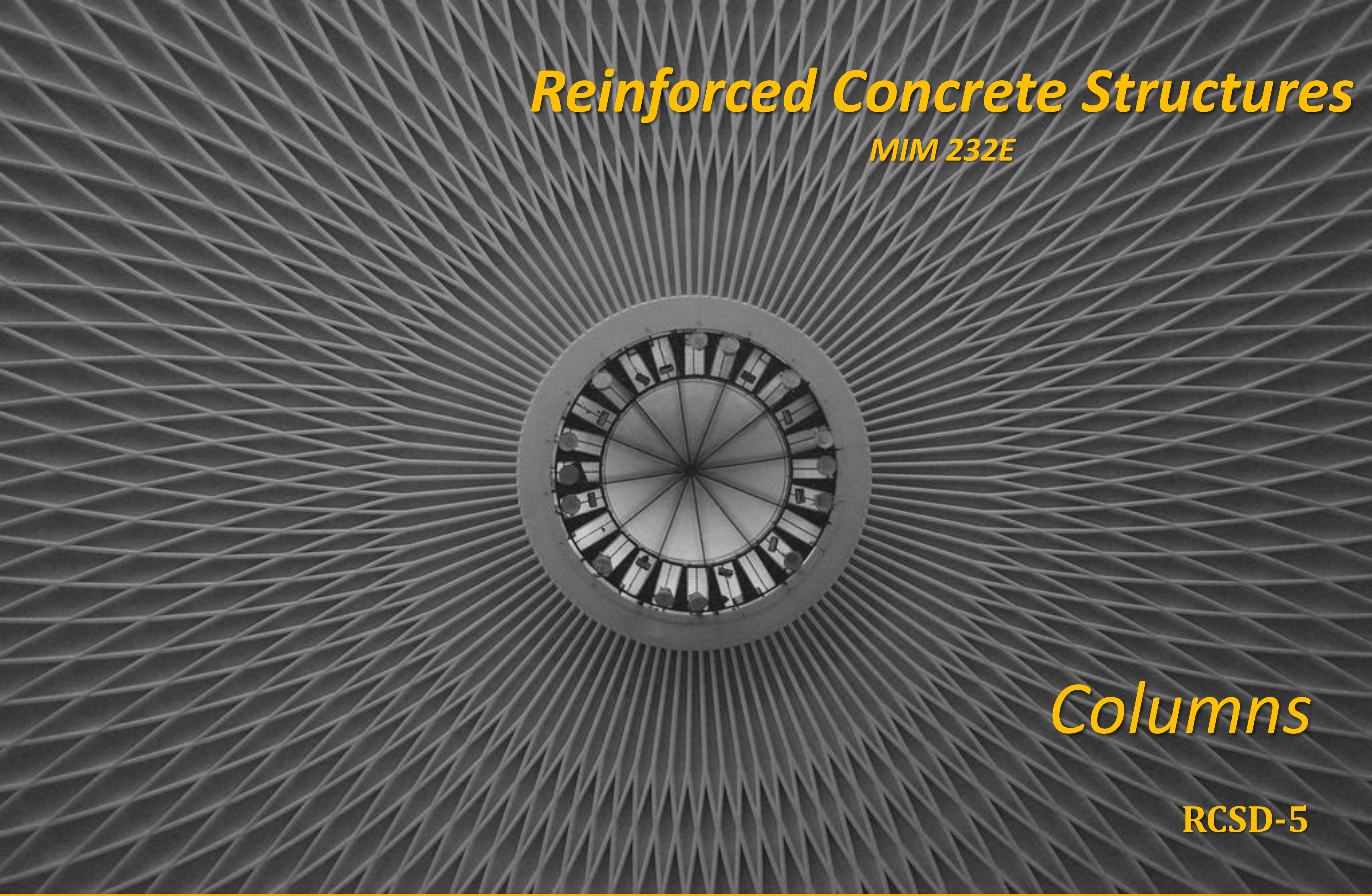


Reinforced Concrete Structures

MIM 232E

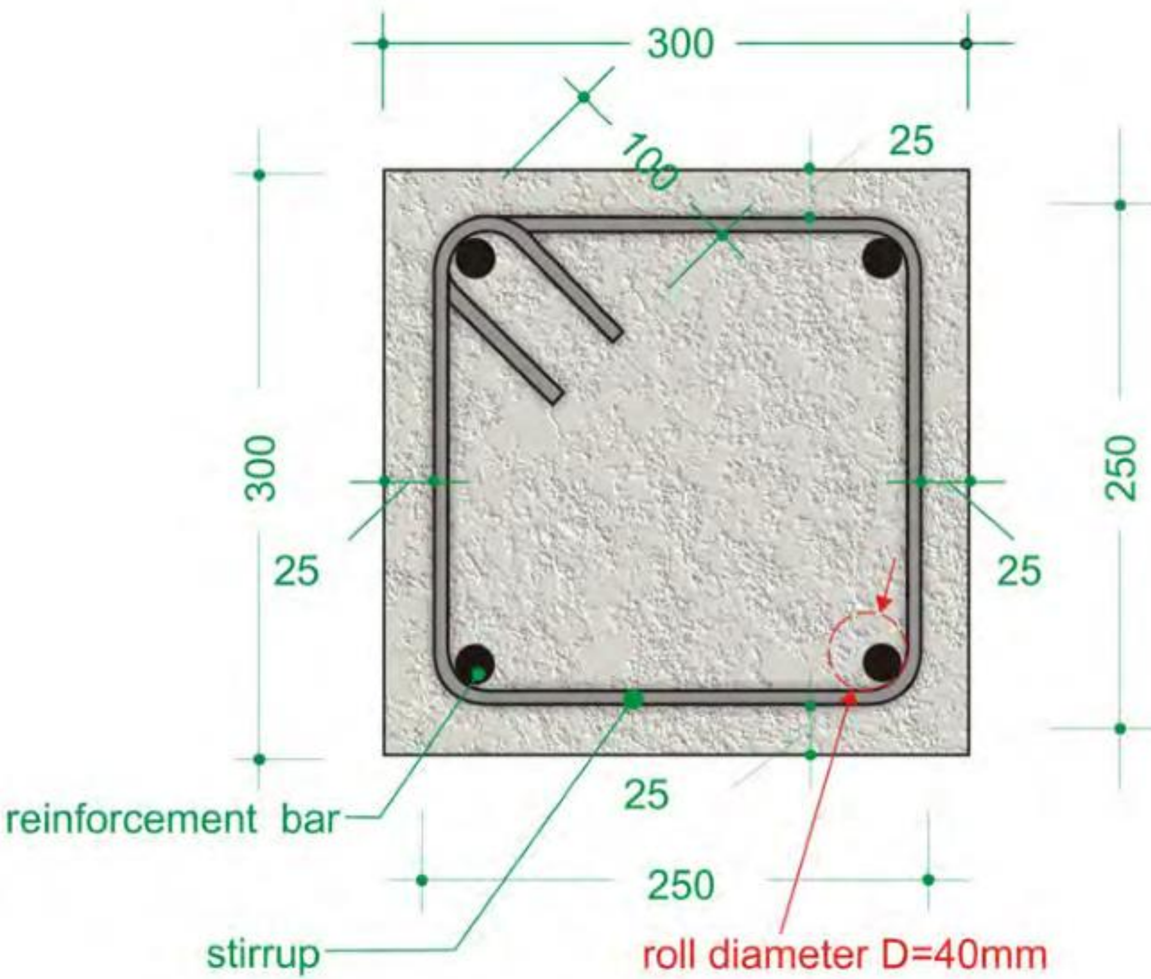
Columns

RCSD-5

Dr. Haluk Sesigür
I.T.U. Faculty of Architecture
Structural and Earthquake Engineering WG

