Practical Design to Eurocode 2

The webinar will start at 12.30

Course Outline

Lecture	Date	Speaker	Title
1	21 Sep	Charles Goodchild	Introduction, Background and Codes
2	28 Sep	Charles Goodchild	EC2 Background, Materials, Cover and effective spans
3	5 Oct	Paul Gregory	Bending and Shear in Beams
4	12 Oct	Charles Goodchild	Analysis
5	19 Oct	Paul Gregory	Slabs and Flat Slabs
6	26 Oct	Charles Goodchild	Deflection and Crack Control
7	2 Nov	Paul Gregory	Detailing
8	9 Nov	Jenny Burridge	Columns
9	16 Nov	Jenny Burridge	Fire
10	23 Nov	Jenny Burridge	Foundations

Columns

Lecture 8

9th November 2016

Model Answers

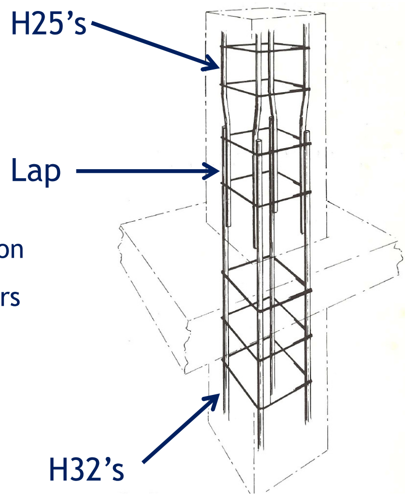
Lecture 7 Exercise:

Lap length for column longitudinal bars

Column lap length exercise

Design information

- C40/50 concrete
- 400 mm square column
- 45mm nominal cover to main bars
- Longitudinal bars are in compression
- Maximum ultimate stress in the bars is 390 MPa



Exercise:

Calculate the minimum lap length using EC2 equation 8.10:

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_{b,rqd} \geq l_{0,min}$$

Column lap length exercise

Procedure

- Determine the ultimate bond stress, f_{bd} EC2 Equ. 8.2
- Determine the basic anchorage length, $l_{b,req}$ EC2 Equ. 8.3
- Determine the design anchorage length, l_{bd} EC2 Equ. 8.4
- Determine the lap length, l_0 = anchorage length x α_6

Solution - Column lap length

Determine the ultimate bond stress, f_{bd}

$f_{bd} = 2.25\ \eta_1\ \eta_2\ f_{ctd}$ EC2 Equ. 8.2

$\eta_1 = 1.0$ 'Good' bond conditions

$\eta_2 = 1.0$ bar size ≤ 32

$f_{ctd} = \alpha_{ct}\ f_{ctk,0,05}/\gamma_c$ EC2 cl 3.1.6(2), Equ 3.16

$\alpha_{ct} = 1.0$ $\gamma_c = 1.5$

$f_{ctk,0,05} = 0.7 \times 0.3\ f_{ck}^{2/3}$ EC2 Table 3.1

$= 0.21 \times 40^{2/3}$

$= 2.456$ MPa

$f_{ctd} = \alpha_{ct}\ f_{ctk,0,05}/\gamma_c = 2.456/1.5 = 1.637$

$f_{bd} = 2.25 \times 1.637 = 3.684$ MPa

Solution - Column lap length

Determine the basic anchorage length, $l_{b,req}$

$l_{b,req} = (Ø/4)\ (\sigma_{sd}/f_{bd})$ EC2 Equ 8.3

Max ultimate stress in the bar, $\sigma_{sd} = 390$ MPa.

$l_{b,req} = (Ø/4)\ (390/3.684)$

$= 26.47\ Ø$

For concrete class C40/50

Solution - Column lap length

Determine the design anchorage length, l_{bd}

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,req} \geq l_{b,min}$ Equ. 8.4

$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ (26.47Ø)$ For concrete class C40/50

For bars in compression $\alpha_1 = \alpha_2 = \alpha_3 = \alpha_4 = \alpha_5 = 1.0$

Hence $l_{bd} = 26.47Ø$

Solution - Column lap length

Determine the lap length, l_0 = anchorage length x α_6

All the bars are being lapped at the same section, $\alpha_6 = 1.5$

A lap length is based on the smallest bar in the lap, 25mm

Hence,

$l_0 = l_{bd} \times \alpha_6$

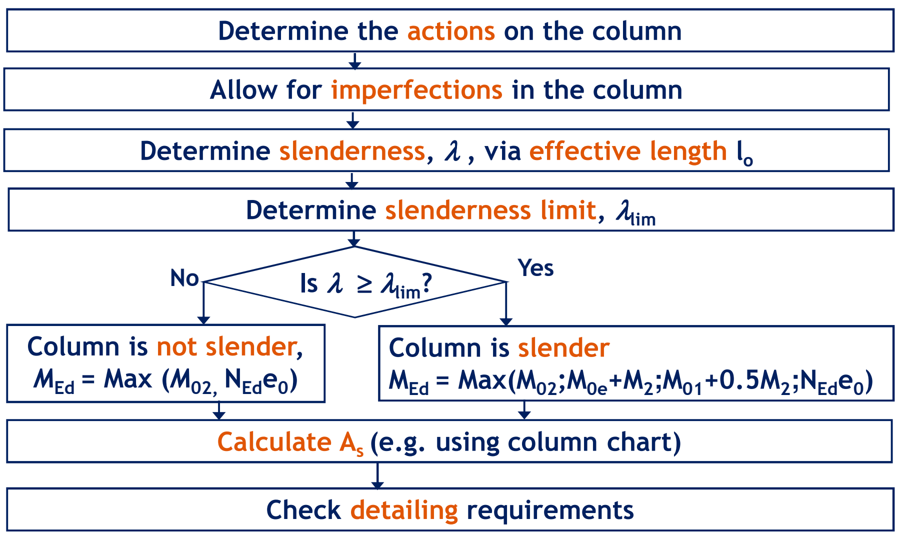
$l_0 = 26.47\ Ø \times 1.5$

$l_0 = 39.71\ Ø = 39.71 \times 25$

$l_0 = 993$ mm

Columns

The Column Design Process



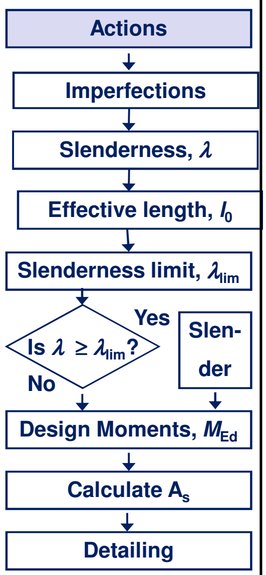
Actions

Actions on the columns are determined using one of the analysis methods we looked at for flexural design.

From the analysis obtain the following data:

- Ultimate axial load, N_{Ed}
- Ultimate moment at the top of the column, M_{top}
- Ultimate moment at the bottom of the column, M_{bottom}

Allow for imperfections . . .



