u = 2335$ kN and $M_u = 142$ kNm, column $l_u = 3050$ (clear height)



Assume hinged

Solution

$M_u = 142\ kNm = M_2$ (GIVEN)

$\therefore e = 142 \times 10^3 / 2335 = 60.8$ mm

$e_{,min} = 15 + .03h = 15 + .03 \times 430 = 27.9$ mm

$\therefore e = 60.8$ mm governs (greater)

First step: Is column short or long:

NONSWAY column has $k \leq 1$

→ Start with $k = 1$ initially:

$k l_u = 1 \times 3050 = 3050$ mm

$k l_u / r = 3050 / (0.3 \times 430) = 23.6$

Column is SHORT if Eq. (12.5') applies:

$$\frac{k l_u}{r} < 34 - 12\frac{M_1}{M_2} \leq 40 \quad M_1 = 0 \text{ (hinged)}$$

08

$= 34$ (اكل الأكبر صحيح)

$k l_u / r = 23.6 < 34$; $\therefore$ Col. is SHORT

$\therefore$ NO MAGNIFICATION of moment M_2

$\gamma = (430 - 2 \times 65) / 430 = 0.7$ — p. 494 F

$\rho_t = 10 \times 490 / 430^2 = 0.0265 \quad \begin{matrix} \le .08 \\ \ge .01 \end{matrix} \Big] \therefore$ OK per ACI code

$\mu = f_y/(.85 f_c') = 400/(.85 \times 21) = 22.41$

$\rho_t \mu = 0.594 \; ; \; e/h = 60.8/430 = 0.141$

GOTO p. 49k Ferguson : $\alpha_F = 0.67$

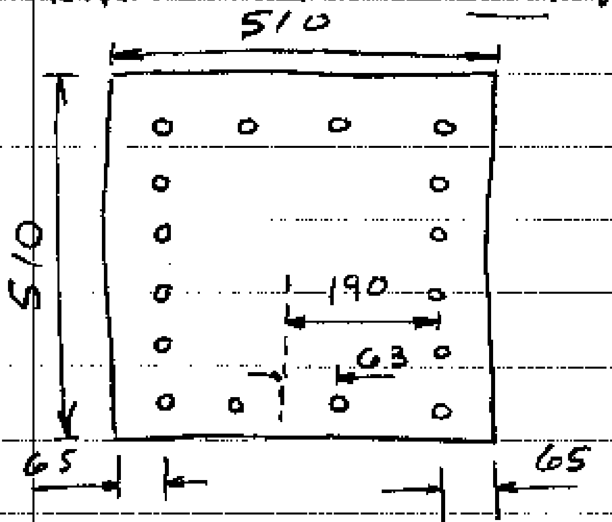
$\therefore P_{uF} = \alpha f_c' bh = .67 \times .021 \times 430^2$

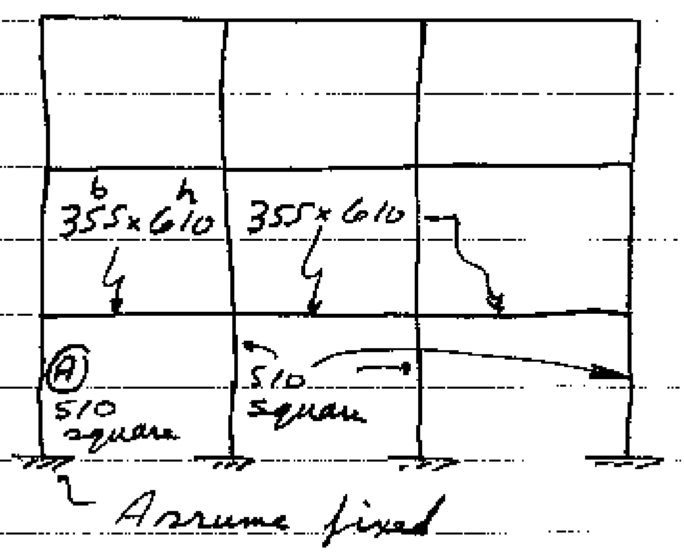
$= 2600 \text{ kN}$

العلوي $\rightarrow P_u = P_{uF} \times .65/.7 = 2600 \times .65/.7$

$= \underset{\text{مقاومة}}{2416} \text{ kN} > \underset{\text{موجود}}{2335} \text{ kN}, \therefore$ Col. is SAFE