Columns

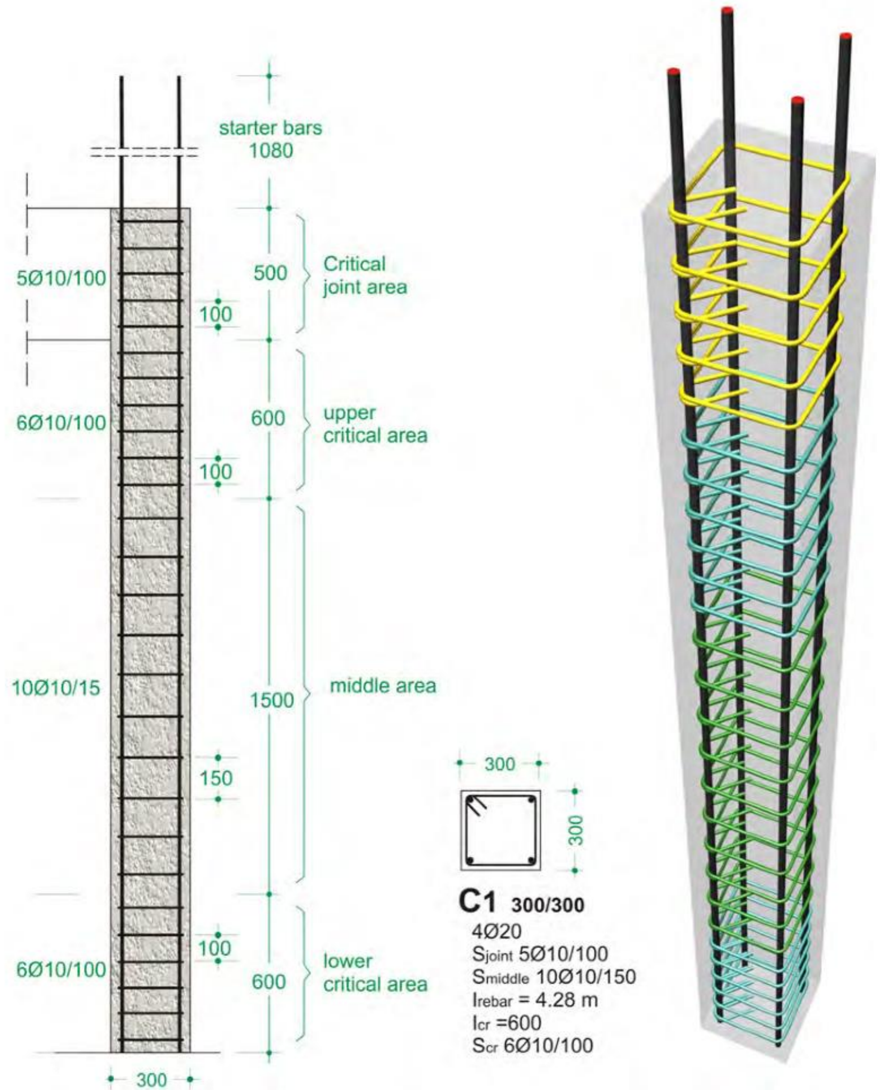
Concept



Columns

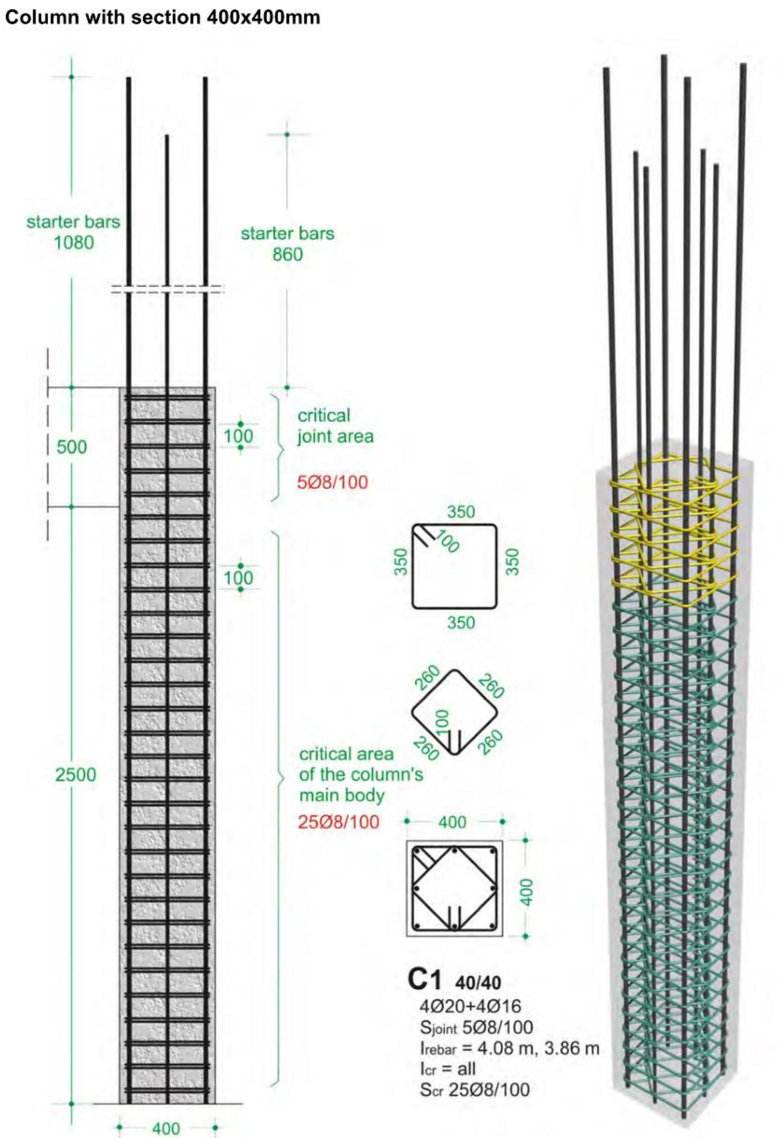
Concept

COLUMN 300 x 300 mm (with critical and non-critical areas)