Geometric Imperfections:

Cl. 5.2 5.5

Deviations in cross-section dimensions are normally taken into account in the material factors and should not be included in structural analysis

Imperfections need not be considered for SLS

But out-of-plumb needs to be considered and is represented by an inclination, θ_i

$$\theta_i = \theta_0 \, \alpha_h \, \alpha_m$$

where $\theta_0 = 1/200$

$\alpha_h = 2/\sqrt{l}$; $2/3 \leq \alpha_h \leq 1$

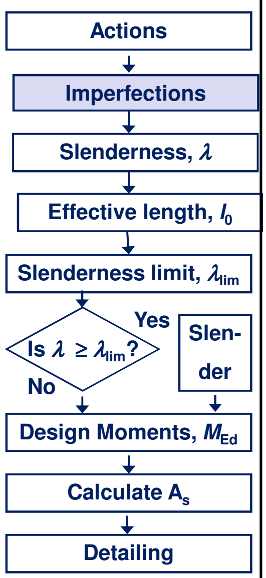
$\alpha_m = \sqrt{(0.5(1+1/m))}$

where l = the length or height (m) (see 5.2(6))

m = no. of vert. members

For isolated columns in braced systems, α_m and α_h may be taken as 1.0

i.e. $\theta_i = \theta_0 = 1/200$



Geometric Imperfections

Cl. 5.2 (7) & (9) 5.6.2.1

For isolated members at ULS, the effect of imperfections may be taken into account in two ways:

a) as an eccentricity, $e_i = \theta_i\, l_0/2$

So for isolated columns in a braced system, $e_i = l_0/400$ may be used.

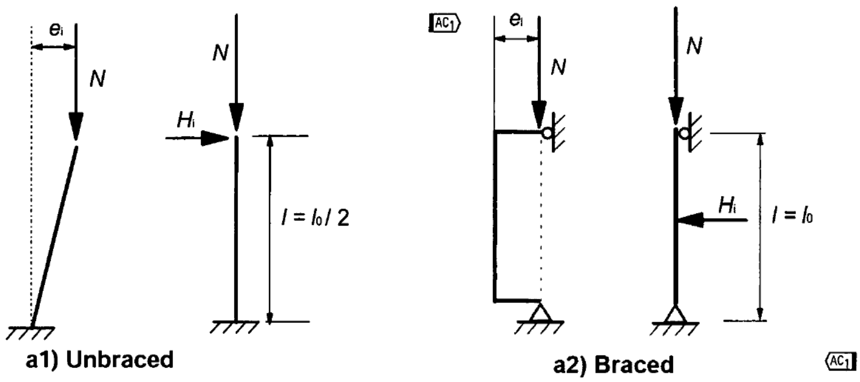
b) as a transverse force, H_i

$H_i = \theta_i N$ for un-braced members

$H_i = 2\theta_i N$ for braced members = N/100

Examples of Isolated Members

Figure 5.1a 5.5.2



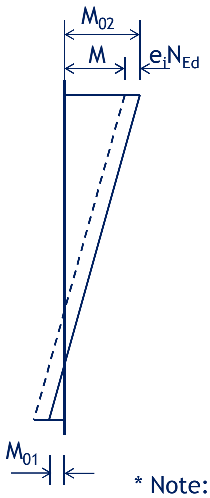
Geometric Imperfections:

Cl 6.1(4)

Subject to a minimum eccentricity:

$e_0 = h/30$ but ≥ 20 mm

Design Moments - Stocky Columns



M_{01} = [Min{|M_{top}|, |M_{bottom}|} - $e_i N_{Ed}$] *

M_{02} = [Max{|M_{top}|, |M_{bottom}|} + $e_i N_{Ed}$] *

$e_i = l_o/400$

N_{Ed} = design load in the column

For a stocky column,

Design moment

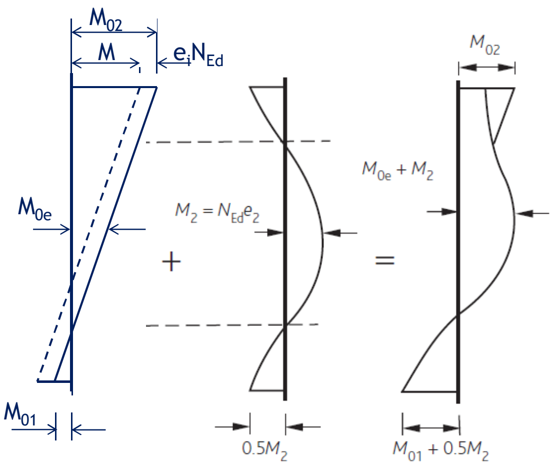
$M_{Ed} = \text{Max}\{M_{02}, e_0 N_{Ed}\}$

$e_0 = \text{Max}\{h/30, 20\text{mm}\}$

* Note: M_{01} and M_{02} return to their original signs after this calculation has been carried out

Moments in Slender Columns

Cl. 5.8.8.2 Fig 5.10



a) First order moments for 'stocky' columns b) Additional second order moments for 'slender' columns c) Total moment diagram for 'slender' columns

Design Moment,

$$M_{Ed} = \text{Max} \{M_{02};\ M_{0e} + M_2;\ M_{01} + 0.5M_2;\ N_{Ed}e_0\}$$

Is the Column Slender?

Cl. 5.8.2, 5.8.3.1 5.6.1

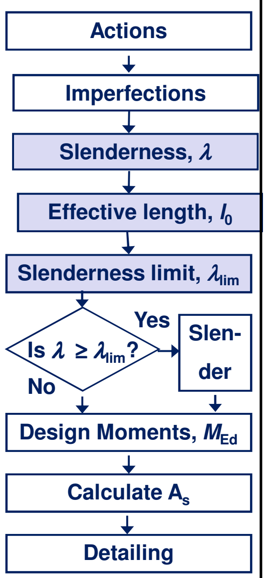
2nd order effects

Second order effects may be ignored if they are less than 10% of the corresponding first order effects

Second order effects may be ignored if the slenderness, λ is less than λ_{lim} where

$$\lambda_{lim} = 20\, A\, B\, C / \sqrt{n}$$

With biaxial bending the slenderness should be checked separately for each direction and only need be considered in the directions where λ_{lim} is exceeded



Slenderness

Slenderness, $\lambda = l_0/i$

l_0 = effective length = F.l

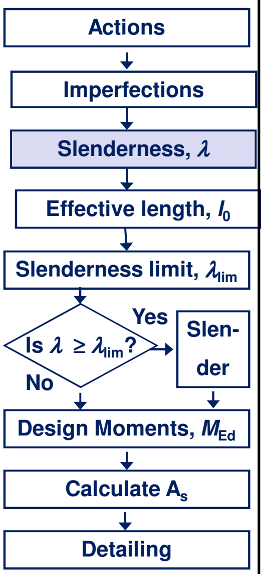
(l = actual length)

i = radius of gyration = $\sqrt{(I/A)}$

hence

for a rectangular section $\lambda = 3.46\ l_0 / h$

for a circular section $\lambda = 4\ l_0 / h$

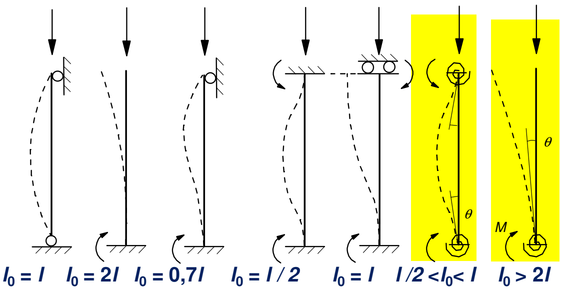


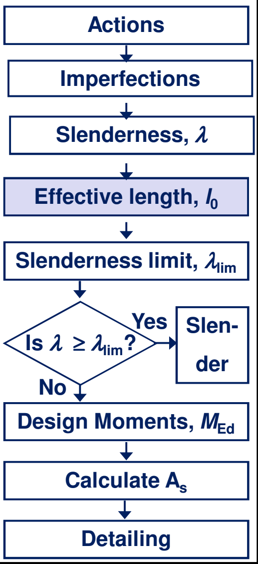
Effective length

Figure 5.7, 5.8.3.2

Figure 5.6, 5.6.1.2

Effective length, $l_0 = Fl$





Braced members:

$$F = 0{,}5\sqrt{\left(1+\frac{k_1}{0{,}45+k_1}\right)\cdot\left(1+\frac{k_2}{0{,}45+k_2}\right)}$$

