Chapter 9

Design of Flat Slabs

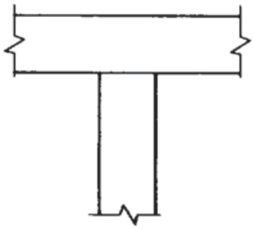
9.0 NOTATION

A	Area of column or area of effective column head
A_{st}	Area of steel in tension
b_e	Effective width of slab for transfer of moment to edge column
C_x	Size of a rectangular column in x-direction
C_y	Size of a rectangular column in y-direction
d	Effective depth of tensile reinforcement
d_h	Depth of column head
f_y	Characteristic yield strength of reinforcement
f_{cu}	Characteristic cube strength of concrete at 28 days
h_c	Effective diameter of column or effective column head
G_k	Characteristic dead load
l_c	Dimension of column in direction of l_h
l_h	Effective dimension of column head
l_x	Shorter span framing onto columns
l_y	Longer span framing onto columns
l_{ho}	Actual dimension of column head
$l_{h,max}$	Maximum dimension of column head taking 45° dispersion
l_1	Centre-to-centre of column in direction of span being considered
l_2	Centre-to-centre of column perpendicular to direction of span being considered
M'	Design limit moment at $h_c/2$
M_t	Moment transferred to column by frame analysis
$M_{t,max}$	Limiting moment between flat slab and edge column
n	Total ultimate load per unit area on flat slab
Q_k	Characteristic live load
V_t	Calculated shear from analysis
V_{eff}	Effective shear at column/slab interface
W_k	Characteristic wind loading
x	Length of side of a perimeter parallel to axis of bending

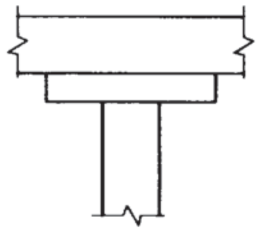
9.1 DEFINITIONS

Flat slab is a reinforced concrete slab supported by columns with, or without, drops. The columns may be with, or without, column heads.

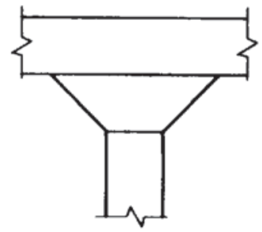
Drop is a local thickening of the slab in the region of the column.



SK 9/1 Flat slab – section.



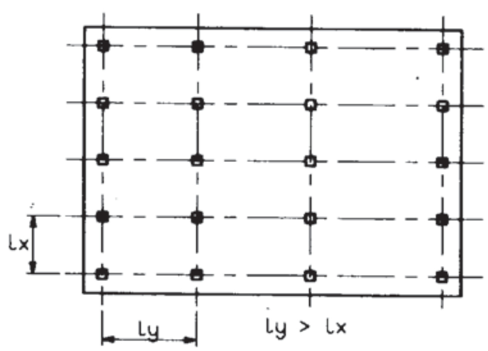
SK 9/2 Flat slab – section.



SK 9/3 Flat slab – section.

Column head is a local enlargement of the column at the junction with the slab.

9.2 ANALYSIS OF FLAT SLABS



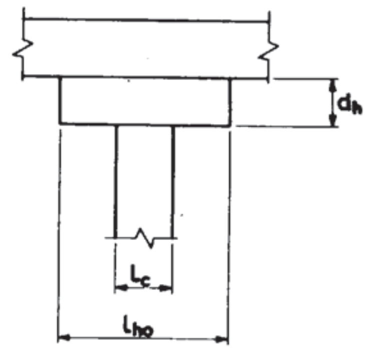
SK 9/4 Typical plan of flat slab.

Flat slabs are usually supported by a rectangular arrangement of columns. The analysis may be carried out by an equivalent frame method or by the use of a finite element computer code. When using the equivalent frame method the ratio of the longer to the shorter span should not exceed 2. The analysis for uniformly distributed vertical load may be carried out by using Tables 9.1 to 9.6.

The properties of the flat slab for analysis are similar to those already discussed for solid slabs in Chapter 3.

9.2.1 Effective dimension of column head

l_h = effective dimension of head
l_{ho} = actual dimension of head
$l_{h,max} = l_c + 2(d_h - 40)$
l_c = dimension of column in direction of l_h
d_h = depth of head
l_h = is taken as the lesser of l_{ho} or $l_{h,max}$



SK 9/5 Flat slab – definitions.

Note: This means that the maximum dimension is limited by a 45° dispersion of column up to 40 mm below the slab.

9.2.2 Effective diameter of a column head

$h_c = (4A/\pi)^{\frac{1}{2}} \leq 0.25 l_x$
h_c = effective diameter of column or column head
A = area of column or area of effective column head as defined by l_h
l_x = shortest span framing onto column

h_c should not be taken greater than one-quarter of shortest span of slab framing into column.

9.2.3 Drops

Drops will be effective in the analysis if the smaller dimension of the drop is at least one-third of the smallest span of surrounding panels.
For the checking of punching shear, this limitation does not apply.