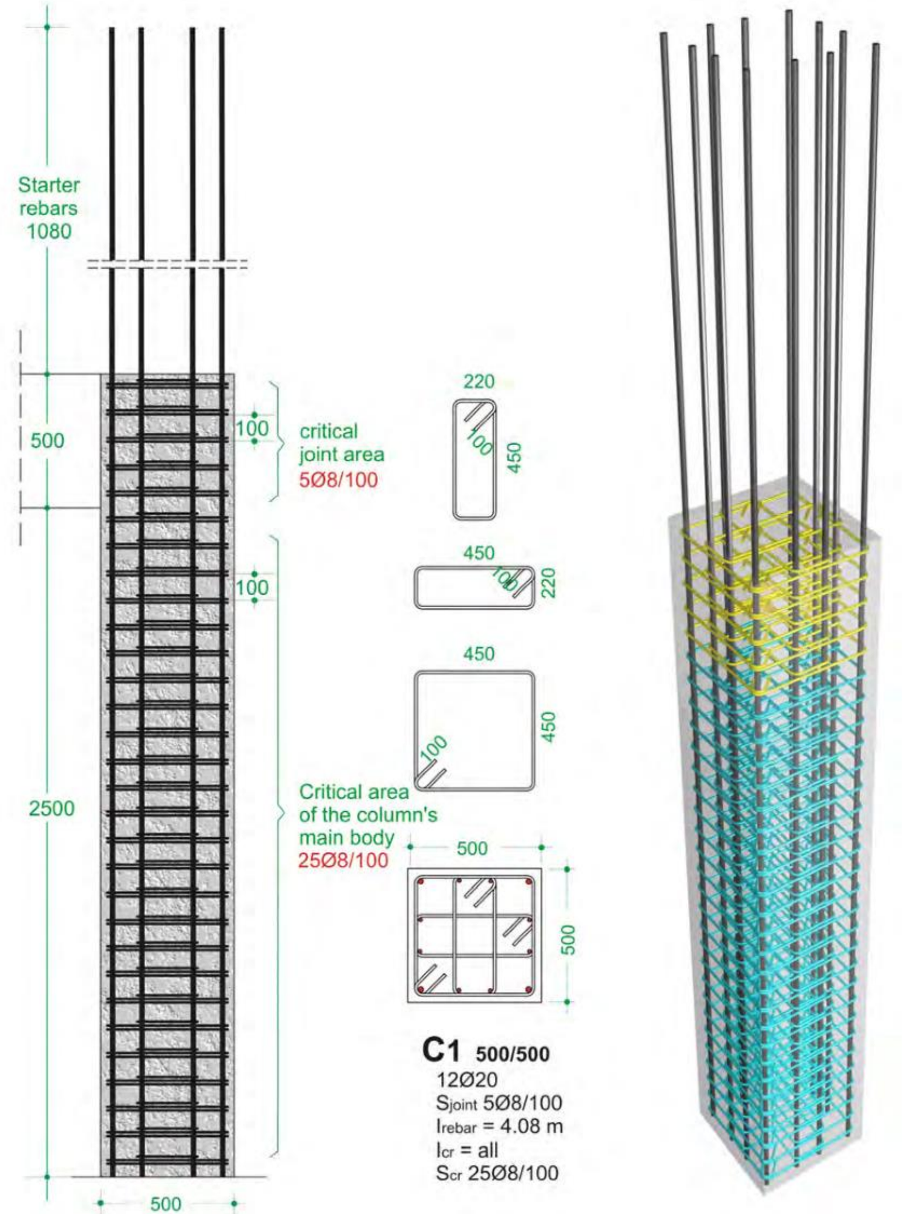
Columns

Concept



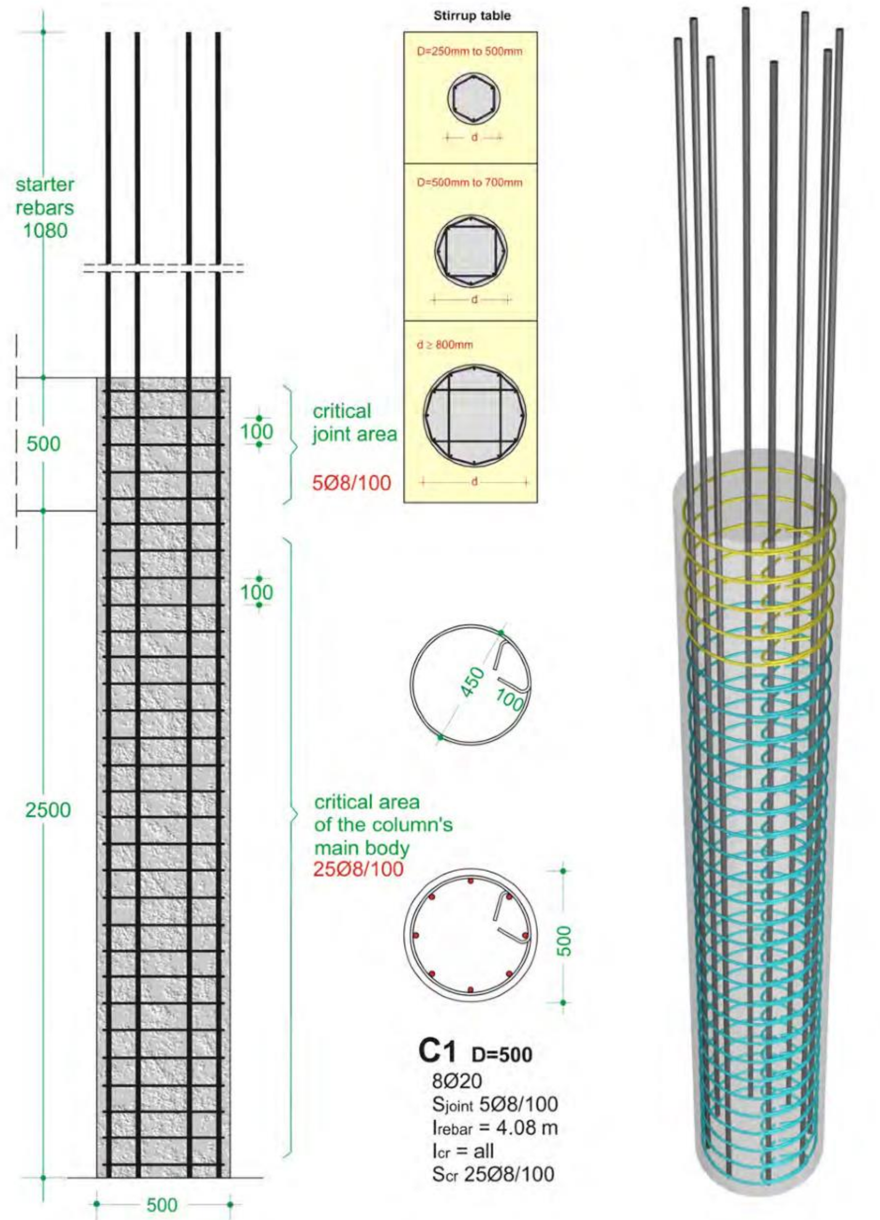
Columns

Concept



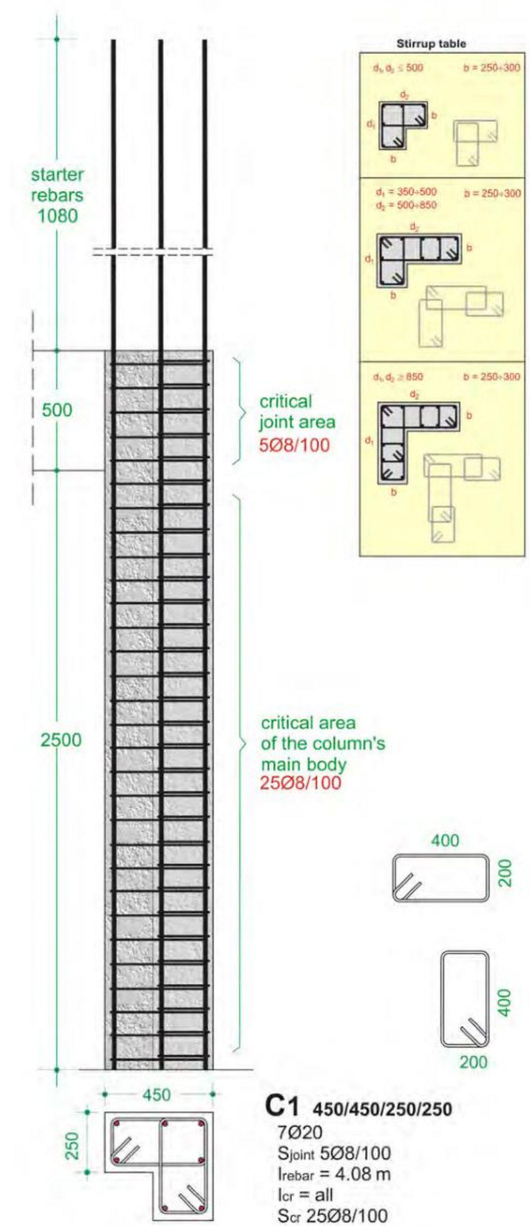
Columns

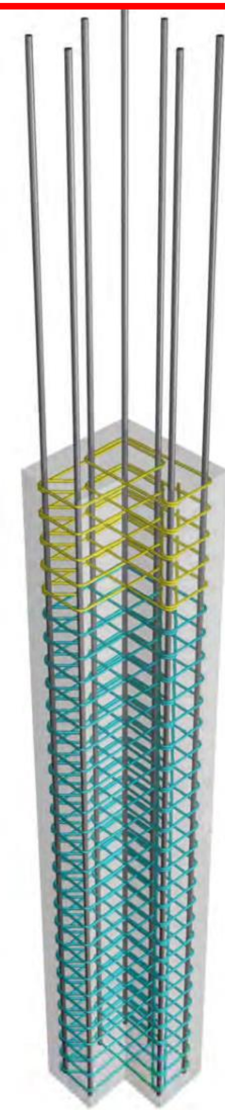
Concept



Columns

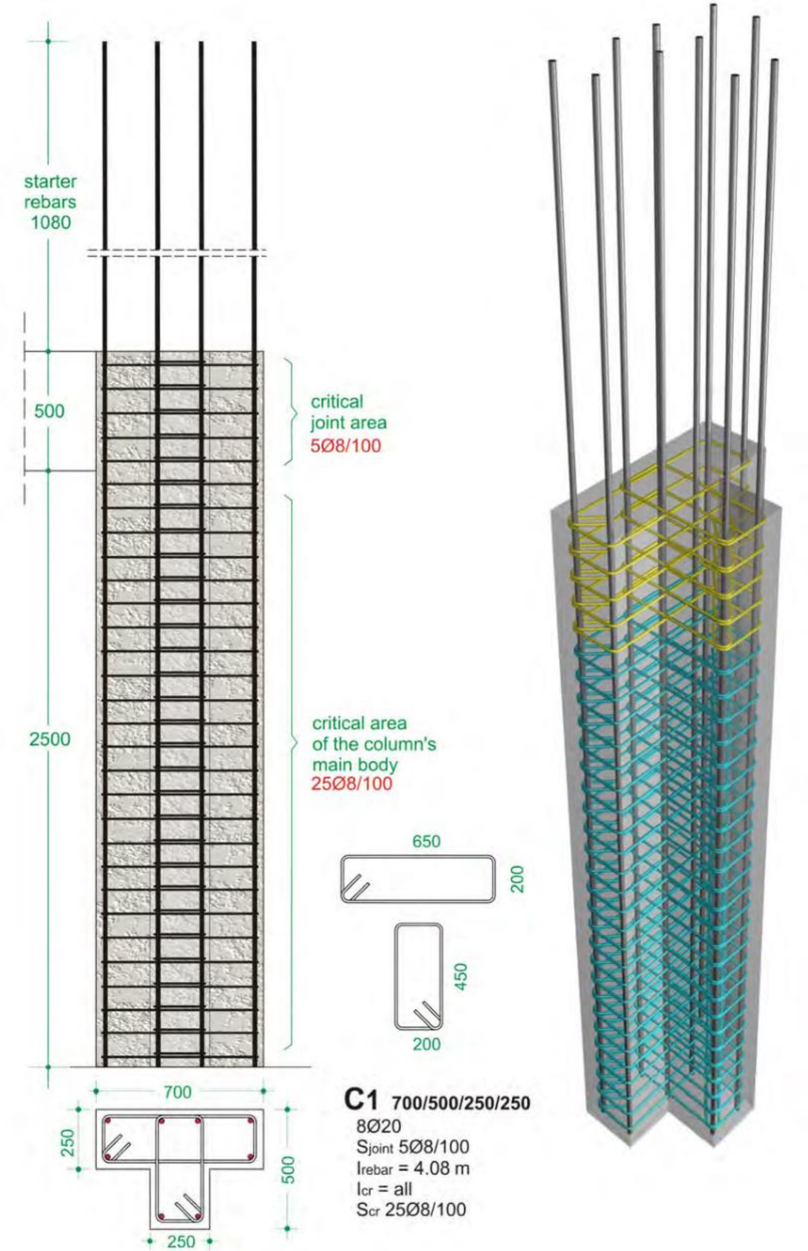
Concept





Columns

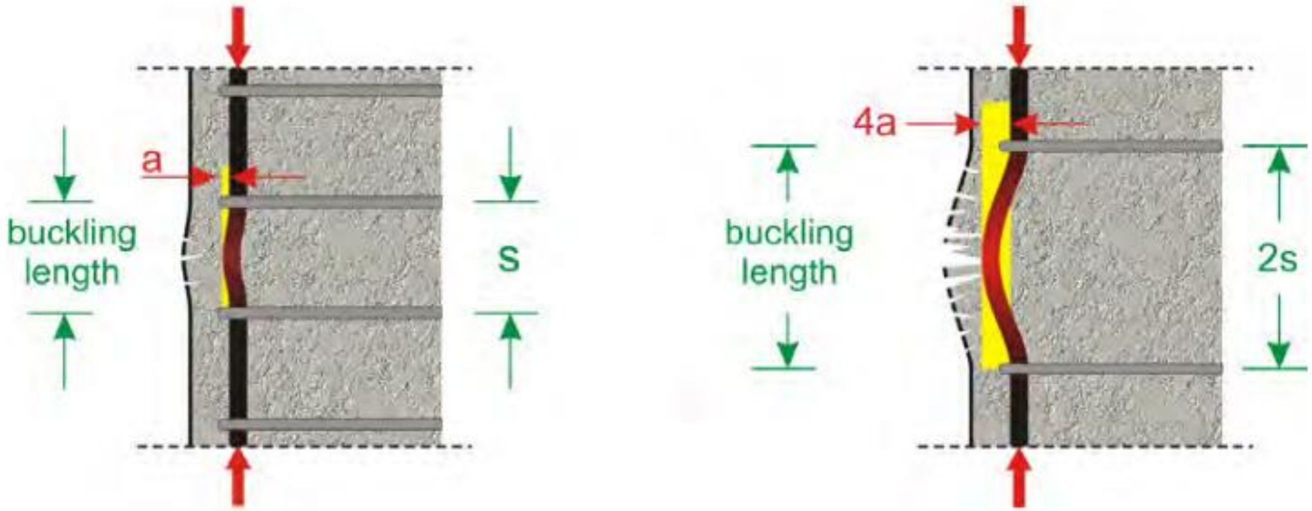
Concept



Columns

Concept

A column with 10% fewer rebars has around 10% lower capacity strength. However, if we remove even a single intermediate stirrup, the capacity strength of that same column will be lowered even by 50%. This happens because the stirrup's removal doubles the buckling length of the rebars previously enclosed by it.



In a seismic event, columns always fail in the same way:

a. When stirrups open, concrete disintegration in the column's head or foot occurs.

b. Once the stirrups' ends become apart, longitudinal reinforcement buckling and concrete disintegration take place.

That type of failure does not appear only to columns dimensioned according to old regulations and therefore have fewer rebars but also to newer columns with large amount of reinforcement, when they are not constructed according to the correct specifications:

a. with internal and external stirrup adequacy,

b. with correctly formed, antiseismic stirrups.

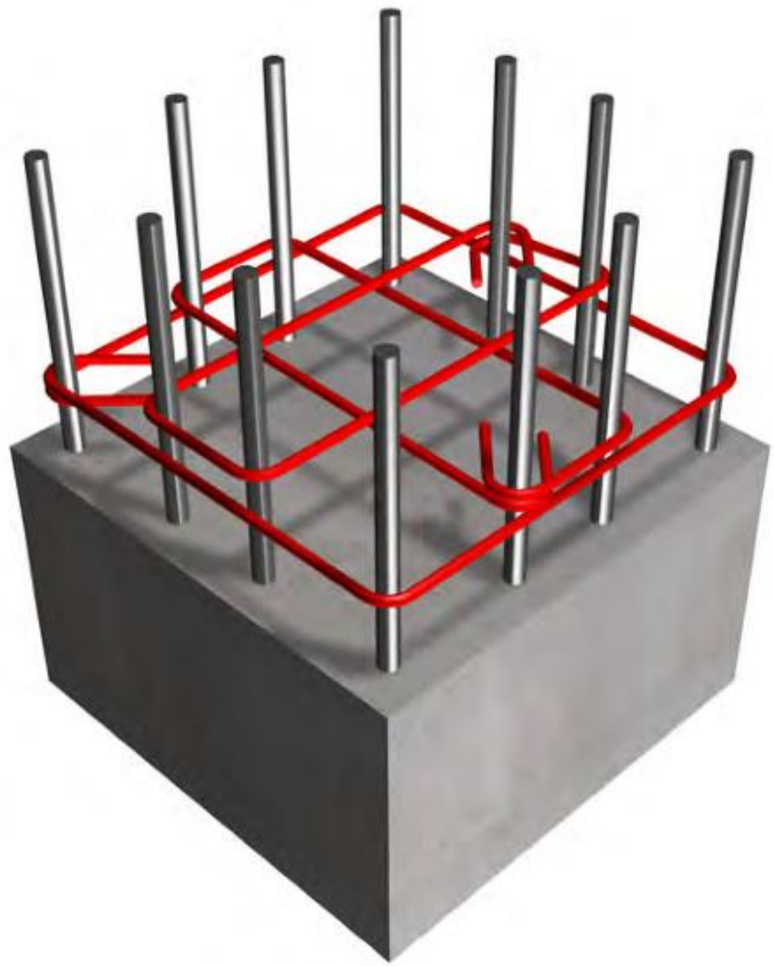
Throughout the world, structures collapse even when they have a large amount of reinforcement. The reason for this is always the same; lack of properly shaped and placed stirrups.

The earthquake resistance of beams and columns depends mainly upon their vertical reinforcement. Stirrups ensure the confinement of the rebars fitted inside them and the integrity of the concrete that tends to spall due to lateral enlargement. If stirrups are not properly anchored they may open even in low intensity seismic events.

Failure of a column dimensioned according to old regulations that required a peripheral stirrup with its end bent in 90° instead of 135° (45°).