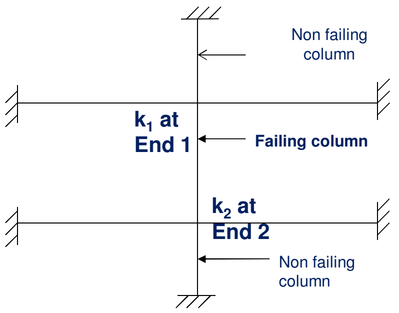
Unbraced members:

$$F = \max\left\{\sqrt{1+10\cdot\frac{k_1\cdot k_2}{k_1+k_2}}\ ;\ \left(1+\frac{k_1}{1+k_1}\right)\cdot\left(1+\frac{k_2}{1+k_2}\right)\right\}$$

where $k = (\theta / M)\cdot(EI / l)$

Effective length & *k* factors

PD 6687 Cl.2.10



From PD 6687

The contribution of 'non failing' columns to the joint stiffness may be ignored

For beams θ / M may be taken as $l/2EI$ (allowing for cracking in the beams)

$k_1 = [EI / l]_{col} \,/\, [\Sigma 2EI / l]_{beams1}$ $\qquad k_2 = [EI / l]_{col} \,/\, [\Sigma 2EI / l]_{beams2}$

Assuming that the beams are symmetrical about the column and their sizes are the same in the two storeys shown, then:

$k_1 = k_2 = [EI / l]_{col} \,/\, [2 \times 2EI / l]_{beams} \geq 0.1$

Effective length & Slenderness

Cl. 5.8.3.1 5.6.1.4

Slenderness $\lambda = l_0/i$

where

$l_0 = Fl$

k = relative stiffness

$$k = \frac{\dfrac{EI_c}{l_c}}{\Sigma \dfrac{2EI_b}{l_b}} \geq 0.1$$

Effective length factor, F, for braced columns

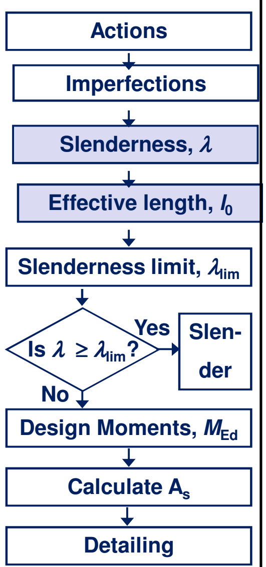
k2	k1										
	0.10	0.20	0.30	0.40	0.50	0.70	1.00	2.00	5.00	9.00	Pinned
0.10	0.59	0.62	0.64	0.66	0.67	0.69	0.71	0.73	0.75	0.76	0.77
0.20	0.62	0.65	0.68	0.69	0.71	0.73	0.74	0.77	0.79	0.80	0.81
0.30	0.64	0.68	0.70	0.72	0.73	0.75	0.77	0.80	0.82	0.83	0.84
0.40	0.66	0.69	0.72	0.74	0.75	0.77	0.79	0.82	0.84	0.85	0.86
0.50	0.67	0.71	0.73	0.75	0.76	0.78	0.80	0.83	0.86	0.86	0.87
0.70	0.69	0.73	0.75	0.77	0.78	0.80	0.82	0.85	0.88	0.89	0.90
1.00	0.71	0.74	0.77	0.79	0.80	0.82	0.84	0.88	0.90	0.91	0.92
2.00	0.73	0.77	0.80	0.82	0.83	0.85	0.88	0.91	0.93	0.94	0.95
5.00	0.75	0.79	0.82	0.84	0.86	0.88	0.90	0.93	0.96	0.97	0.98
9.00	0.76	0.80	0.83	0.85	0.86	0.89	0.91	0.94	0.97	0.98	0.99
Pinned	0.77	0.81	0.84	0.86	0.87	0.90	0.92	0.95	0.98	0.99	1.00

F

$i = \sqrt{(I/A)}$

for a rectangular section $\lambda = 3.46\, l_0 / h$

for a circular section $\lambda = 4\, l_0 / h$



Slenderness limit

Allowable Slenderness

$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n}$

where:

$A = 1 / (1 + 0.2\varphi_{ef})$

φ_{ef} is the effective creep ratio;

(if φ_{ef} is not known, A = 0,7 may be used)

$B = \sqrt{(1 + 2\omega)}$ $\omega = A_s f_{yd} / (A_c f_{cd})$

(if ω is not known, B = 1,1 may be used)

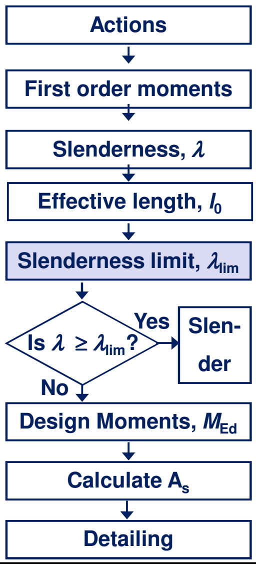
$C = 1.7 - r_m$

$r_m = M_{01}/M_{02}$

M_{01}, M_{02} are first order end moments, $|M_{02}| \geq |M_{01}|$

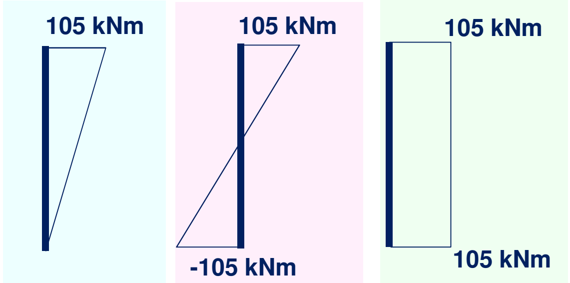
(if r_m is not known, C = 0.7 may be used)

$n = N_{Ed} / (A_c f_{cd})$



Factor *C*

$\lambda_{lim} = 20 \cdot A \cdot B \cdot C / \sqrt{n}$



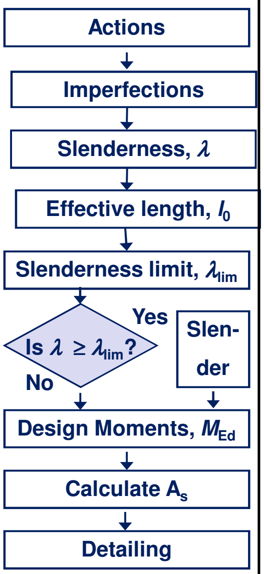
$r_m = M_{01}/M_{02}$	$r_m = M_{01}/M_{02}$	$r_m = M_{01}/M_{02}$
= 0 / 105	= 105 / -105	= 105 / 105
= 0	= -1	= 1
C = 1.7 – 0	C = 1.7 + 1	C = 1.7 – 1
= 1.7	= 2.7	= 0.7

26

Is $\lambda \geq \lambda_{lim}$?

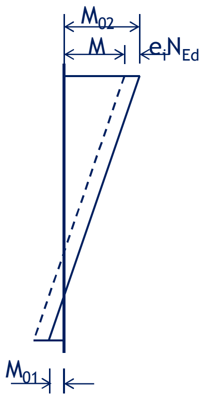
Is

$$l_0/i = \lambda \geq \lambda_{lim} = 20 \cdot A \cdot B \cdot C/\sqrt{n} \ ?$$



No, $\lambda < \lambda_{lim}$

Design Moments - Stocky Columns
(aka 1st order moments plus effects of imperfections) As before !