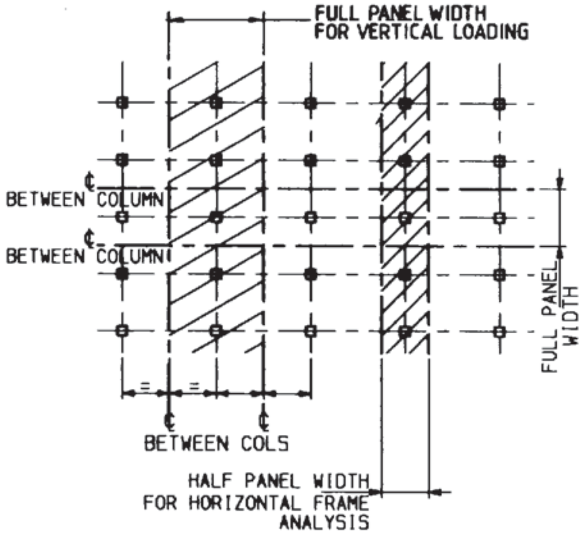
9.2.4 Load combinations for analysis

$LC_1 = 1.4G_k + 1.6Q_k$ on all spans
$LC_2 = 1.4G_k + 1.6Q_k$ on alternate spans and other spans loaded with $1.0G_k$

where G_k = characteristic dead load
Q_k = characteristic live load.

9.2.5 Effective width of slab for analysis

For vertical loading assume full width of panel between columns for frame analysis.
For horizontal loading as a frame assume stiffness of half width of panel.



SK 9/6 Plan of flat slab showing panel widths for analysis.

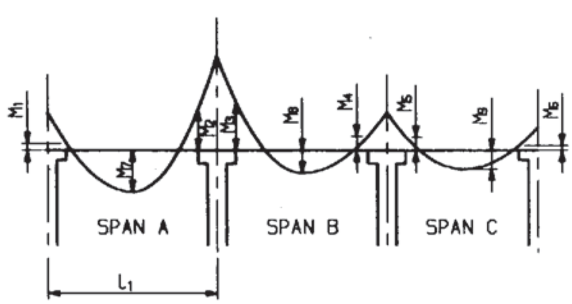
The analysis should be carried out using a computer program or a moment distribution method. The analysis may also be carried out for uniformly distributed vertical loads using Tables 9.1 to 9.6.

The analysis may be carried out using Table 3.13 of BS 8110: Part 1: 1985[1] provided the lateral stability is not dependent on the slab–column connection and loading on the flat slab for the design is based on a single load case, i.e. LC_1, the ratio of Q_k/G_k does not exceed 1.25, Q_k does not exceed 5 kN/m^2, and there are at least three rows of panels.

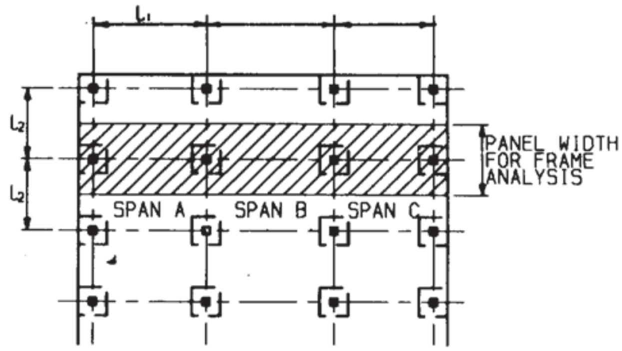
9.3 DESIGN OF FLAT SLABS

The design may be based on the negative moment at $h_c/2$ from the centreline of the column. But this negative moment will have to be modified if the sum of the positive design moment and the average negative design moment is less than the following expression:

$$M' = \left(\frac{nl_2}{8}\right)\left(l_1 - \frac{2h_c}{3}\right)^2$$



SK 9/7 Negative moment limitation for flat slabs – section.



SK 9/8 Typical plan of flat slab – negative moment limitation.

where l_1 = centre-to-centre of column in direction of span being considered

l_2 = centre-to-centre of column perpendicular to direction of span being considered

n = total ultimate load on slab (kN/m^2).

To give an example:

For Span A

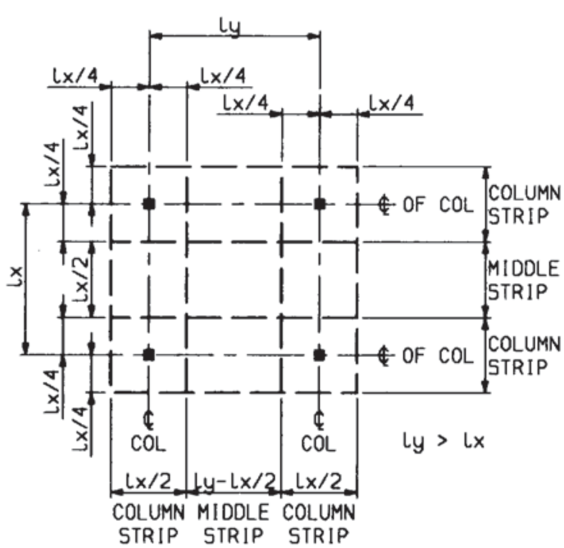
$$0.5\ (M_1 + M_2) + M_7 \geq M'$$

For Span B

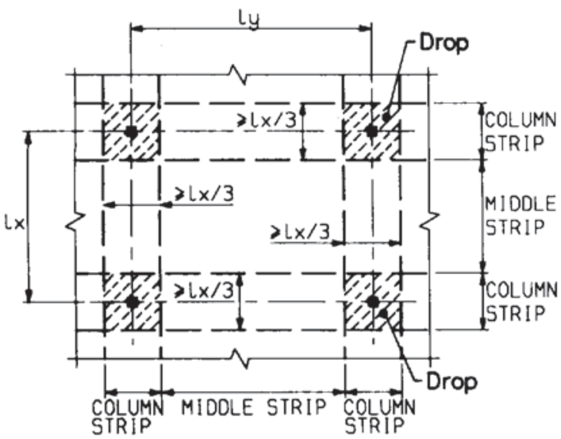
$$0.5(M_3 + M_4) + M_7 \geq M'$$

Increase negative moments M_1, M_2, M_3, etc. until these conditions are satisfied.

9.3.1 Division of panels



SK 9/9 Flat slab – division of strips.



SK 9/10 Flat slab – division of strips.

Panels are divided into column strips and middle strips as shown.
For slab without drop the column strip is $l_x/4$ wide on either side of the centreline of column, where l_x is the shorter span.
For slab with drop the column strip is the size of the drop. Ignore drop if the size of the drop is less than $l_x/3$.

9.3.2 Division of moments between columns and middle strips

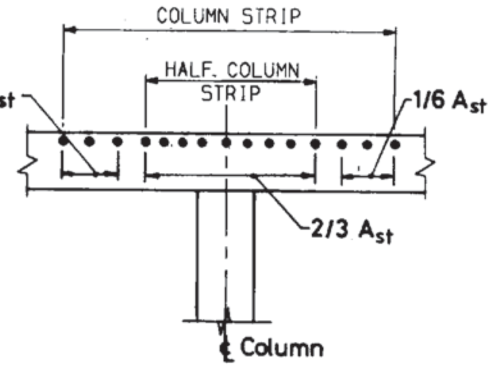
The moments obtained from analysis of frames should be divided as follows (these percentages are for slabs without drops):

	Column strip	Middle strip
Negative	75%	25%
Positive	55%	45%

Note: Where column drops are used and column strips are determined from the width of the drop, it may so happen that the middle strip is bigger than the middle strip in a slab without drop. In that case the moments in the middle strip will be proportionately increased and those in the column strip decreased to keep the total positive and negative moment unchanged.

9.3.3 Design of flat slab panels

The design is similar to the design of slabs and the worked examples are in Chapter 3.

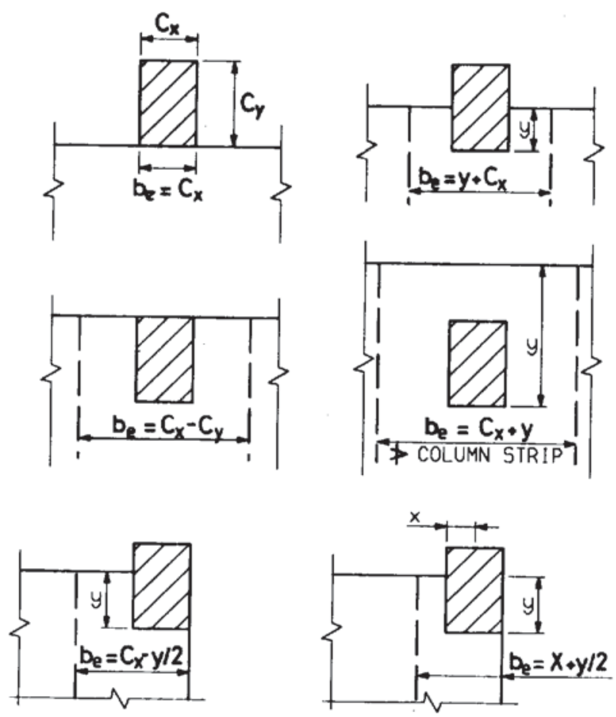


SK 9/11 Detailing of reinforcement in flat slabs.

Internal panels and edge panels

Two-thirds of the negative support reinforcement in the column strip should be placed in half the width of the column strip centred over the column.

9.3.4 Moment connection to edge column



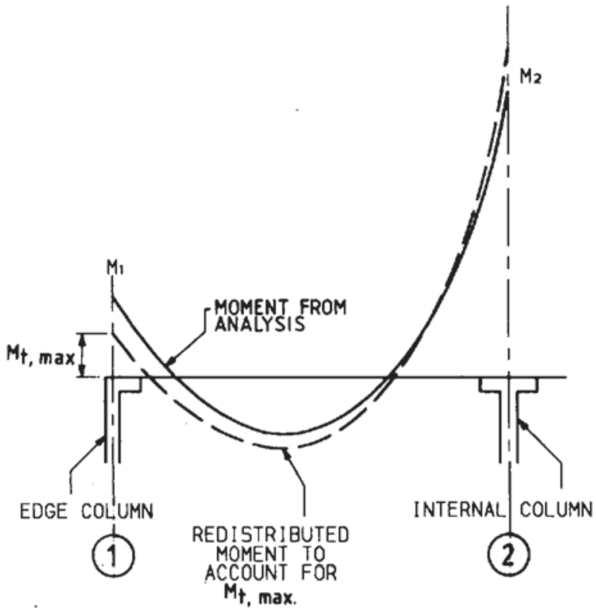
SK 9/12 Effective width of slab for moment connection to edge column.