Concept

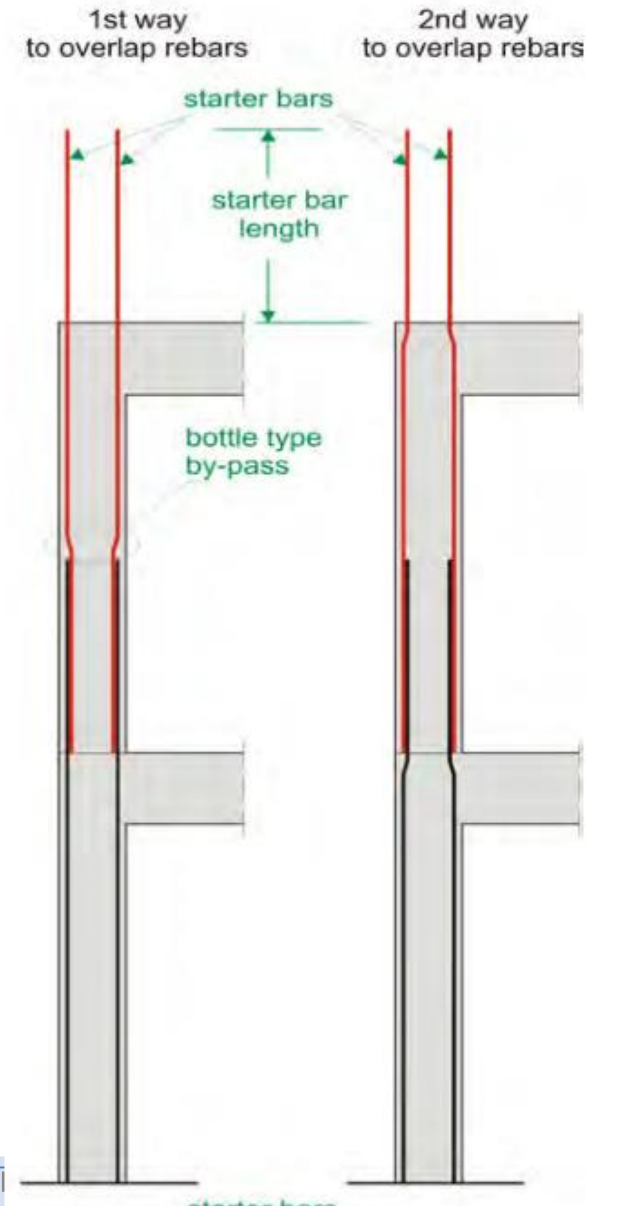
Properly closed stirrup with a hook length equal to 100 mm, bent in a 40 mm diameter roll (for stirrup bar Ø 10)



Properly reinforced column 500x500 with 3 stirrups in each layer

Columns

Lap Splices in columns



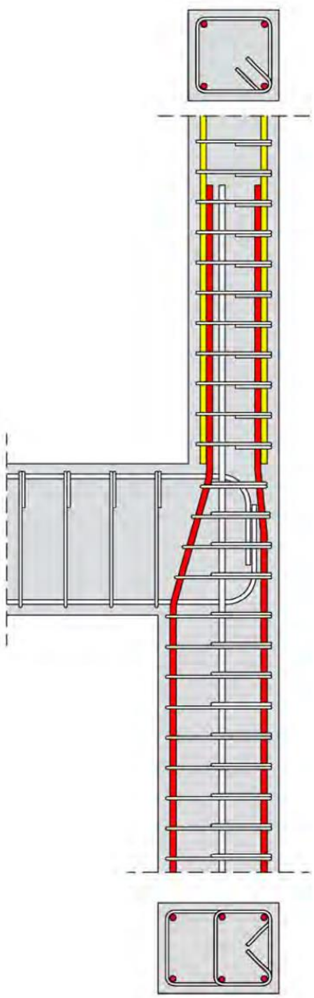
In multi-storey buildings it would be ideal if each of the column's longitudinal rebars could be placed as one single piece throughout the structure's entire height. This however cannot be accomplished for practical reasons therefore, the length of the longitudinal rebars is equal to the height of each storey.

The length of the placed bars must be extended by an additional length called **'lap length'**, which has to be equal or greater than the length required for the lapping of corresponding rebars between two successive storeys. This length is equal to the rebar's diameter multiplied by the 'contact coefficient' (its value varies from 45 to 60).

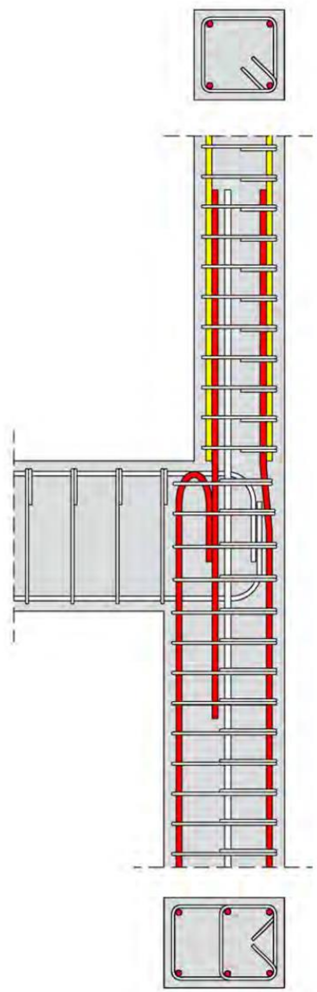
Columns

Reduction of the column's section size

The reduction of the column's size from one storey to the other is not favorable either from a theoretical or from a practical point of view. Especially in columns where earthquake resistant behavior is required it must be avoided. However, there are cases where the subjacent column is larger than is the superjacent column.



Reduction of the section's size along the height by bending rebars into the shape of a bottle



Reduction of the section's size along the height by terminating some of the old rebars and implanting new ones

Columns

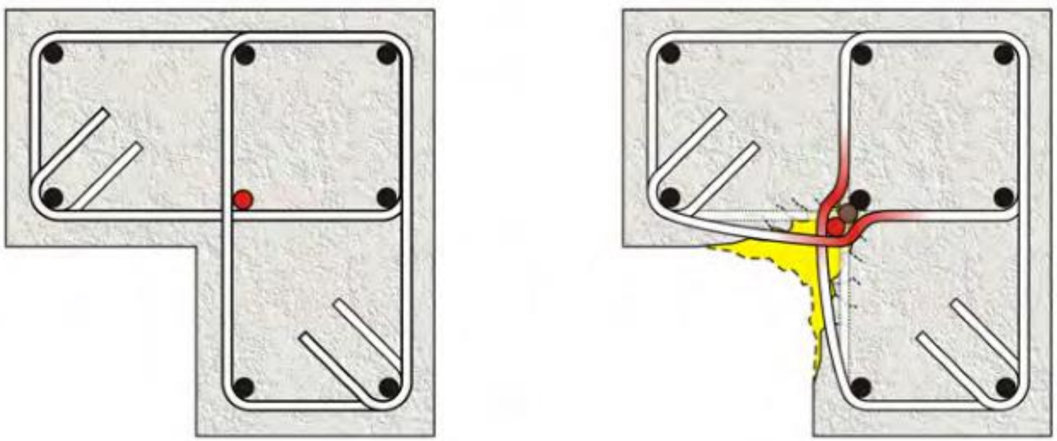
Reinforcement detail

Placing a rebar in the internal corner of composite sections (‘Γ’, ‘Z’ section etc) is optional because:

a) it does not significantly enhance the flexural strength since it is not placed in a section’s corner (based on the fact that tensile stresses appear to the external corners of the elements’ sections)

b) it does not increase the confinement because in case of an intense earthquake, where concrete spalling will occur, that rebar might buckle since it is not completely restrained.

c) however, in all cases this corner rebar may be constructional used without though calculating its contribution to the section’s confinement.



The internal rebar (in red), in a severe seismic event, might buckle (due to the bulking of stirrups) and therefore it will not contribute to the confinement.

Interaction Diagrams

(a) Cross section.

(b) Eccentric load.

(c) Axial load and moment.

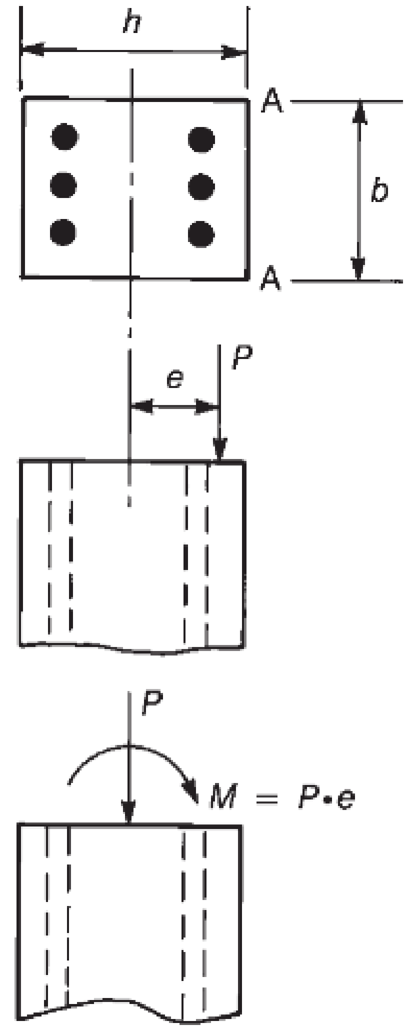


Fig. 11-9
Load and moment on column.

To illustrate conceptually the interaction between moment and axial load in a column, an idealized homogeneous and elastic column with a compressive strength, f_{cu}, equal to its tensile strength, f_{tu}, will be considered. For such a column, failure would occur in compression when the maximum stresses reached f_{cu}, as given by

$$\frac{P}{A} + \frac{My}{I} = f_{cu} \tag{11-2}$$

where

A, I = area and moment of inertia of the cross section, respectively

y = distance from the centroidal axis to the most highly compressed surface (surface A–A in Fig. 11-9a), positive to the right

P = axial load, positive in compression

M = moment, positive as shown in Fig. 11-9c

Dividing both sides of Eq. (11-2) by f_{cu} gives

$$\frac{P}{f_{cu}A} + \frac{My}{f_{cu}I} = 1$$

The maximum axial load the column can support occurs when $M = 0$ and is $P_{max} = f_{cu}A$. Similarly, the maximum moment that can be supported occurs when $P = 0$ and M is $M_{max} = (f_{cu}I/y)$. Substituting P_{max} and M_{max} gives