$M_{01} = \text{Min}\{|M_{top}|, |M_{bottom}|\} - e_i N_{Ed}$

$M_{02} = \text{Max}\{|M_{top}|, |M_{bottom}|\} + e_i N_{Ed}$

$e_i = l_o/400$

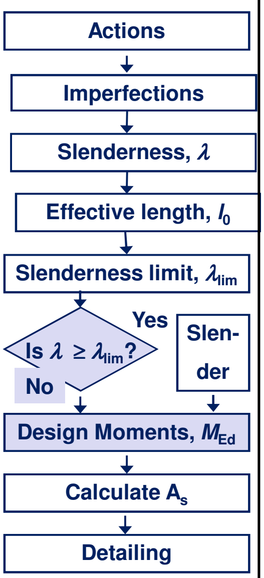
N_{Ed} = design load in the column

For a stocky column,

Design moment

$M_{Ed} = \text{Max}\{M_{02}, e_0 N_{Ed}\}$

$e_0 = \text{Max}\{h/30, 20\text{mm}\}$



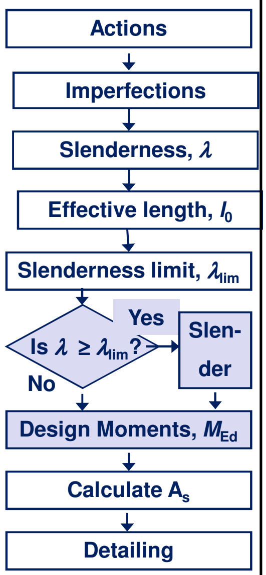
Yes, $\lambda \geq \lambda_{lim}$ 2nd order effects

Cl 5.8.5, cl 5.8.8 5.6.2.1

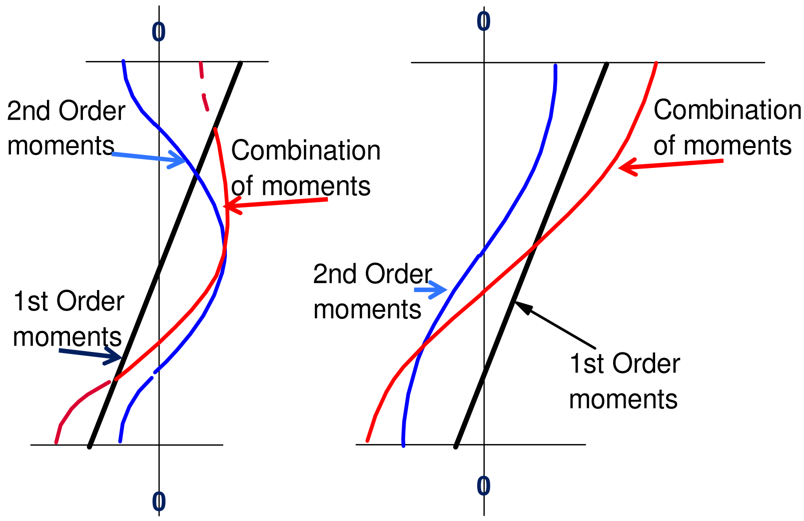
The methods of analysis include a general method, for 2nd order effects based on non-linear second order analysis and the following two simplified methods:

- Method based on nominal stiffness
- Method based on nominal curvature

  This method is primarily suitable for isolated members with constant normal force and defined effective length. The method gives a nominal second order moment based on a deflection, which in turn is based on the effective length and an estimated maximum curvature. (preferred in UK)



Moments in Slender Columns



Typical braced column Typical unbraced column

Nominal Curvature Method

Cl. 5.8.8.2 5.6.2.2

$M_{Ed} = M_{0Ed} + M_2$

M_{0Ed} is the 1st order moment including the effect of imperfections
M_2 is the nominal 2nd order moment.

Differing 1st order end moments M_{01} and M_{02} may be replaced by an equivalent 1st order end moment M_{0e}:

$$M_{0e} = (0.6\,M_{02} + 0.4\,M_{01}) \geq 0.4M_{02}$$

HOWEVER, this is only the mid-height moment the two end moments should be considered too. PD 6687 advises for braced structures:

$$M_{Ed} = \text{Max}\,\{M_{02},\ M_{0e}+M_2;\ M_{01}+0.5M_2\} \geq e_0N_{Ed}$$

where $M_{02} = \text{Max}\{|M_{top}|;|M_{bot}|\} + e_iN_{Ed}$

$M_{01} = \text{Min}\,\{|M_{top}|;|M_{bot}|\} + e_iN_{Ed}$

M_{top} & M_{bot} are frame analysis 1st order end moments

Effectively: $M_{Ed} = \text{Max}\,\{M_{02},\ M_{0e}+M_2;\ M_{01}+0.5M_2;\ e_0N_{Ed}\}$

Second order moment

Cl. 5.8.8 5.6.2.2

$M_2 = N_{Ed}\,e_2$

$e_2 = (1/r)l_0^2/c$

$1/r = K_rK_\varphi/r_0$

$K_r = (n_u - n)/(n_u - n_{bal}) \leq 1$ (or see Column charts)

$n = N_{Ed}/(A_cf_{cd})$

$n_u = 1 + \omega$

$\omega = A_sf_{yd}/(A_cf_{cd})$

$n_{bal} = 0.4$

$K_\varphi = 1 + \beta\varphi_{ef} \geq 1$

$\beta = 0.35 + f_{ck}/200 - \lambda/150$

$1/r_0 = \varepsilon_{yd}/(0.45d)$

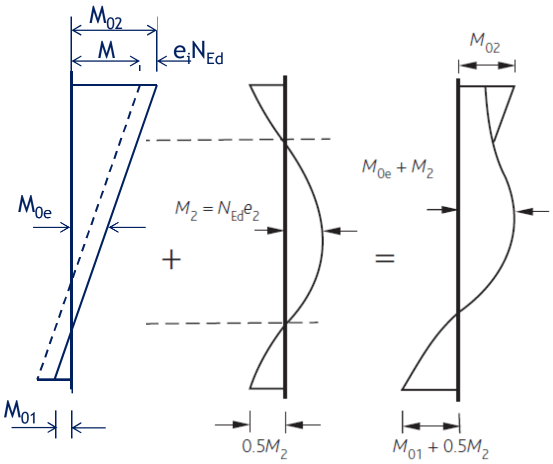
$\varepsilon_{yd} = f_{yd}/E_s$

c = 10 (for a constant cross section)

Moments in Slender Columns

Cl. 5.8.8.2

Fig 5.10



a) First order moments for 'stocky' columns

b) Additional second order moments for 'slender' columns

c) Total moment diagram for 'slender' columns

Design Moment,

$$M_{Ed} = \text{Max}\ \{M_{02};\ M_{0e} + M_2;\ M_{01} + 0.5M_2;\ N_{Ed}e_0\}$$

Can be used generically

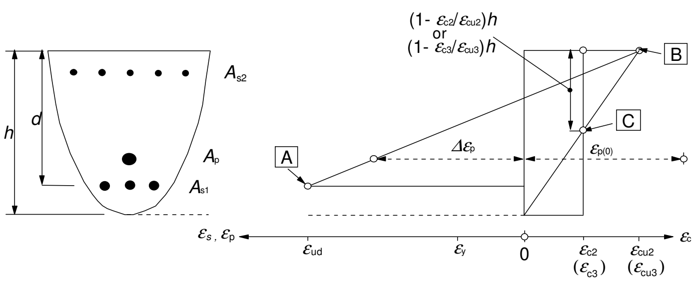
Section Design:

Bending with/without Axial Load

EC2 Figure 6.1

Concise Figure 6.3

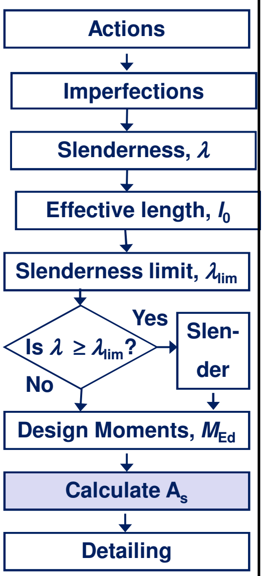
Basis of design:



A reinforcing steel tension strain limit

B concrete compression strain limit

C concrete pure compression strain limit

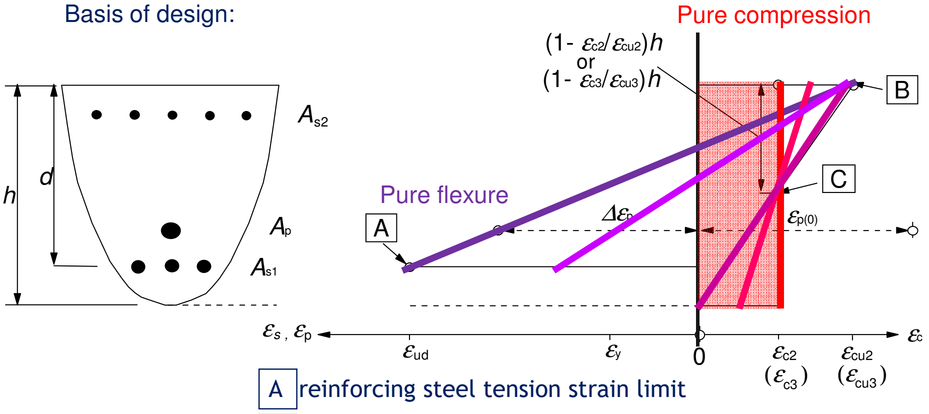


Section Design:
Bending with/without Axial Load

Concise Figure 6.3

EC2 Figure 6.1

Basis of design:



A reinforcing steel tension strain limit

B concrete compression strain limit

C concrete pure compression strain limit

Section Design:
Bending with/without Axial Load

Concise 15.9.2, 15.9.3

Design

Either: iterate such that $A_{sN} = A_{sM}$

- For axial load

  $A_{sN}/2 = (N_{Ed} - \alpha_{cc} h f_{ck} b d_c /\gamma_c)/(\sigma_{sc} - \sigma_{st})$
- For moment

  $A_{sM}/2 = [M_{Ed} - \alpha_{cc} h f_{ck} b d_c (h/2 - d_c/2)/\gamma_c]/[(h/2 - d_2)(\sigma_{sc} + \sigma_{st})$

Or: Calculate d_2/h, N_{Ed}/bhf_{ck} and $M_{Ed}/bh^2 f_{ck}$

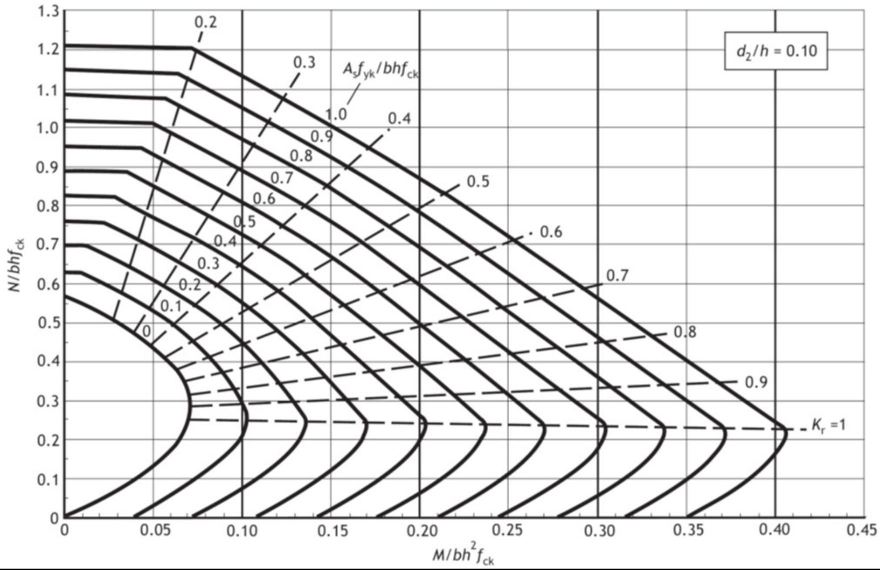
And use column charts . . . .

to find $A_s f_{yk}/bhf_{ck}$ and thus A_s

Column Design Chart

-

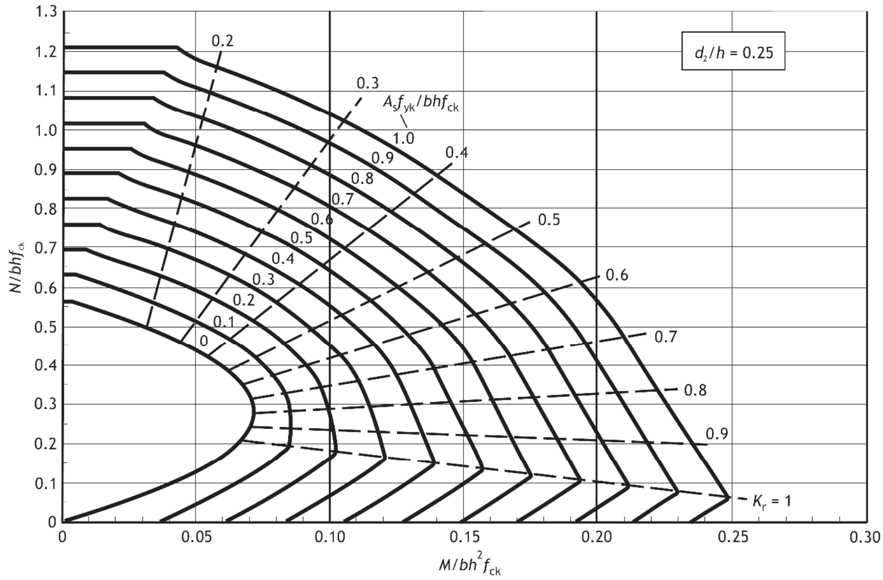
Figure 15.5b



Column Design Chart

-

Figure 15.5e



Biaxial Bending

Cl. 5.8.9 5.6.3

Having done the analysis and design one way, you have to do it in the other direction and check biaxial bending. Often it will be non-critical by inspection but one should check . . . .

$$\left(\frac{M_{\mathrm{Edz}}}{M_{\mathrm{Rdz}}}\right)^{a} + \left(\frac{M_{\mathrm{Edy}}}{M_{\mathrm{Rdy}}}\right)^{a} \leq 1{,}0$$

Note: imperfections need only be taken in the one, more critical direction so either M_{Edz} or M_{Edy} might be reduced in this check