See sketches above to find effective width of slab b_e for transfer of moment between flat slab and edge column. This moment should be limited to

$$M_{t,max} = 0.15 b_e d^2 f_{cu}$$

where d = effective depth of top reinforcement in column strip.

The moment $M_{t,max}$ should not be less than half the design moment from an equivalent frame analysis or 70% of the design moment from a grillage or finite element analysis. The structural arrangement may be changed if $M_{t,max}$ does not satisfy the above condition.



SK 9/13 Insufficient moment transfer capacity at edge column.

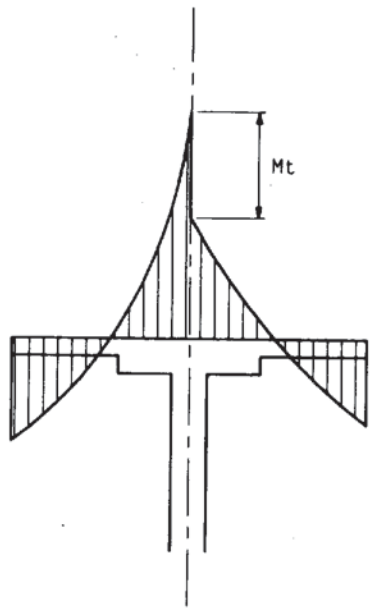
Where the design moment is larger than $M_{t,max}$, redistribution of moment may be carried out to reduce the design moment to $M_{t,max}$. Otherwise, to transfer moments in excess of $M_{t,max}$ to edge column, the edge of the slab should be reinforced by an edge beam or an edge strip. The edge beam will be designed to carry the additional moment by torsion to the column.

9.3.5 Shear in flat slabs

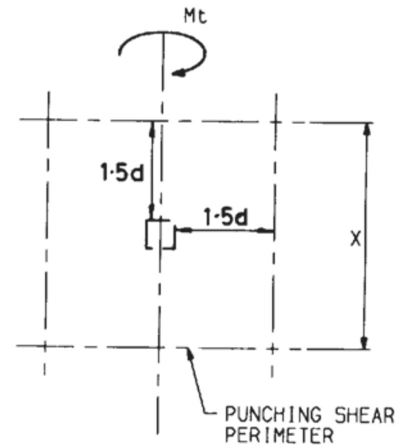
Punching shear around columns should be checked according to Step 7 of Section 3.3. The shear to be considered for the punching shear calculation is increased from the calculated column shear by an amount dependent on the moment transferred to the column by frame action.

For internal column connections,

$$V_{eff} = V_t\left(1 + \frac{1.5M_t}{V_t x}\right)$$



SK 9/14 Moment diagram at an internal column of a flat slab.



SK 9/15 Definition of dimension x.

where V_t = calculated shear from analysis
M_t = moment transferred to column by frame analysis
x = length of side of perimeter considered parallel to axis of bending.

Alternatively,

$V_{eff} = 1.15V_t$ for simplicity

For corner column connections,

$V_{eff} = 1.25V_t$

For edge column connections,

$V_{eff} = 1.25V_t$ for bending about axis parallel to free edge

$V_{eff} = V_t\left(1.25 + \frac{1.5M_t}{V_t x}\right)$ for bending about axis perpendicular to free edge

Alternatively,

$V_{eff} = 1.4V_t$

The moment M_t may be reduced by 30% where the equivalent frame analysis is used and both load cases LC_1 and LC_2 have been considered. The shear reinforcement will be calculated according to Step 7 of Section 3.3.

9.4 STEP-BY-STEP DESIGN PROCEDURE FOR FLAT SLABS

Step 1 Carry out analysis as in Section 9.2.

Step 2 Find moment connection to edge column as per Section 9.3.4 and redistribute moments if necessary.

Step 3 Draw bending moment diagrams and calculate moments at $h_c/2$ following Section 9.3.

Step 4 Check limitation of negative design moments following Section 9.3.

Step 5 Carry out division of panels as in Section 9.3.1.

Step 6 Divide moments between column strips and middle strips as per Section 9.3.2.

Step 7 Determine cover to reinforcement (see Step 3 of Section 3.3).

Step 8 Carry out design for flexure as per Step 4 of Section 3.3.

Step 9 Distribute reinforcement as per Section 9.3.3.

Step 10 ***Check punching shear stress***
Follow Step 7 of Section 3.3.

Step 11 ***Check span/effective depth ratio***
Follow Step 11 of Section 3.3 for slabs with drops. For slabs without drops follow the same step but multiply l_e/d from Table 11.3 by 0.9.

Step 12 ***Curtailment of bars***
Follow Step 12 of Section 3.3.

Step 13 ***Spacing of bars***
Follow Step 13 of Section 3.3.

Step 14 ***Check early thermal cracking***
Follow Step 14 of Section 3.3.