Ex II The frame is SWAY. Find if column (A) is safe. Column size is 510 x 510 mm, $f_c' = 27.6 \text{ N/mm}^2$, $f_y = 400 \text{ N/mm}^2$ for the 16-30 mm longitudinal bars. Total load of the 1st level $\Sigma P_u = 14\,000$ kN for all 4 columns. Assume that $\beta_d = 0.4$ for all calculations (i.e including sustained sway influence), $l_u = 3050$ for col. (A), $P_u = 2335$ kN, $M_s = 360$ kNm & $M_{ns} = 95$ kNm at the TOP of column (A).





Solution :

يتم الحل للأكثر دقة للموضوع :

(30) COL. /08

(a) Slenderness of "SWAY" Part

$l_u = 3050$ mm (من معطيات السؤال)

To find $\underline{\psi}$ apply Eqs. (7S) & (8S)

Eq. (7S): $\psi = \Sigma(EI/l_c)/\Sigma(EI/l_b)$

For column use $0.7 I_g = 0.7 \times 510^4/12 = 3.946 \times 10^9$ mm^4

" beam use $.35 I_g = .35 \times 355 \times 610^3/12 = 2.350 \times 10^9$ mm^4

$$\psi_A \left(\begin{array}{c}\text{TOP OF} \\ \text{EXERIOR} \\ \text{COLUMN}\end{array}\right) = \frac{\Sigma EI/l_c}{\Sigma EI/l_b} \qquad (7S)$$

COLUMN خارجي

$$= \frac{\overset{\text{عدد}}{2} \times 3.946 \times 10^9/3660}{\text{عدد} \rightarrow 1 \times 2.350 \times 10^9/9150} = 8.40 \qquad [\text{خارجي}]$$

$$\psi_A \left(\begin{array}{c}\text{TOP OF} \\ \text{INTERIOR} \\ \text{COLUMN}\end{array}\right) = \frac{\overset{\text{عدد}}{2} \times 3.946 \times 10^9/3660}{\text{عدد} \rightarrow 2 \times 2.35 \times 10^9/9150} = 4.20 \qquad [\text{داخلي}]$$

COLUMN داخلي

Notes: (1) For l_c (& l_b) use ℄ to ℄ distance which contrasts with l_u — unsupported length

(2) At the FIXED end theoretically $\psi = 0$; however the "Structural Stability Council" recommends that for practical purposes $\underline{\psi}$ should not be < 1 (additional safety vs. $\psi = 0$). ∴ Use $\psi_B = 1$ at column bottom in all cases (داخلي، خارجي).

From the JACKSON-MORELAND Alignment chart (p. 531 FERGUSON): $k = 1.85$ (external column UNBRACED) & $k = 1.64$ (internal column UNBRACED)

For the exterior column: $\frac{kl_u}{r} = \frac{1.85 \times 3050}{.3 \times 510} = 36.9 > 22$; ∴ Column is slender SWAY part

(b) "SWAY" Magnification

Because $\underline{Q}$ is unknown, we can only use Eq. (22)

$E_c = 4700\sqrt{f_c'} = 4.7\sqrt{27.6} = 24.7$ kN/m^2

تستعمل له معرفة الحديد الطولي

$I_{se} = 12 \times 706 \times 190^2 + 4 \times 706 \times 63^2 = 3.17 \times 10^8 \text{ mm}^4$

Eq.(25): $EI = (0.2 E_c I_g + E_s I_{se}) / (1 + \beta_d)$

$I_g = 510^4/12 = 5.638 \times 10^9 \text{ mm}^4$

$\therefore EI = (0.2 \times 24.7 \times 5.638 \times 10^9 + 200 \times 3.17 \times 10^8) / (1.0$

$= 6.518 \times 10^{10} \text{ kN-mm}^2$

EXTERIOR col.