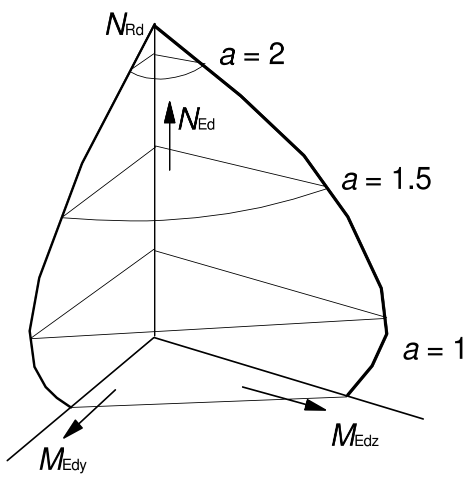
For rectangular cross-sections

N_{Ed}/N_{Rd}	0.1	0.7	1.0
a	1.0	1.5	2.0

where $N_{Rd} = A_c f_{cd} + A_s f_{yd}$

For circular cross-sections $a = 2.0$

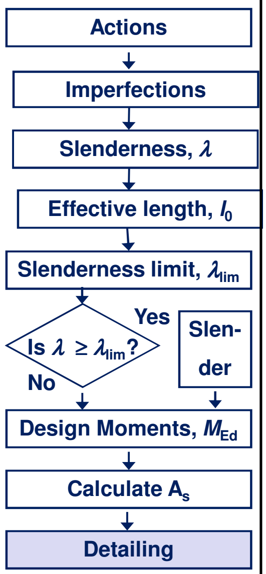
Biaxial bending for a rectangular column



Details/Detailing

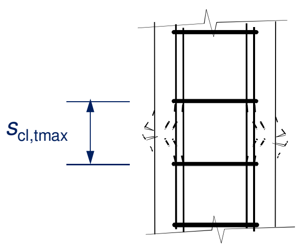
EC2 (9.5.2)

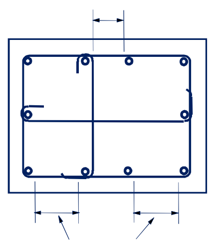
- $h \leq 4b$ (otherwise a wall)
- $\phi_{min} \geq 12$
- $A_{s,min} = 0,10 N_{Ed}/f_{yd}$ but $\geq 0,002\ A_c$
- $A_{s,max} = 0.04\ A_c$ ($0,08 A_c$ at laps)
- Minimum number of bars in a circular column is 4.
- Where direction of longitudinal bars changes more than 1:12 the spacing of transverse reinforcement should be calculated.



Links

EC2 (9.5.3)





No compression bar > 150 mm from a restraining bar

Link diam

= max (6, $\phi_{max}/4$)

(For HSC columns see NA)

Link spacing

$s_{cl,tmax}$ = min {20 ϕ_{min}; b ; 400mm}

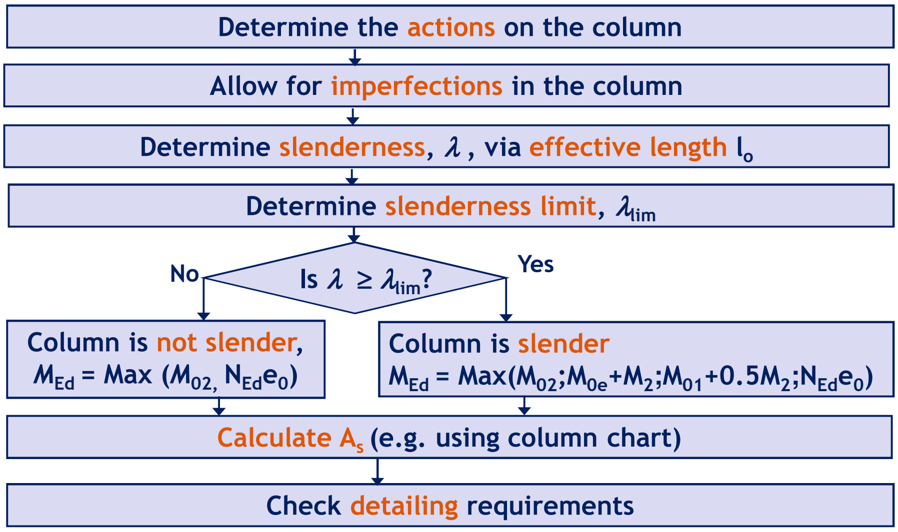
but $s_{cl,tmax}$ should be reduced by a factor 0.6:

- in sections within h above or below a beam or slab
- near lapped joints where ϕ > 14.

$s_{cl,tmax}$ = min {12 ϕ_{min}; 0.6b ; 240mm}

A min of 3 links are required in lap length

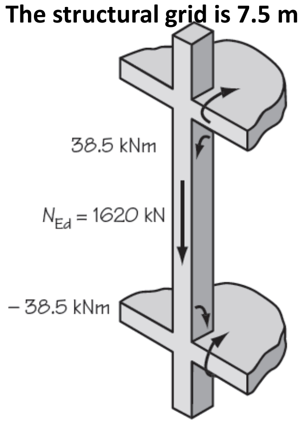
The Column Design Process



Worked Example

A 300 mm square column on the edge of a flat slab structure supports an axial load of 1620 kN and first order moments of 38.5 kNm top and –38.5 kNm bottom in one direction only‡. The concrete is grade C30/37, f_{ck} = 30 MPa and cover, c_{nom}, = 25 mm. The 250 mm thick flat slabs are at 4000 mm vertical centres.

The structural grid is 7.5 m in each direction. What is M_{Ed} & A_s?



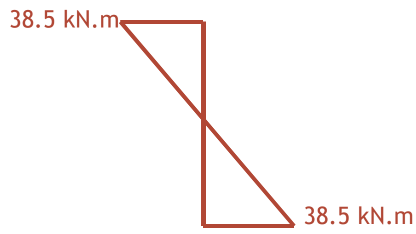


Figure 5.1 Forces in edge column

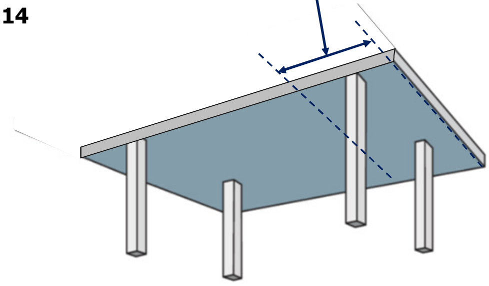
Worked Examples to EC2 - Example 5.1

Solution - effective length & slenderness (M_2)

Using PD 6687 method

$$k = \frac{\frac{EI_c}{L_c}}{\sum \frac{2EI_b}{L_b}} = \frac{\frac{300^4/12}{3750}}{\frac{2 \times 3750 \times 250^3/12}{7500}} = 0.14$$

NB. 3750 ≡ half bay width for flat slab



From table (Table 4 of How to Columns)

k2	k1										
	0.10	0.20	0.30	0.40	0.50	0.70	1.00	2.00	5.00	9.00	Pinned
0.10	0.59	0.62	0.64	0.66	0.67	0.69	0.71	0.73	0.75	0.76	0.77
0.20	0.62	0.65	0.68	0.69	0.71	0.73	0.74	0.77	0.79	0.80	0.81
0.30	0.64	0.68	0.70	0.72	0.73	0.75	0.77	0.80	0.82	0.83	0.84
0.40	0.66	0.69	0.72	0.74	0.75	0.77	0.79	0.82	0.84	0.85	0.86
0.50	0.67	0.71	0.73	0.75	0.76	0.78	0.80	0.83	0.86	0.86	0.87
0.70	0.69	0.73	0.75	0.77	0.78	0.80	0.82	0.85	0.88	0.89	0.90
1.00	0.71	0.74	0.77	0.79	0.80	0.82	0.84	0.88	0.90	0.91	0.92
2.00	0.73	0.77	0.80	0.82	0.83	0.85	0.88	0.91	0.93	0.94	0.95
5.00	0.75	0.79	0.82	0.84	0.86	0.88	0.90	0.93	0.96	0.97	0.98
9.00	0.76	0.80	0.83	0.85	0.86	0.89	0.91	0.94	0.97	0.98	0.99
Pinned	0.77	0.81	0.84	0.86	0.87	0.90	0.92	0.95	0.98	0.99	1.00