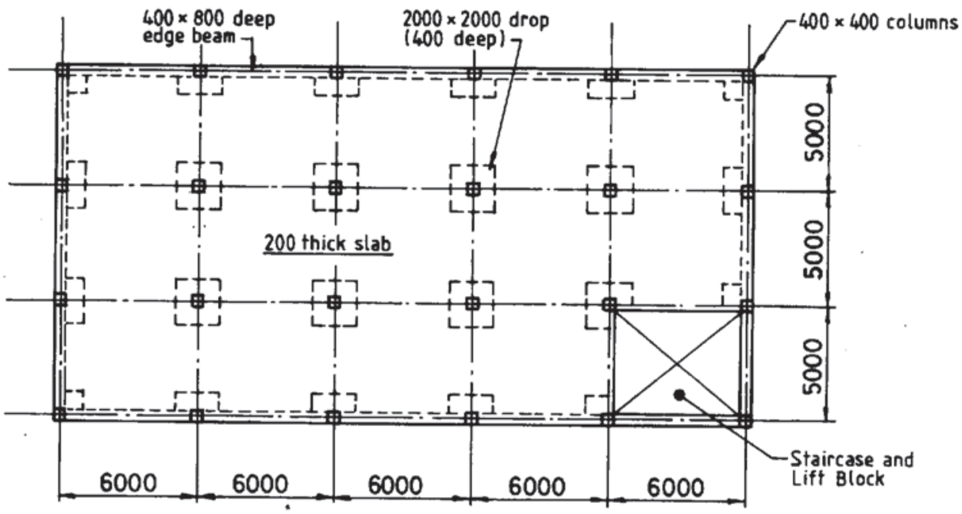
Step 15 ***Calculate minimum reinforcement***
Follow Step 9 and Step 15 of Section 3.3.

Step 16 ***Calculate flexural crack width***
Follow Step 16 of Section 3.3.

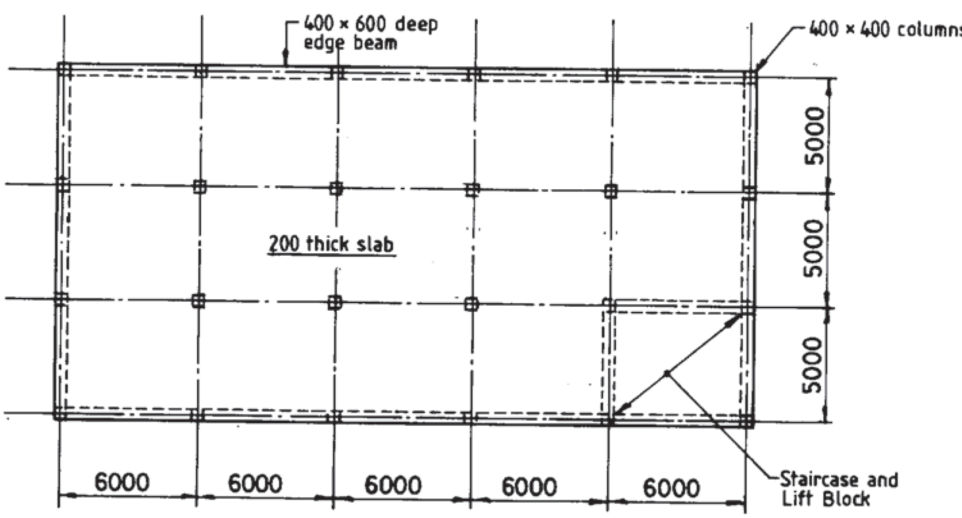
Step 17 ***Design of connections***
Follow Chapter 11.

9.5 WORKED EXAMPLE

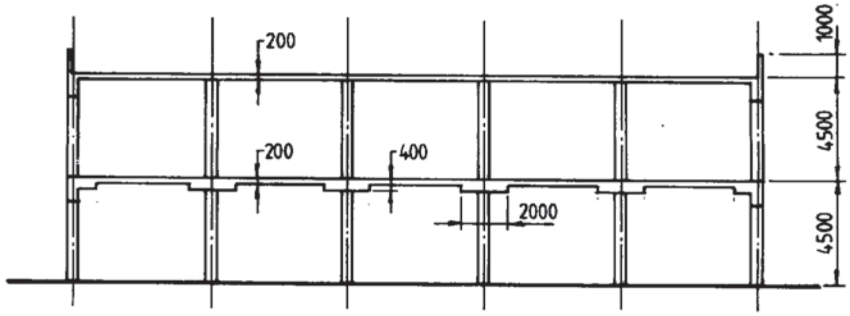
Example 9.1 Flat slab construction for a sports hall



SK 9/16 Plan on first floor.



SK 9/17 Plan on roof.



SK 9/18 Section through building.

Two-storey building plan size: 15 m × 30 m
Column grid: 5 m × 6 m
Column size: 400 × 400
Height of building = 10 m overall
Topography factor, $S_1 = 1.0$
Ground roughness factor, $S_2 = 0.95$
Statistical factor for wind, $S_3 = 1.0$
Basic wind speed = 42 m/s = V
Design wind speed = $S_1 S_2 S_3\ V = 40$ m/s
$q = kV_s^2 = 1\,\text{kN/m}^2$
$C_{pe} = +0.7$ and -0.3
$C_{pi} = -0.3$ (four faces equally permeable)
The above wind pressure coefficients are obtained from CP3: Chapter V *Wind loads*.[14]
Live load on roof = 1.5 kN/m²
Live load on floor = 5 kN/m²
Floor slab has 2000 × 2000 drop at columns
Thickness of roof and floor slab = 200 mm
Thickness at drop of floor slab = 400 mm
Continuous perimeter edge beam 400 wide × 800 deep
Centre-to-centre height of floor = 4.5 m

Step 1 ***Carry out analysis***

Only one frame in the short direction of the building will be analysed. Column head has not been used.