$$\frac{P}{P_{max}} + \frac{M}{M_{max}} = 1 \qquad (11\text{-}3)$$

Interaction Diagrams

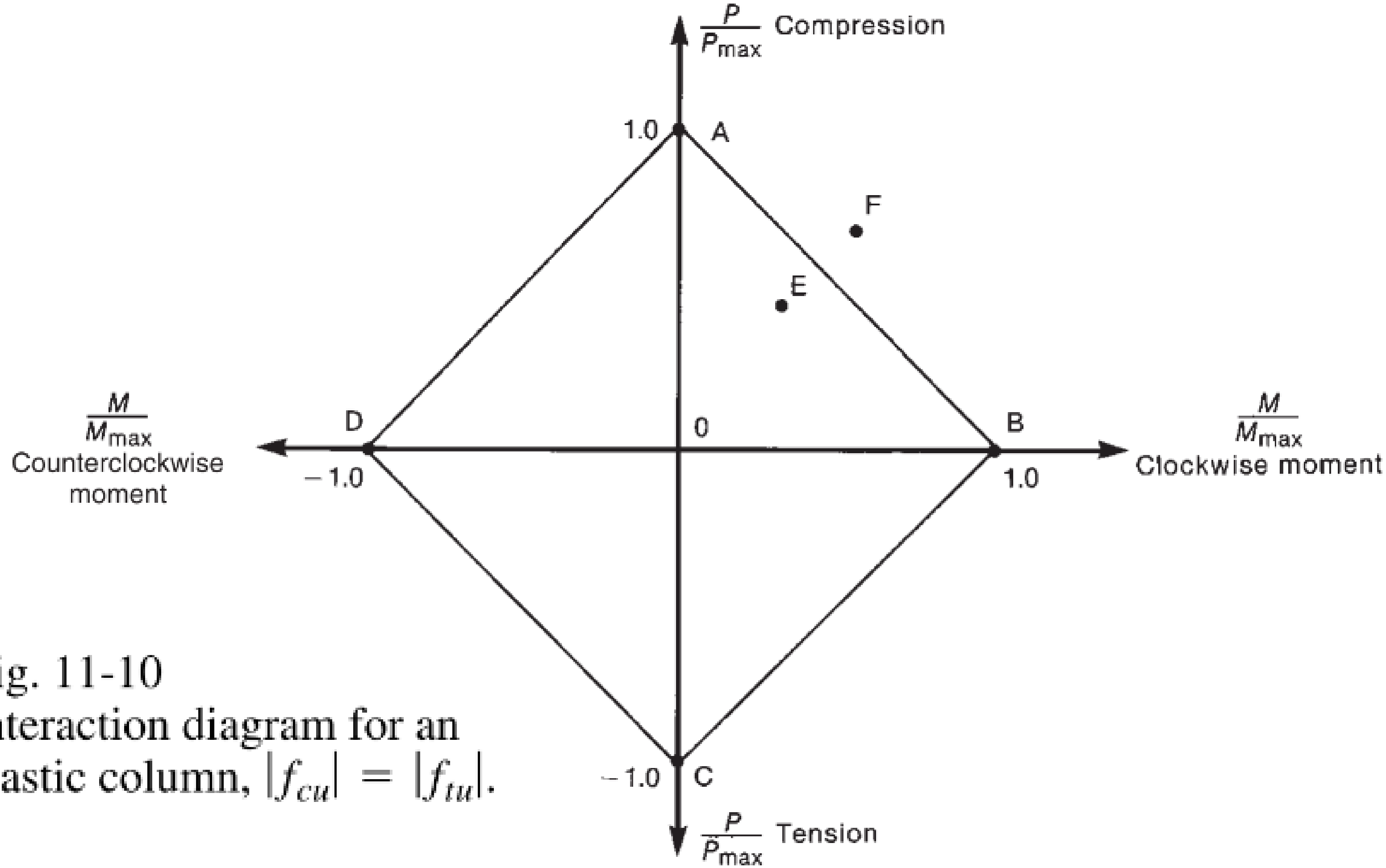
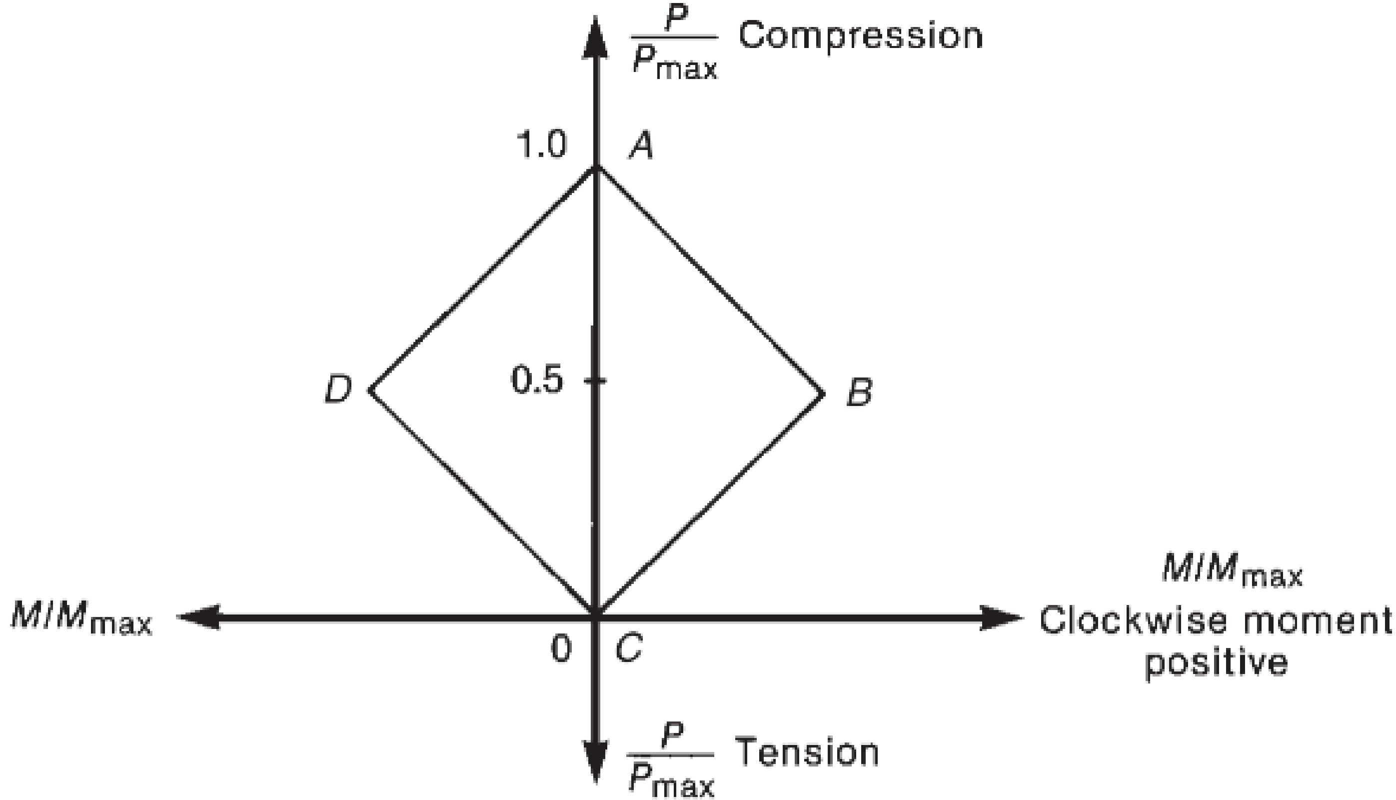


Fig. 11-10
Interaction diagram for an elastic column, $|f_{cu}| = |f_{tu}|$.

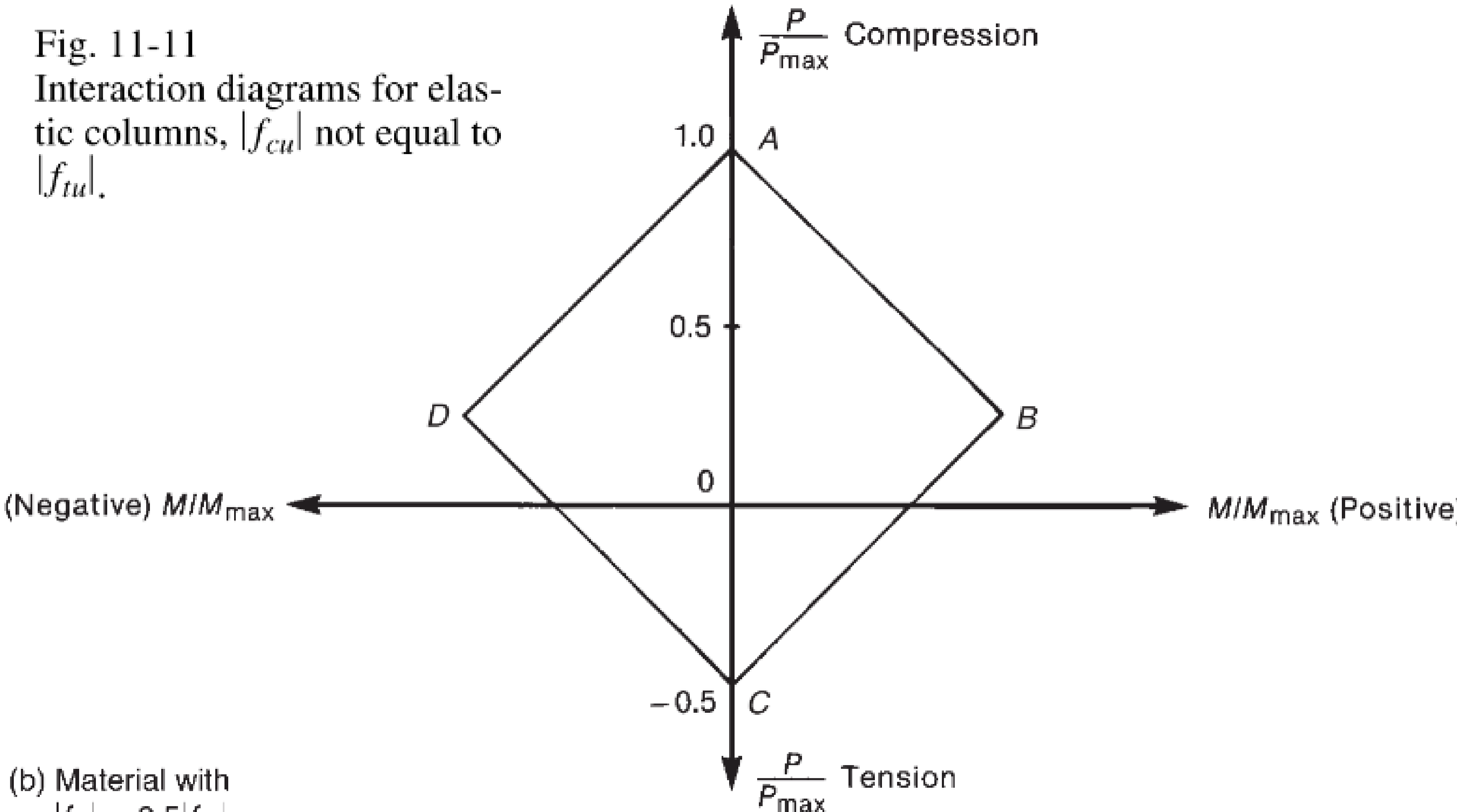
Interaction Diagrams



(a) Material with $f_{tu} = 0$.

Interaction Diagrams

Fig. 11-11
Interaction diagrams for elastic columns, $|f_{cu}|$ not equal to $|f_{tu}|$.



(b) Material with $|f_{tu}| = 0.5|f_{cu}|$.