F = 0.61. So l_o = 0.61 x 3.750 = 2.29m

Calculate slenderness:

$\lambda = 3.46\, l_o/h$

$= 3.46 \times 2.29 / 0.3 = 26.4$

Solution - slenderness limit (M_2)

Limiting slenderness:

$\lambda_{lim} = 20\,ABC / \sqrt{n}$

where

$A = 0.7$ (default)

$B = 1.1$ (default)

$C = 1.7 - r_m$

$= 1.7 - M_{01}/M_{02}$

$= 1.7 - (-38.5 + 9.3) / (38.5 + 9.3)$

$= 1.7 - -29.2 / 47.8 = 2.31$

$n = N_{Ed} / A_c f_{cd}$

$= 1620 \times 10^3 / (300^2 \times 0.85 \times 30 / 1.5)$

$= 1.06$

$\lambda_{lim} = 20\,ABC / \sqrt{n} \quad = 20 \times 0.7 \times 1.1 \times 2.31 / 1.06^{0.5} = 34.5$

So

$\lambda_{lim} = 34.5$ i.e. > 26.4 ∴ column is not slender.

And $M_2 = 0$ kNm.

$e_i = l_0 / 400$
$= 2290/400$
$= 5.7$ mm
$N_{Ed} = 1620$ kN
$e_i N_{Ed} = 1620 \times 0.0057$
$= 9.3$ kNm

Solution - design moments

M_{Ed} = max[M_{02}; $M_{0Ed} + M_2$; $M_{01} + 0.5M_2$; $N_{Ed}e_0$]
where:

$M_{02} = M + e_i N_{Ed}$
where

M = 38.5 kNm
$e_i = l_0 / 400$ = 2290/400 = 5.7 mm
N_{Ed} = 1620 kN

M_{02} = 38.5 + 1620 × 0.0057
= 38.5 + 9.3
= 47.8 kNm

M_{01} = - 38.5 + 9.3 = - 29.2 kNm

$M_{0Ed} = (0.6M_{02} + 0.4M_{01}) \geq 0.4M_{02}$
= 0.6 × 47.8 + 0.4 × (– 29.2) ≥ 0.4 × 47.8
= 17.0 ≤ 19.1
= 19.1 kNm

M_2 = 0 kNm

Solution - design moments

Minimum moment

e_0N_{Ed}

e_0 = max[h/30; 20] = max[300/30; 20] = 20 mm
N_{Ed} = 1620 kN

e_0N_{Ed} = 0.02 × 1620
= 32.4 kNm

Solution - design moments

M_{Ed} = max[M_{02}; M_{0Ed} + M_2; M_{01} + 0.5M_2 ; $N_{Ed}e_0$]
where:

M_{02} = 47.8 kNm
M_{01} = - 29.2 kNm
M_{0Ed} = 19.1 kNm
M_2 = 0 kNm
e_0N_{Ed} = 32.4 kNm

M_{Ed} = max[*47.8; 19.1 + 0 ; -29.2 + 0; 32.4*]
= <u>47.8 kNm</u>

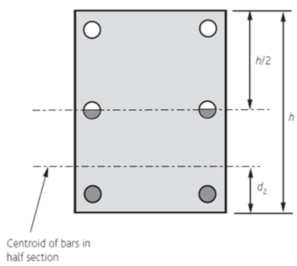
Solution - determine A_s

Using design charts: Worked Examples to EC2

Require d_2 /h to determine which chart(s) to use:

d_2 = c_{nom} + link + ϕ / 2 = 25 + 8 + say 32/2 = 49 mm
d_2 / *h* = 49 / 300
= 0.163

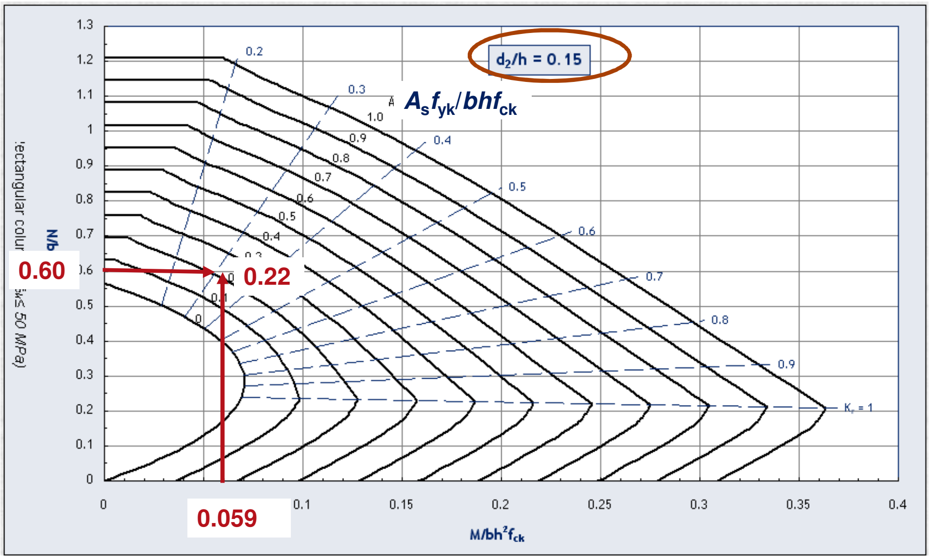
NB.
Charts are for symmetrically reinforced. They actually work on centroid of the reinforcement in half the section. So when no. of bars > 4 beware. See Concise 15.9.3



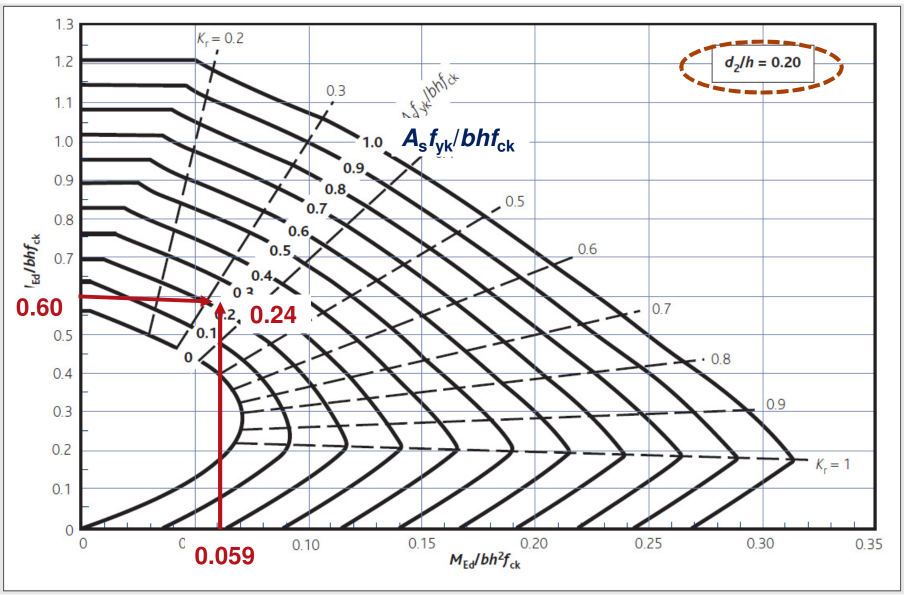
∴ Assuming 4 bars and interpolating between d_2 / *h* = 0.15 (Fig 15.5c) and 0.20 (Fig 15.5d)
for:-