Effective diameter of column, $h_c = \left(\frac{4A}{\pi}\right)^{\frac{1}{2}} \leq 0.25 l_x$

$A = 400 \times 400 = 160\,000\,\text{mm}^2$

$$\therefore \quad h_c = \left(\frac{4 \times 160\,000}{\pi}\right)^{\frac{1}{2}}$$

$$= 451\,\text{mm} < 0.25 l_x$$

$$l_x = 5000\,\text{mm}$$

$$0.25 l_x = 1250\,\text{mm}$$

Drop of 2000 mm in floor slab is greater than $l_x/3 = 1667$ mm.
Drop will be effective in the distribution of moment.

Loading

Frames in short direction are 6 m apart.

Roof slab

G_k = characteristic dead load = $0.2 \times 25 = 5\,\text{kN/m}^2$

$Q_k = 1.5\,\text{kN/m}^2$

$LC_1 = 1.4G_k + 1.6Q_k = 9.4\,\text{kN/m}^2 = 56.4\,\text{kN/m}$

$LC_2 = 9.4\,\text{kN/m}^2$ and $5\,\text{kN/m}^2$ on alternate spans
or $LC_2 = 56.4\,\text{kN/m}$ and $30\,\text{kN/m}$ on alternate spans

Floor slab

$G_k = 5\,\text{kN/m}^2$ at slab without drop
$= 10\,\text{kN/m}^2$ at slab with drop (area $2\,\text{m} \times 2\,\text{m}$)
$= 30\,\text{kN/m}$ or $30 + 5 \times 2 = 40\,\text{kN/m}$

$Q_k = 5\,\text{kN/m}^2$
$= 30\,\text{kN/m}$

$LC_1 = 1.4G_k + 1.6Q_k$

LC_2 = alternate spans loaded with LC_1 and dead load only

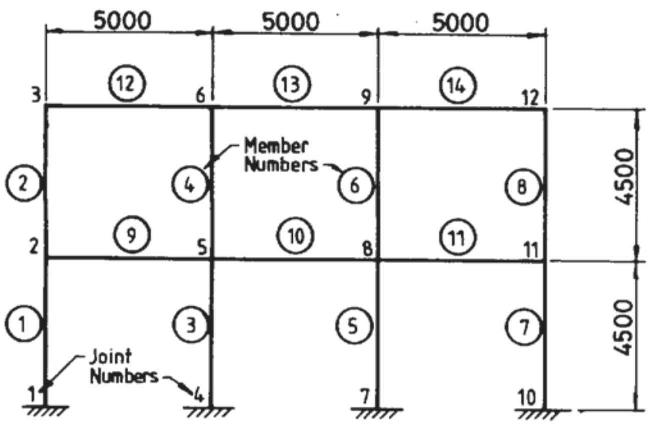
Columns

Horizontal load on columns is due to wind load at the rate of $1\,\text{kN/m}^2$ which is equivalent to 6 kN/m on the column. The wind loading analysis will be carried out separately and combined later with the vertical loading because the stiffness of the slab to resist horizontal loading is half of that to resist vertical loading.
Load cases with wind load W_k are as follows:

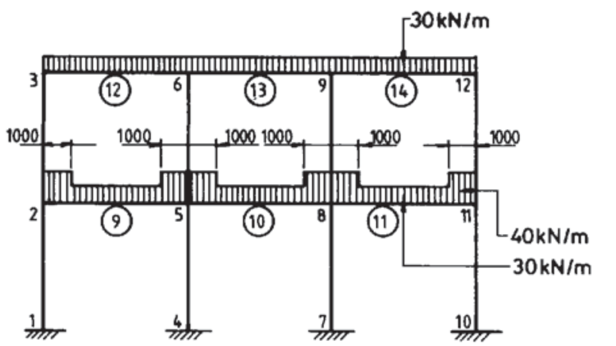
$LC_3 = 1.4G_k + 1.4W_k$
$LC_4 = 1.2G_k + 1.2Q_k + 1.2W_k$

Frame analysis using a computer software

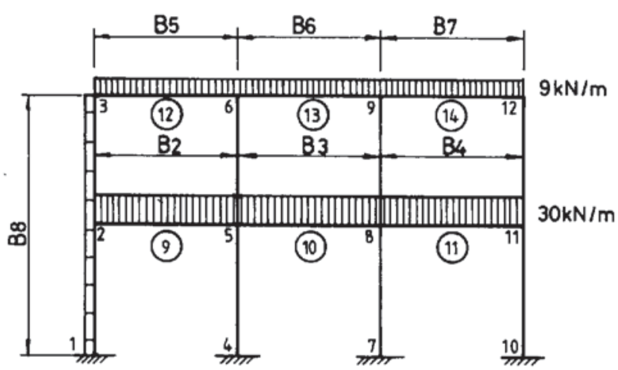
E = Young's modulus = $28 \times 10^6\,\text{kN/m}^2$
12 joints 14 members
Joints 1, 4, 7 and 10 rigidly fixed.
Column size 400×400
Slab size 6000×200 (deep)
Load cases:
B_1 – dead load
B_2 to B_7 – live loads on members 9 to 14 respectively
B_8 – wind load



SK 9/19 Frame diagram for analysis.