Columns

Interaction Diagrams

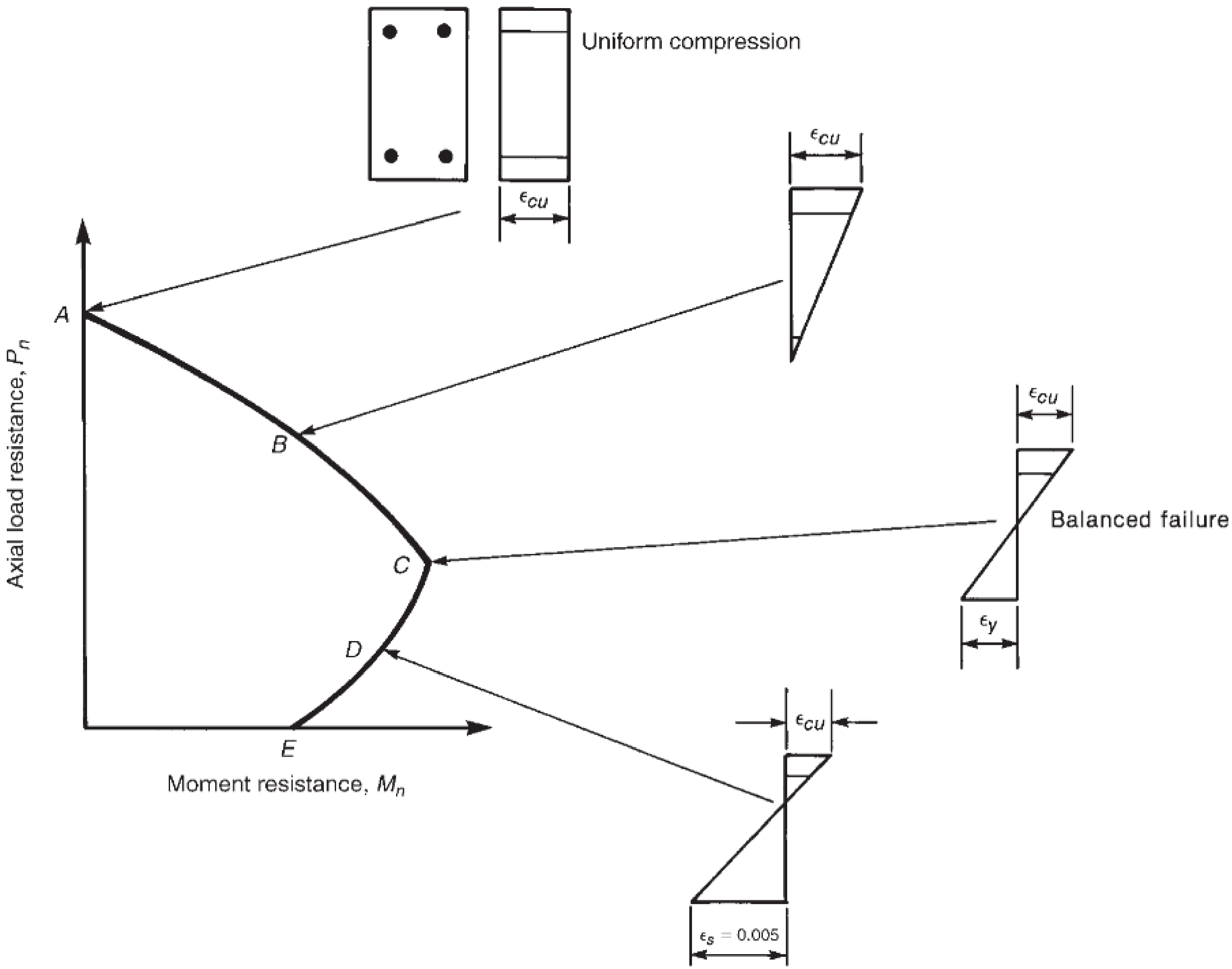


Fig. 11-13
Strain distributions corresponding to points on the interaction diagram.

Interaction Diagrams

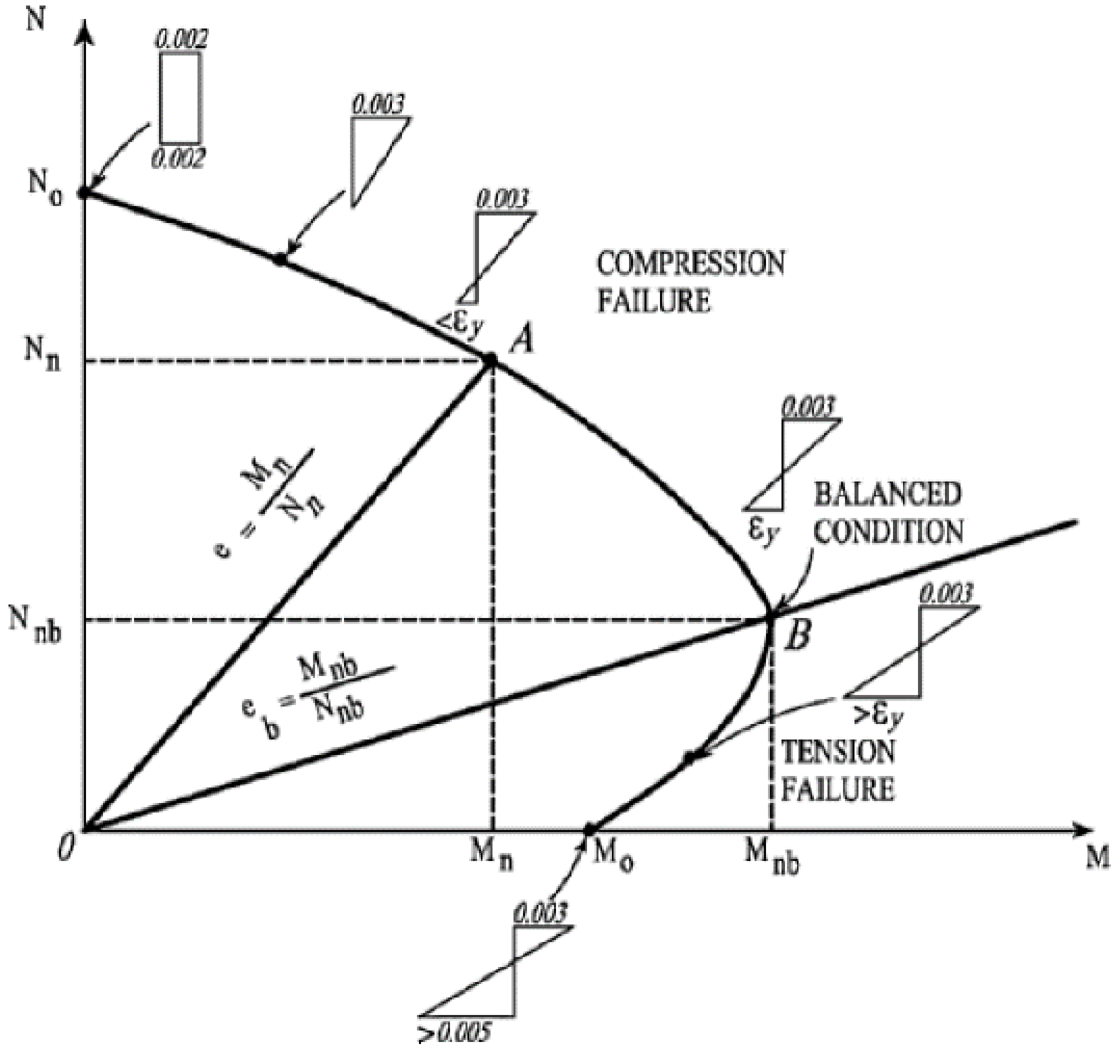


Figure 3.23 Bending–axial load interaction curve

e=M/N

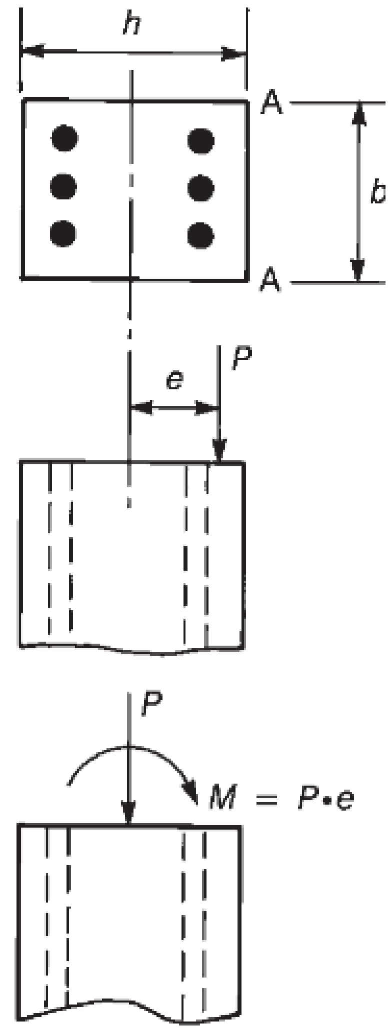
e: eccentricity of load

$e = 0$:	axial load
$e < e_b$:	compression failure
$e = e_b$:	balanced condition
$e > e_b$:	tension failure
$e = \infty$:	pure bending

(a) Cross section.

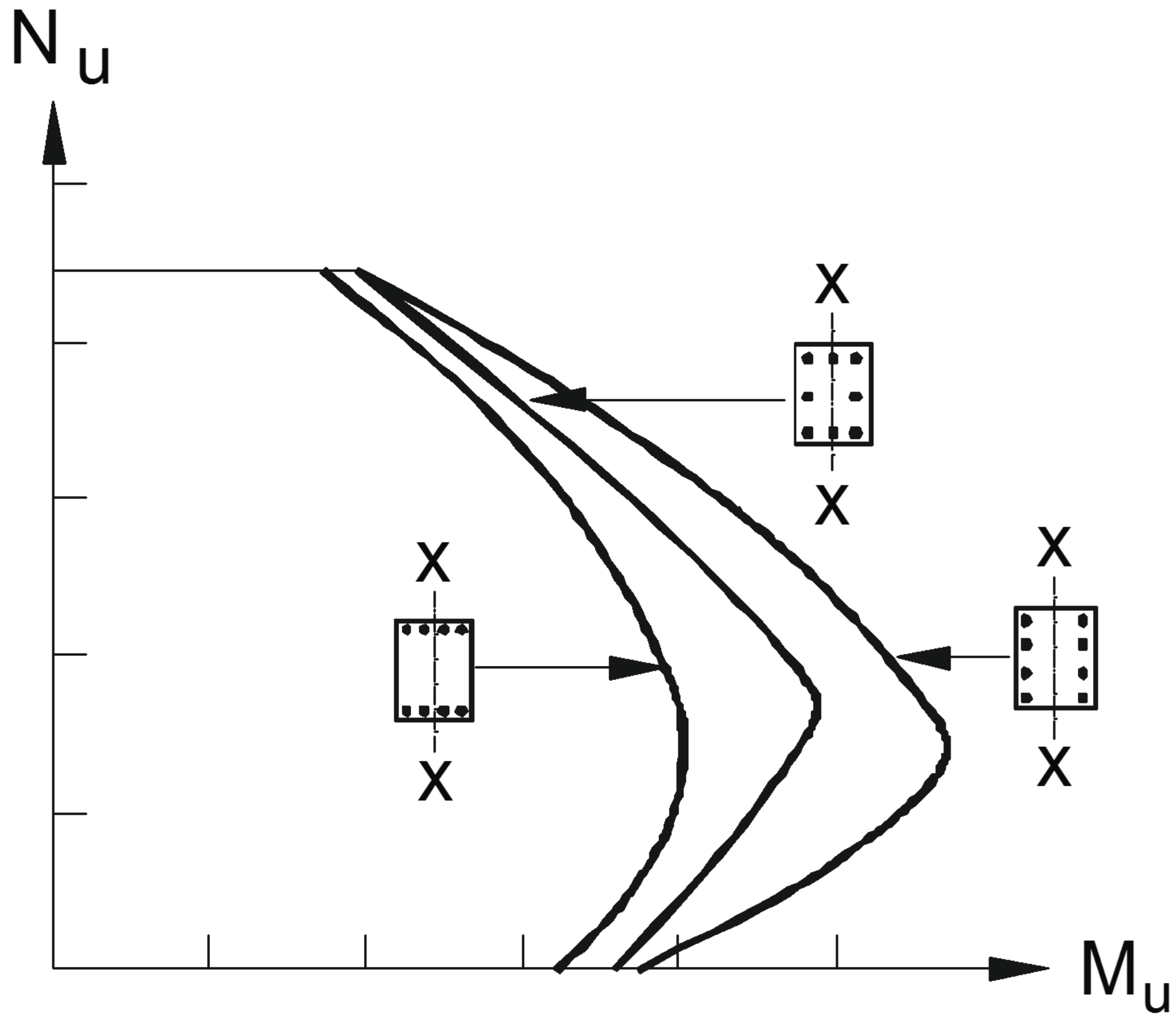
(b) Eccentric load.