$\therefore P_c = \pi^2 \times 6.518 \times 10^{10} / (1.85 \times 3050)^2 = 20206$ kN

For INTERIOR col.: $P_c = \pi^2 \times 6.518 \times 10^{10} / (1.64 \times 3050)^2 = \underline{25711}$ kN ← داخلية

$\therefore$ For the 4 - 1st level columns: $\Sigma P_c = 2 \times 25711 + 2 \times 20206 = \underline{91834}$ kN storey EULER capacity

Eq.(28): $\delta_s = \dfrac{1}{1 - \dfrac{\Sigma P_u}{.75 \Sigma P_c}} = \dfrac{1}{1 - \dfrac{14000}{.75 \times 91834}} = 1.255$

ملاحظة: الرقم 1.255 يعني بانه لو عرفت Q لكان اكل

حل يكثر باستعمال ≤ 1.5 Eq. 27 : $\delta_s = \dfrac{1}{1 - Q}$

Eq.(26): $M_2 = M_{2ns} + \delta_s M_{2s}$

$= 95 + 1.255 \times 360 = \underline{546.8 \text{ kN/m}}$

(c) "NONSWAY" Magnification

With $\left.\begin{matrix} \psi_A = 8.4 \\ \psi_B = 1.0 \end{matrix}\right\}$ $k_{ns} = 0.85$, From Jackson-Moreland (p. 531 FERG.)

For a "NONSWAY" case, apply Eq.(12.5):

Column is SHORT if $\dfrac{k l_u}{r} \le 34 - 12\, M_1/M_2 \le 40$

Actual $k l_u / r = .85 \times 3050 / (.3 \times 510)$

$= 16.9$

$\therefore$ "NONSWAY" part is "SHORT", since $16.9 <$ the minimum possible value of Eq.(12.5), which is 22.

<u>Note</u>: Even if the column is <u>SLENDER</u> for the <u>NONSWAY</u> part, magnification with δ_{ns} may not be applied unless Eq.(29.5') requires that:

Eq.(29.5'): ONLY magnify 2nd time (مرة ثانية) if:

$$\frac{l_u}{r} > \frac{35}{\sqrt{\dfrac{P_u}{f_c' A_g}}} = \frac{35}{\sqrt{\dfrac{2335}{.0276 \times 510^2}}} = 61.4$$

$$l_u/r = \frac{3050}{.3 \times 510} = 19.9 < 61.4$$

∴ لا يكبر العزم مرة ثانية حتى لو كانت طويلة (SLENDER) في حالة NONSWAY.

Taking only the OUTSIDE 12-30mm bars (on the <u>SAFE</u> side): $A_{st} = 706 \times 12 = 8472\ mm^2$

$\rho_t = 8472/510^2 = 0.0326$

$\mu = f_y/(.85 f_c') = 17.05$; ∴ $\rho_t \mu = 0.555$

$e = \dfrac{M_c}{P_u} = \dfrac{546.8 \times 10^3}{2335} = 234.6\ mm$ $\quad \gamma = 380/510 = .75$

$e/h = 234.6/510 = .459$

∴ Interpolating between pp 494 & 495 FERGUSON:

$\alpha = 0.37$

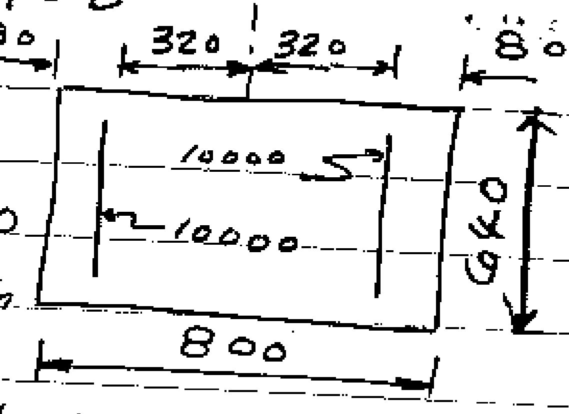
∴ $P_{uF} = \alpha f_c' bh = .37 \times .0276 \times 510^2 = 2656\ kN$