SK 9/20 Dead load on frame (B_1).



SK 9/21 Basic live loads B_2 to B_8.

Combinations:
$C_1 = 1.4B_1 + 1.6(B_2 + B_3 + B_4 + B_5 + B_6 + B_7)$
$C_2 = 1.4B_1 + 1.6(B_2 + B_4 + B_5 + B_7)$
$C_3 = 1.4B_1 + 1.6(B_3 + B_6)$
$C_4 = 1.4B_1 + 1.6(B_2 + B_3 + B_4)$

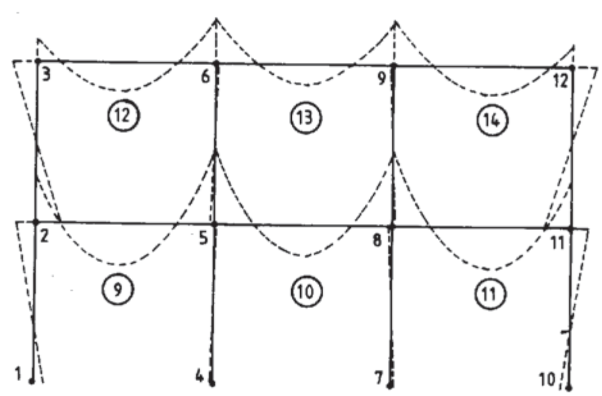
Output from analysis
Envelope of load cases (vertical loads)
Elastic analysis – no redistribution

Floor slab	Member 9		
Joint	Maximum BM	Shear	Combination
2	130.80	228.5	C_2
5	215.5	258.2	C_4
Midspan	131.2	–	C_2

Floor slab	Member 10		
Joint	Maximum BM	Shear	Combination
5	199.9	239.0	C_4
8	199.9	239.0	C_4
Midspan	112.5	–	C_3

Roof slab	Member 12		
Joint	Maximum BM	Shear	Combination
3	67.8	130.0	C_2
6	129.0	153.5	C_1
Midspan	82.0	–	C_2

Roof slab	Member 13		
Joint	Maximum BM	Shear	Combination
6	121.3	141.0	C_1
9	121.3	141.0	C_1
Midspan	60.4	–	C_3



SK 9/22 Combination C_1 – bending moment diagram.

Envelope of load cases (vertical + horizontal loads) (The analysis of horizontal load is carried out with half stiffness of slab)
Elastic analysis – no redistribution
Combinations:
$C_5 = 1.4B_1 + 1.4B_8$
$C_6 = 1.2B_1 + 1.2(B_2 + B_3 + B_4 + B_5 + B_6 + B_7) + 1.2B_8$

Floor slab	Member 9		
Joint	Maximum BM	Shear	Combination
2	121.9	185.9	C_6
5	191.4	215.2	C_6
Midspan	95.8	—	C_6
Floor slab	Member 10		
Joint	Maximum BM	Shear	Combination
5	176.0	198.6	C_6
8	176.0	198.6	C_6
Midspan	71.4	—	C_6
Roof slab	Member 12		
Joint	Maximum BM	Shear	Combination
3	57.0	108	C_6
6	112.2	128.9	C_6
Midspan	67.1	—	C_6
Roof slab	Member 13		
Joint	Maximum BM	Shear	Combination
6	108.1	119.8	C_6
9	108.1	119.8	C_6
Midspan	45.4	—	C_6

Carry out redistribution of moment:

Maximum bending moment at joint $5 = 215.5$ kNm

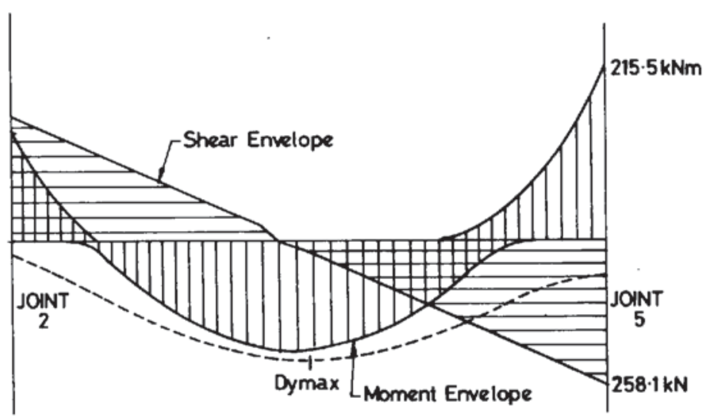
Assume 20% redistribution.