(c) Axial load and moment.



Columns

Interaction Diagrams



Rectangular CS with symmetric reinforcing

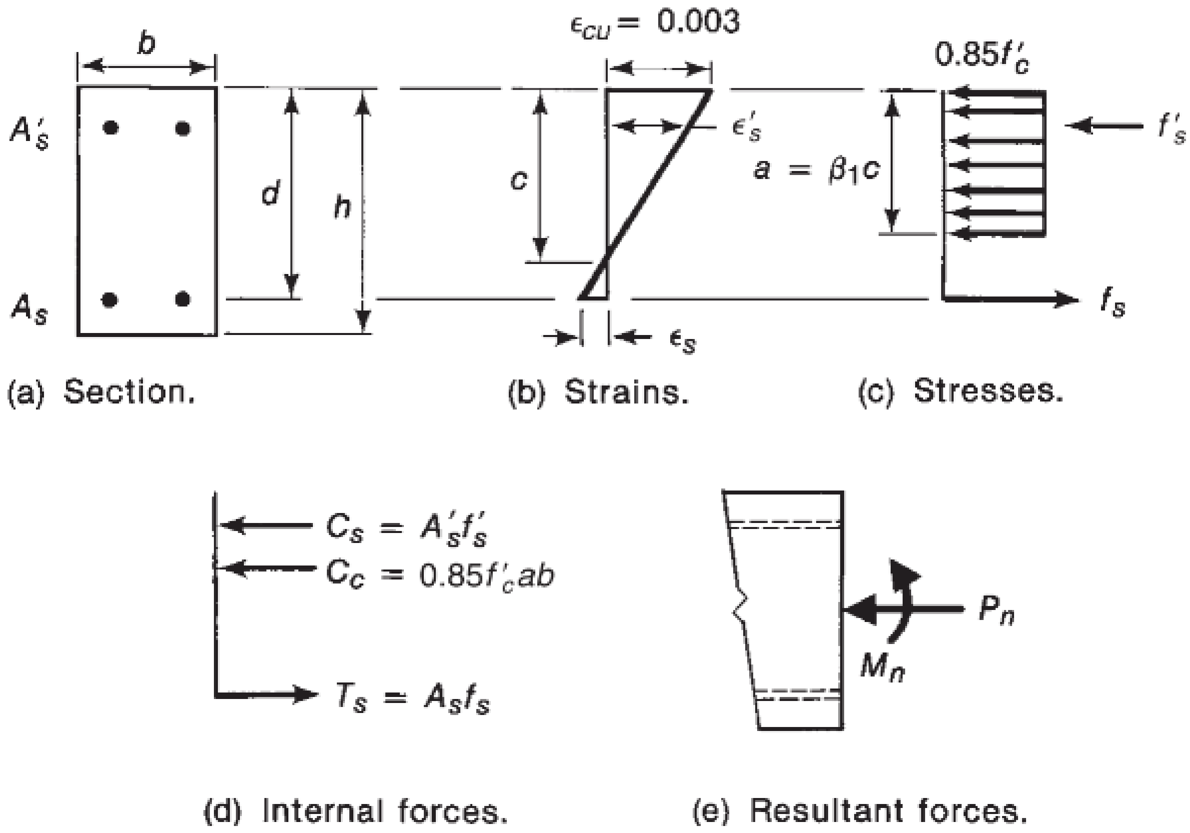


Fig. 11-12
Calculation of P_n and M_n for a given strain distribution.

M+N effect

Geometric reinforcement ratio $$\rho_0 = \frac{A_s}{b.h}$$

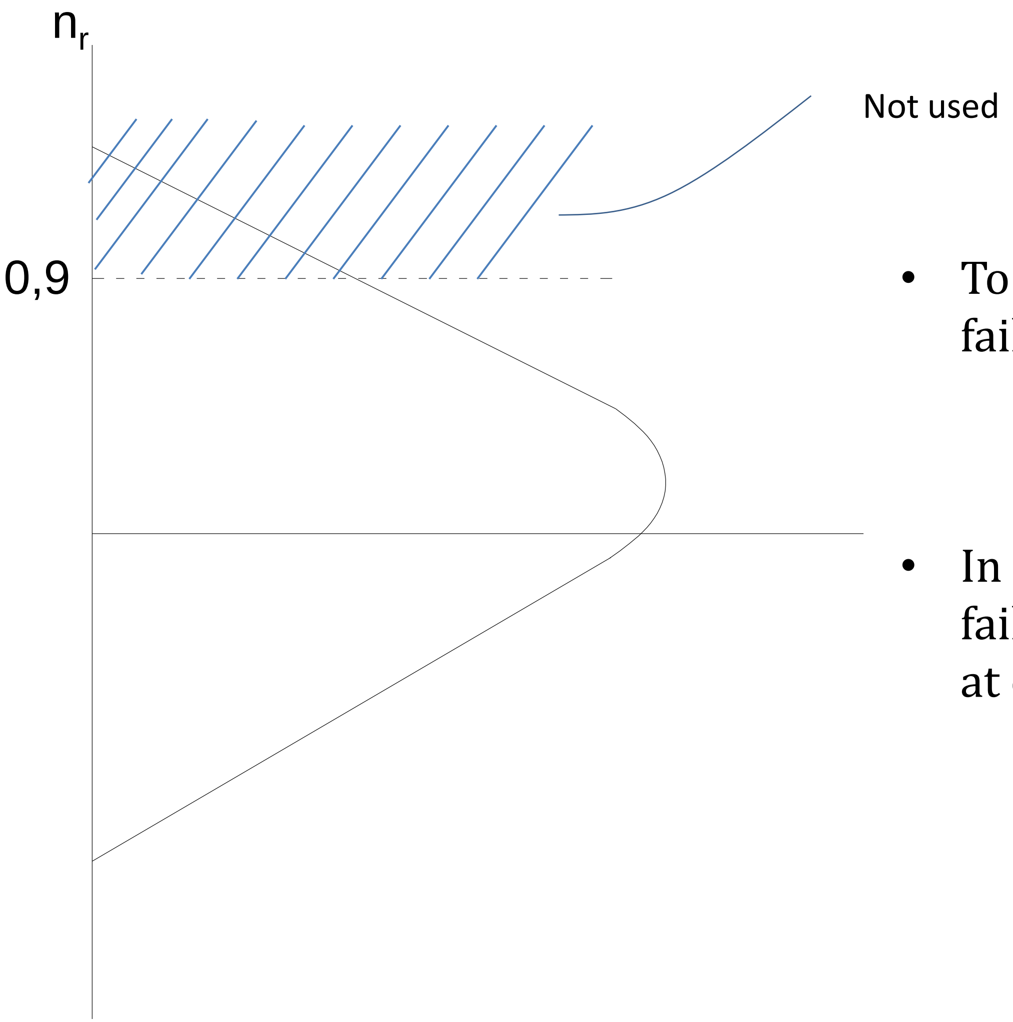
Mechanical reinforcement ratio $$\rho_{0m} = w = \rho_0.\frac{f_{yd}}{f_{cd}} = \frac{A_s}{b.h}.\frac{f_{yd}}{f_{cd}}$$

Dimensionless
Moment $$m_r = \frac{M_r}{b.h^2.fcd}$$

Dimensionless
Normal force $$n_r = \frac{N_r}{b.h.fcd}$$

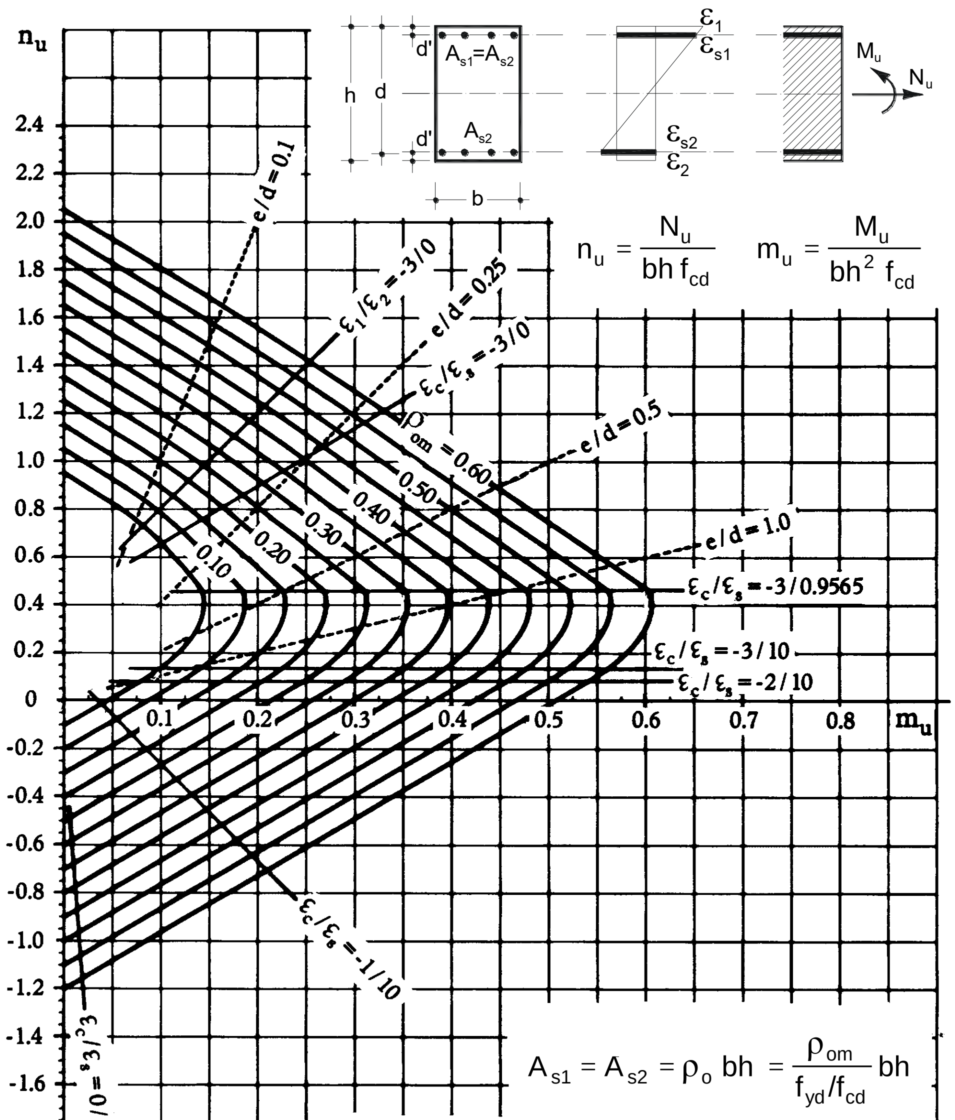
$$A_{s1} = A_{s2} = \rho_0.b.h = \frac{\rho_{0m}}{f_{yd}/fcd}.b.h$$