N_{Ed} / bhf_{ck} = 1620 × 10³ / (300² × 30) = 0.60
M_{Ed} / bh^2f_{ck} = 47.8 × 10⁶ / (300³ × 30) = 0.059

Interaction Chart



Interaction Chart



Solution - determine A_s

$A_s f_{yk} / bhf_{ck}$ = 0.225 by interpolation

$A_s = 0.225 \times 300^2 \times 30 / 500 = 1215 \text{ mm}^2$

<u>Try 4 no. H20 (1260 mm²)</u>

Links

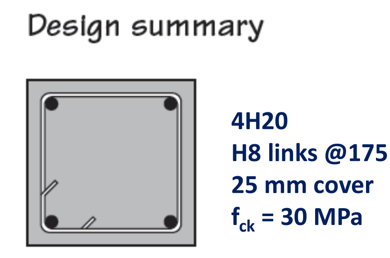
Diam min. $\geq \phi / 4$

$\geq 20 / 4 = 5$ mm

Max. spacing = 0.6×300

= 180 mm

<u>So use H8 links @ 175c/c</u>



Design summary: edge column

Often the analysis and design would have to be undertaken for the other axis and where necessary checked for biaxial bending: in this case neither is critical

Workshop Problem - Your turn

The suspended slabs (including the ground floor slab) are 300 mm thick flat slabs at 4500 mm vertical centres. Between ground and 5th floors the columns at C2 are 500 mm square; above 5th floor they are 465 mm circular. Assume an internal environment, 1-hour fire resistance and f_{ck} = 50 MPa.

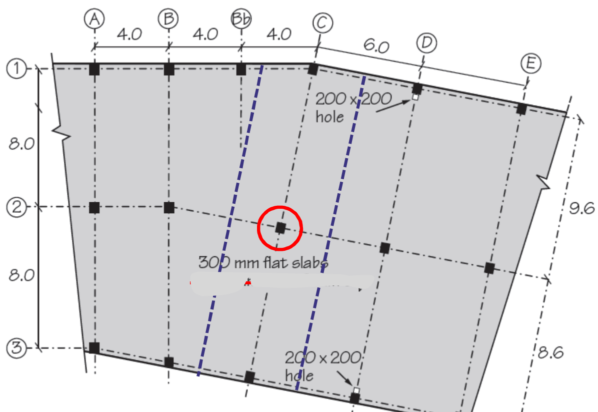


Figure 5.6 Part plan of flat slab

Assume the following:

- Axial load: 7146kN
- Top Moment: 95.7kN
- Bottom Moment: -95.7kN
- Nominal cover: 35mm
- Bay width is 6.0 m
- Elastic modulus is the same for column and slab

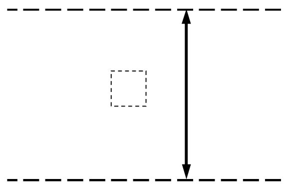
Design the column C2 between 1st and 2nd floors for bending about axis parallel to line 2.

Your solution

Use PD 6687 method

Column clear span is 4500 - 300 = 4200 mm and $k_1 = k_2$

$$k = \frac{\frac{EI_c}{L_c}}{\sum \frac{2EI_b}{L_b}} =$$



Take 'beam' width as, say, half the bay width

From Table (Table 4 of How to…Columns)

k2	k1										
	0.10	0.20	0.30	0.40	0.50	0.70	1.00	2.00	5.00	9.00	Pinned
0.10	0.59	0.62	0.64	0.66	0.67	0.69	0.71	0.73	0.75	0.76	0.77
0.20	0.62	0.65	0.68	0.69	0.71	0.73	0.74	0.77	0.79	0.80	0.81
0.30	0.64	0.68	0.70	0.72	0.73	0.75	0.77	0.80	0.82	0.83	0.84
0.40	0.66	0.69	0.72	0.74	0.75	0.77	0.79	0.82	0.84	0.85	0.86
0.50	0.67	0.71	0.73	0.75	0.76	0.78	0.80	0.83	0.86	0.86	0.87
0.70	0.69	0.73	0.75	0.77	0.78	0.80	0.82	0.85	0.88	0.89	0.90
1.00	0.71	0.74	0.77	0.79	0.80	0.82	0.84	0.88	0.90	0.91	0.92
2.00	0.73	0.77	0.80	0.82	0.83	0.85	0.88	0.91	0.93	0.94	0.95
5.00	0.75	0.79	0.82	0.84	0.86	0.88	0.90	0.93	0.96	0.97	0.98
9.00	0.76	0.80	0.83	0.85	0.86	0.89	0.91	0.94	0.97	0.98	0.99
Pinned	0.77	0.81	0.84	0.86	0.87	0.90	0.92	0.95	0.98	0.99	1.00

F =

$\therefore l_o$ =

Check slenderness:

$\lambda = 3.46\, l_o/h$

=

Your solution

$M_{01} = \text{Min}\{|M_{top}|, |M_{bottom}|\} + e_i N_{Ed}$

$M_{02} = \text{Max}\{|M_{top}|, |M_{bottom}|\} + e_i N_{Ed}$

$\text{Min}\{|M_{top}|, |M_{bottom}|\} =$

$\text{Max}\{|M_{top}|, |M_{bottom}|\} =$

$e_i N_{Ed}$: $e_i = l_0/400 =$

$N_{Ed} =$

$e_i N_{Ed} =$ kNm

$\therefore M_{02} =$

$M_{01} =$

$M_{0Ed} = (0.6M_{02} + 0.4M_{01}) \geq 0.4M_{02}$

$=$

Minimum moment ($e_0 N_{Ed}$)

$e_0 = \text{Max}\{h/30; 20\text{mm}\}$

Your solution

Check whether the column is stocky or slender using λ_{lim} method:

A = 0.7 (use default value)

B = 1.1 (use default value)

C = $1.7 - r_m = 1.7 - M_{01}/M_{02}$ =

n = $N_{Ed}/A_c f_{cd}$ =

(f_{ck} = 50 MPa, $f_{cd} = \alpha_{cc}.f_{ck}/\gamma_m$)

λ_{lim} = $20\,ABC/\sqrt{n}$

=

λ =

The column is/is not slender

M_2 =

Your solution

$M_{Ed} = \max[M_{02};\ M_{0Ed} + M_2;\ M_{01} + 0.5M_2\ ;\ e_0N_{Ed}]$

M_{02} =

$M_{0Ed} + M_2$=

$M_{01} + 0.5M_2$=

e_0N_{Ed}=

$\therefore M_{Ed}$ =

Your Solution - determine A_s

$d_2 = c_{nom}$ + link + $\phi/2$ =

d_2/h =

$M_{Ed}/(bh^2 f_{ck})$ =

$N_{Ed}/(bhf_{ck})$ =

Look up charts are in "Concise" pages 97-99

Your Solution - determine A_s

$A_s f_{yk}/bhf_{ck}$ =

A_s = mm^2 Try H (mm^2)

Links

Diameter = max {6, ø/4} = mm

$s_{cl,tmax}$ = min {12 ϕmin; 0.6b ; 240mm} = mm

Try H @ c/c

mpa The Concrete Centre

End of Lecture 8