<u>CORRECTING</u>: $P_u = (.65/.7) P_{uF}$

$= (.65/.7) 2656$

$= 2466\ kN > 2335$ OK

(القاعدة المتوفرة) موجود



Ex. III The tied column has $l_u = 11$ m ; $\psi_{TOP} = \psi_{BOTT} = 0.9$; $P_u = 6400$ kN ; Top & Bottom moments are equal causing SINGLE curvature : $M_{ns} = 200$ kNm & $M_s = 550$ kNm ; $f'_c = 20$ N/mm^2 ; $f_y = 400$ N/mm^2 ; $\beta_d = 0.4$ for the NONSWAY part. Assume that $\beta_d = 0$ for the SWAY part which is caused by short duration loading. $A_{st} = 20000$ mm^2. Assuming that all STOREY columns have the same P_u & P_c values, find if the column is SAFE per latest ACI 318 M code.

Solution:

$\psi_{TOP} = \psi_{BOTT}$; $\therefore k_s = 1.26$ & $k_{ns} = 0.76$ — Jackson-Moreland

$kl_u/r = 1.26 \times 11/(.3 \times .8) = 57.8 > 22$: Slender SWAY

$E_c = 4.7\sqrt{20} = 21.02$ kN/mm^2 ; $I_g = 640 \times 800^3/12 = 2.73 \times 10^{10}$ mm^4

$EI = (.2 E_c I_g + E_s I_{se})/(1 + \beta_d)$

$$\therefore (EI)_{SWAY} = \frac{.2 \times 21.02 \times 2.73 \times 10^{10} + 20000 \times 320^2 \times 200}{1 + 0} = 5.244 \times 10^{11} \text{ kN-mm}^2$$ — P_c(SWAY)

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2 \times 5.244 \times 10^{11}/(1.26 \times 11000)^2$

$= 26940$ kN the CRITICAL (EULER) load for the "SWAY" part

Eq.(28) : $\delta_s = 1/[1 - \Sigma P_u/.75 \Sigma P_c]$

$$\delta_s = \frac{1}{1 - \dfrac{6400}{.75 \times 26940}} = 1.464$$

Eq. (26S): $M_2 = M_{2ns} + \delta_s M_s$ single curvature

$M_2 = 200 + 1.464 \times 550 = 1005.2$ kNm $= M_1$

<u>"NONSWAY" Part</u>

$(kl_u/r)_{NONSWAY} = .76 \times 11 / (.3 \times .8) = 34.83$

Eq. (12S) Upper limit for <u>SHORT</u> column:

$kl_u/r \le 34 - 12\, M_1/M_2 = 34 - 12 = 22$

$\therefore$ actual $(kl_u/r)_{NONSWAY} = 34.83 > 22$.

$\therefore$ <u>NONSWAY</u> column is slender

Eq. (29S): $35/\sqrt{P_u/(f_c' A_g)} = \dfrac{35}{\sqrt{6400/(.02 \times 800 \times 640)}}$

$= 44.3$

$l_u/r = 11/(.3 \times .8) = 45.83 > 44.3$

$\therefore$ Apply Eq. (14S) to magnify M_2:

Eq. (20S): $C_m = 0.6 + .4\, M_1/M_2$ $\quad M_1 = M_2$

$\therefore C_m = .6 + .4 \times 1 = 1$

$EI = 5.244 \times 10^{11} / (1 + 0.4) = 3.746 \times 10^{11}$ kN.mm²

$P_c = \pi^2 EI/(kl_u)^2 = 3.746 \times 10^{11} \times \pi^2 / (.76 \times 11000)^2$

$= 52900$ kN CRITICAL (EULER) load

$\delta_{ns} = \dfrac{C_m}{1 - P_u/.75 P_c} = \dfrac{1}{1 - 6400/(.75 \times 52900)} = 1.192$