Interaction diagrams:



- To prevent brittle compression failure
- In the case of a sudden brittle failure; maximum stress occur at each point of CS

Interaction Diagrams



$$n_u = \frac{N_u}{bh\, f_{cd}} \qquad m_u = \frac{M_u}{bh^2\, f_{cd}}$$

$$A_{s1} = A_{s2} = \rho_o\, bh = \frac{\rho_{om}}{f_{yd}/f_{cd}}\, bh$$

Beton	C14	C16	C18	C20	C25	C30	C35	C40
f_{yd}/f_{cd}	20.5	17.9	15.9	14.3	11.5	9.6	8.2	7.2

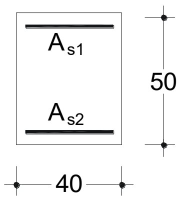
S220 [$f_{yk} = 2200$ kgf/cm^2]

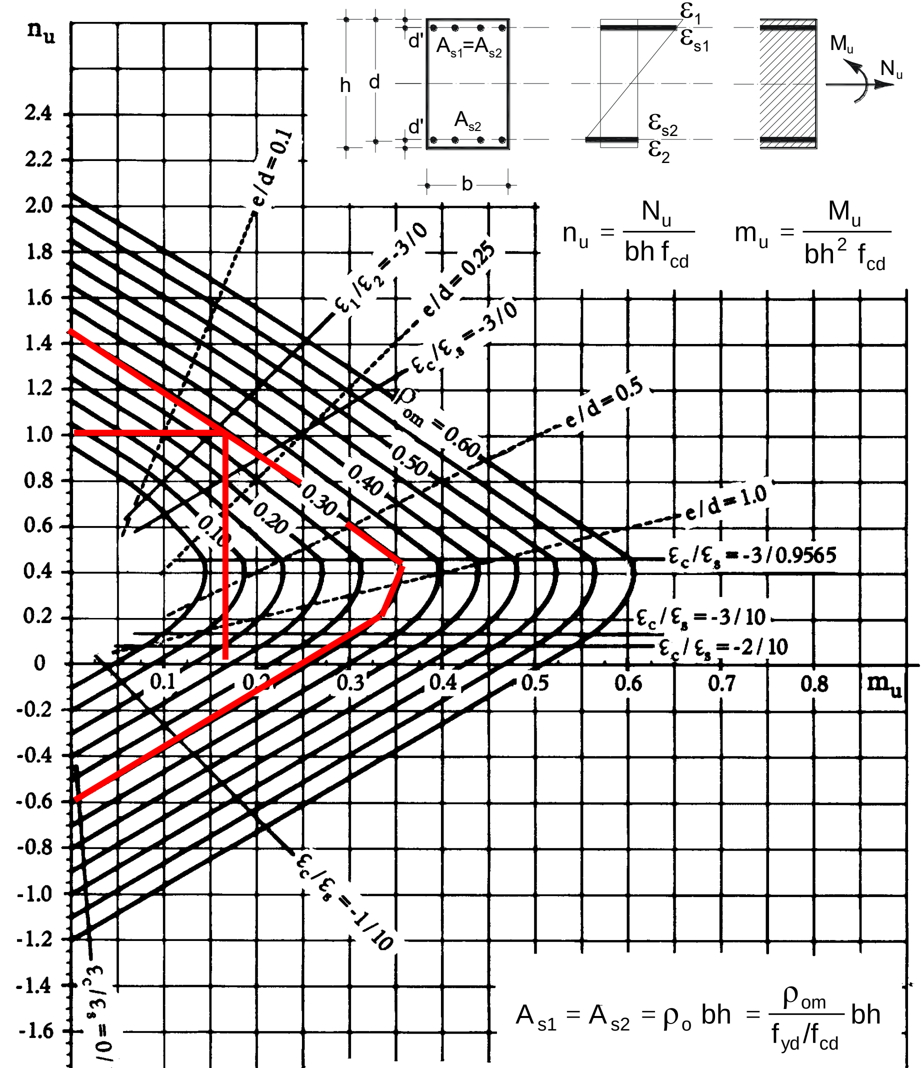
$\xi_1 = d' / h = 0.08$

ÖRNEK:

Boyutları ve malzeme türü verilen bir dikdörtgen kesitli kolonda tasarım kesit zorları M_d = 18 tm ve N_d = – 213.4 t (basınç) olduğuna göre gerekli donatının karşılıklı etki diyagramı ile simetrik olarak belirlenmesi.

Malzeme: C16 / S220





$$n_u = \frac{N_u}{bh\,f_{cd}} \qquad m_u = \frac{M_u}{bh^2\,f_{cd}}$$

$$A_{s1} = A_{s2} = \rho_o\,bh = \frac{\rho_{om}}{f_{yd}/f_{cd}}\,bh$$

Beton	C14	C16	C18	C20	C25	C30	C35	C40
f_{yd}/f_{cd}	20.5	17.9	15.9	14.3	11.5	9.6	8.2	7.2

$$C16 \rightarrow f_{cd} = \frac{160}{1.50} = 106.7 kgf/cm^2$$

$$S220 \rightarrow f_{yd} = \frac{2200}{1.15} = 1913.04 kgf/cm^2$$

C16 ; S220 $\rightarrow f_{yd}/f_{cd} = 17.9$

$$n_u = \frac{213400}{40 \times 50 \times 106.7} = 1.00 \qquad m_u = \frac{18 \times 10^5}{40 \times 50^2 \times 106.7} = 0.17$$

h = 50cm, d' (pas payı) = 4cm

$\xi = d'/h = 0.08$, $\rho_{0m} = 0.30$, $f_{yd} / f_{cd} = 17.9$

$$A_{s1} = A_{s2} = \rho_0 bh = \frac{\rho_{0m}}{f_{yd}/f_{cd}} bh$$

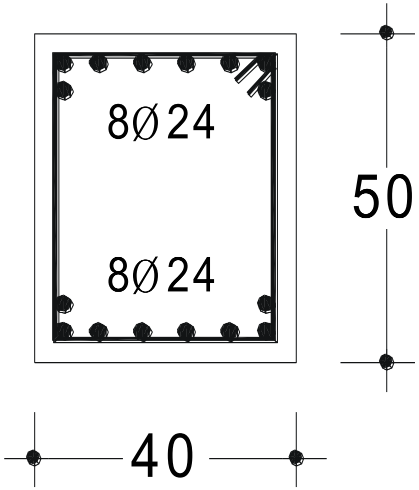
$A_{s1} = A_{s2} = 33.52\ cm^2 \rightarrow 8\phi24$ (36.19 > 33.52 cm²)

Donatı çubukları enkesit alanları

ϕ mm	g kg/m	Çubuk Sayısı 1	2	3	4	5	6	7	8	9	10
8	0.395	0.50	1.01	1.51	2.01	2.51	3.01	3.52	4.02	4.52	5.03
10	0.617	0.79	1.57	2.36	3.14	3.93	4.71	5.50	6.28	7.07	7.85
12	0.888	1.13	2.26	3.39	4.52	5.65	6.79	7.92	9.05	10.18	11.31
14	1.21	1.54	3.08	4.62	6.16	7.70	9.24	10.78	12.32	13.85	15.39
16	1.58	2.01	4.02	6.03	8.04	10.05	12.06	14.07	16.08	18.10	20.11
18	2.00	1.54	5.09	7.63	10.18	12.72	15.26	17.81	20.36	22.90	25.45
20	2.47	3.14	6.28	9.42	12.57	15.71	18.84	21.99	25.14	28.28	31.42
22	2.98	3.80	7.60	11.40	15.27	19.01	22.81	26.61	30.41	34.21	38.01
24	3.55	4.52	9.05	13.57	18.10	22.62	27.14	31.67	36.19	40.72	45.24
26	4.17	5.31	10.62	15.93	21.24	26.55	31.86	37.17	42.47	47.78	53.09
28	4.83	6.16	12.31	18.47	24.63	30.79	36.94	43.10	49.26	55.42	61.58
30	5.55	7.07	14.14	21.21	28.27	35.34	42.41	49.48	56.55	63.62	70.69
32	6.31	8.04	16.06	24.13	32.17	40.21	48.26	56.30	64.34	72.38	80.42
34	7.13	9.08	18.16	27.24	36.32	45.40	54.48	63.56	72.63	81.71	90.79
36	7.99	10.18	20.36	30.54	40.72	50.90	61.07	71.25	81.43	91.61	101.7
38	8.90	11.34	22.68	34.02	45.36	56.70	68.04	79.38	90.73	102.0	113.4
40	9.87	12.57	25.13	37.70	50.26	62.83	75.40	87.96	100.5	113.1	125.6
45	12.48	15.90	31.81	47.71	63.62	79.52	95.43	111.3	12723	143.1	159.0
50	15.41	19.64	39.27	58.91	78.54	98.15	117.8	137.4	157.0	176.7	196.3

…ve çapı ve A_s donatısı [cm²]

								… Çubuk		7 Çubuk		8 Çubuk	
								…w	A_s	b_w	A_s	b_w	A_s
								….4	6.8	24.6	7.9	27.8	9.1
14	8	12.4	4.6	15.8	6.2	19.2	7.7	22.6	9.2	26.0	10.8	29.4	12.3
16	8	13.0	6.0	16.6	8.0	20.0	10.0	23.8	12.1	27.4	14.1	31.0	16.1
18	8	13.6	7.6	17.4	10.2	21.2	12.7	25.0	15.3	28.8	17.8	32.6	20.4
20	8	14.6	9.4	18.6	12.6	22.6	15.7	26.6	18.8	30.6	22.0	34.6	25.1
22	8	15.6	11.4	20.0	15.2	24.4	19.0	28.8	22.8	33.2	26.6	37.6	30.4
24	8	16.6	13.6	21.4	18.1	26.2	22.6	31.0	27.1	35.8	31.7	40.6	36.2
26	10	18.0	15.9	23.2	21.2	28.4	26.5	33.6	31.9	38.8	37.2	44.0	42.5
28	10	19.0	18.5	24.6	24.6	30.2	30.8	35.8	36.9	41.4	43.1	47.0	49.3
30	10	20.0	21.2	26.0	28.3	32.0	35.3	38.0	42.4	44.0	49.5	50.0	56.5
32	12	21.4	24.1	28.0	32.2	34.2	40.2	40.6	48.3	47.0	56.3	53.4	64.3
34	12	22.4	27.2	29.2	36.3	36.0	45.4	42.8	54.5	49.6	63.6	56.4	72.6
36	12	23.4	30.5	30.6	40.7	37.6	50.9	45.0	61.1	52.2	71.3	59.4	81.4
38	14	24.8	34.0	32.4	45.4	40.0	56.7	47.6	68.0	55.2	79.4	62.8	90.7
40	14	25.8	37.7	33.8	50.3	41.8	62.8	49.8	75.4	57.8	88.0	65.8	100



8 adet donati 40cm’e sığmadığı için yandaki şekildeki gibi bir donatı düzenlemesi yapılmıştır.