Eq. (14S): $M_c = \delta_{ns} M_2 = 1.192 \times 1005.2 = \underline{1198}$ kNm

$\rho_t \mu = \dfrac{20000}{800 \times 640} \times \dfrac{400}{.85 \times 20} = 0.919$; $\gamma = .8$ بعد تقريب مرتبة

GOTO p. 495 FERGUSON with $e/h = \dfrac{1198 \times 10^3}{800} \times \dfrac{1}{6400} = .234$

$\alpha_{FERG} = 0.69$; $\therefore P_{uf} = .69 \times .02 \times 800 \times 640 = \underline{7066}$ kN

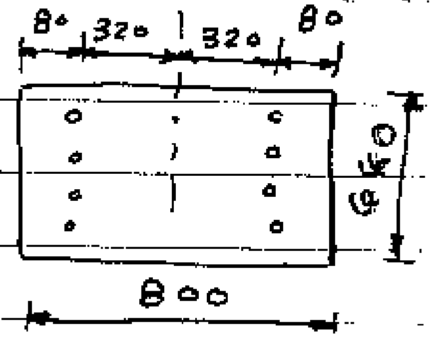
$P_u = \dfrac{.65}{.7} P_{uf} = \dfrac{.65}{.7} \times 7066$

مقاوم

$= 6561 > 6400$ O.K.

مقاوم موجود

$\therefore$ <u>Column is SAFE</u>



EX. IV The tied col. has $l_u = 4.2$ m

$\psi_{TOP} = \psi_{BOTT} = 2.75$; $P_u = 7000$ kN ;

Factored moments at the top are

$M_{ns} = 300$ kNm & $M_s = 370$ kNm ;

Factored moments at the bottom causing SINGLE curvature are $M_{ns} = 300$ kNm & $M_s = 370$ kNm.

$f_c' = 28$ N/mm² ; $f_y = 400$ N/mm² for the 8-30mm longitudinal bars. Storey computations indicate that the storey stability index $Q = 0.09$. Is the column safe?

Solution: Jackson-Moreland: $k_s = 1.8$ & $k_{ns} = 0.88$ $\left(\frac{k_s = 4.8}{2.75}\right)$

For the SWAY part $kl_u/r = 1.8 \times 4.2/(.3 \times .8) = 31.5 > 22$

∴ "SWAY" column is SLENDER. الحد الاعلى!

Eq.(27.5): $\delta_s = \frac{1}{1-Q} \geq 1$; $\delta_s = \frac{1}{1-.09} = \underline{\underline{1.099}} < 1.5$, OK

$M_2 = M_{2ns} + \delta_s M_{2s} = 300 + 1.099 \times 370 = 706.6$ kNm TOP

$M_1 = M_{1ns} + \delta_s M_{1s} = 300 + 1.099 \times 370 = 706.6$ " BOTT

NONSWAY part : $kl_u/r = .88 \times 4.2/(.3 \times .8) = 15.4$

upper limit for "SHORT" column : $34 - 12 \times 706.6/706.6 = 22$

$(kl_u/r)_{\text{الحرج}} = 15.4 < 22$ ∴ NONSWAY part is short

$\rho_t \mu = [5655/(800 \times 640)] \times 400/(.85 \times 28) = \underline{\underline{0.186}}$

$\gamma = 0.8$; $e/h = (706.6/7000)/.8 = \underline{0.126}$

GOTO P.495 Ferguson : $\alpha = 0.5$

∴ $P_{uF} = .53 \times .028 \times 800 \times 640 = 7598$ kN

Now correct $P_u = (.65/.7)7598 = 7055$ kN

7055 kN > 7000 kN , OK

مقاومة   مطلوب

∴ Column is SAFE

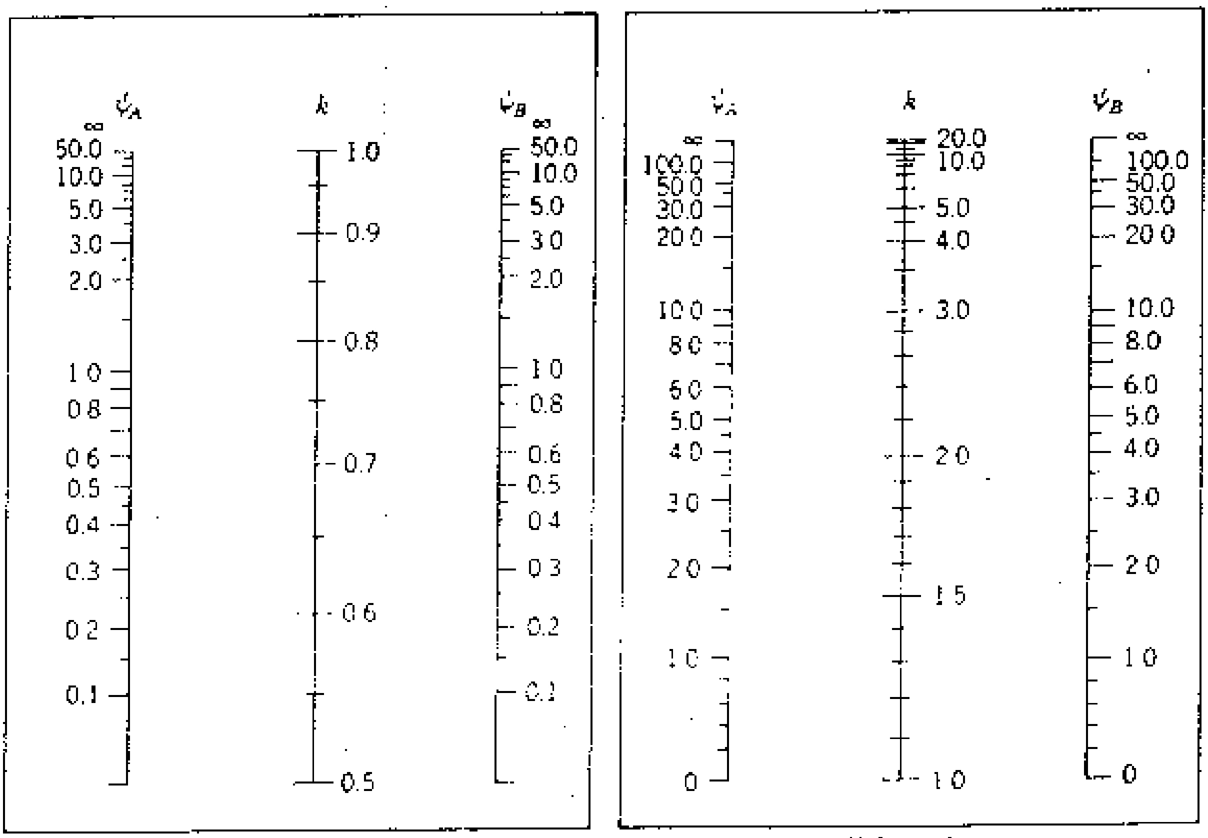
36 col 08

moments might be more appropriate, since creep is a function of sustained loading.) The author feels that this β_d correction may be unimportant in the assessment of an EI value that probably involves a typical error of at least 30%. Fortunately, the effect of β_d (and even of EI) is far removed from the end result of a design. Experience may show that it is only on very unusual long columns that β_d is really of significance.

19.8 EFFECTIVE COLUMN LENGTH, $k\ell_u$

In simple mechanics the concept of effective column length is well established; a fixed ended column has an effective length of half its overall height between fixed ends. If it were only a matter of the frame reaction to a statically determined moment applied directly as a load on the column, the effective length would be a relatively simple matter of frame or joint stiffness.

Jackson and Moreland[2] solved the $k\ell_u$ problem in terms of relative



Braced
(a)

Unbraced
(b)

FIGURE 19.6 Effective length factors. (From ACI Code Commentary.) (*a*) Braced frames. (*b*) Unbraced frames. ψ = ratio of $\Sigma EI/\ell_c$ of compression members to $\Sigma EI/\ell$ of flexural members in a plane at one end of a compression member; k = effective length factor.