Set plastic moment capacity at joint $5 = 0.8 \times 215.5 = 172.4$ kNm

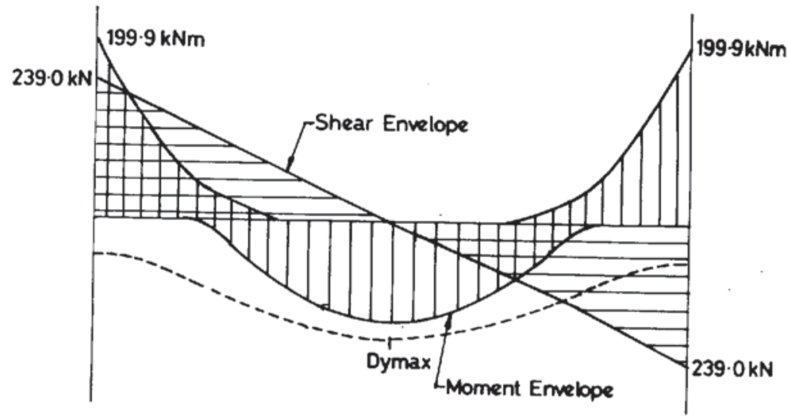
Similarly

maximum bending moment at joint $6 = 129$ kNm

Assume 20% redistribution.



SK 9/23 Shear and moment envelope for member 9.



SK 9/24 Shear and moment envelope for member 10.

Set plastic moment capacity at joint $6 = 0.8 \times 129 = 103.2\,\text{kNm}$

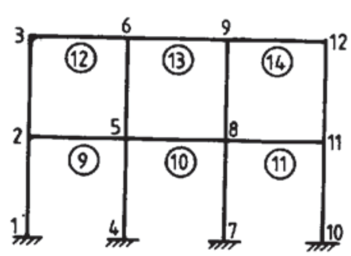
The following steps of reanalysis of frame are carried out:

Step 1: For one combination at a time increase live load on span until plastic moment is reached at a joint in a member. Plastic moment capacity of members on first floor is 172.4 kNm and member on roof is 103.2 kNm.
Step 2: Release joint where plastic moment is reached and increase loading until plastic moment capacity is reached at another joint.
Step 3: Progressively release joints and increase live load until full complement of live load is on structure.
Step 4: Find cumulative effect of all incremental live load on structure.

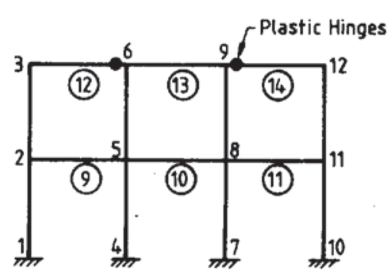
The following tables become useful if a non-linear finite element computer package is not available.

Frame types:

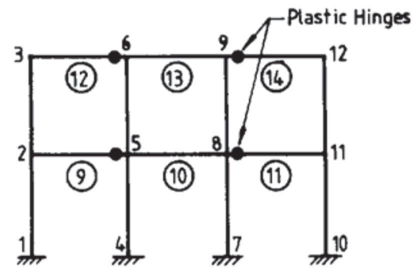
F_1 = no member end releases
F_2 = member in F_1 nos 12 and 14 ends released at joints 6 and 9
F_3 = member in F_2 nos 9 and 11 ends released at joints 5 and 8
F_4 = member in F_3 no. 13 ends released at joints 6 and 9
F_5 = member in F_4 no. 10 ends released at joints 5 and 8



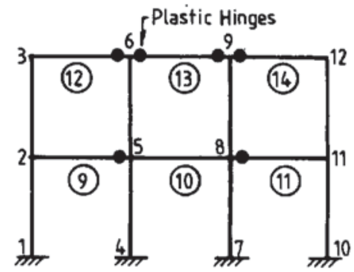
SK 9/25 Frame type F_1.



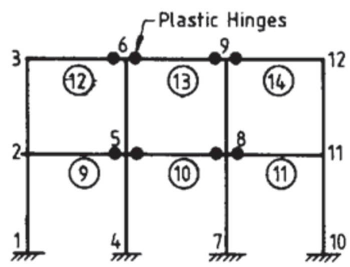
SK 9/26 Frame type F_2.



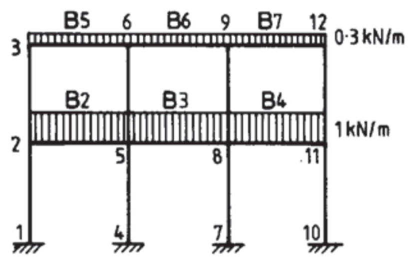
SK 9/27 Frame type F_3.



SK 9/28 Frame type F_4.



SK 9/29 Frame type F_5.



SK 9/30 Unit live load on frame.

Combination $C_7 = 1.4 \times$ dead load or $1.4B_1$

The method is illustrated for combinations C_1' and C_2' only.

$$C_1' = 1.6(B_2' + B_3' + B_4' + B_5' + B_6' + B_7')$$
$$C_2' = 1.6(B_2' + B_4' + B_5' + B_7')$$

$$B_2', B_3', B_4' = 1\,\text{kN/m}$$
$$B_5', B_6', B_7' = 0.3\,\text{kN/m}$$

Combination	Frame type	Member end bending moments (kNm)							
		Member 9		Member 10		Member 12		Member 13	
		Joint 2	Joint 5	Joint 5	Joint 8	Joint 3	Joint 6	Joint 6	Joint 9
C_7	F_1	62.7	102.5	95.5	95.5	44.7	97.5	91.6	91.6
C_1'	F_1	2.0	3.7	3.4	3.4	0.7	1.1	1.0	1.0
C_1'	F_2	2.1	3.6	3.5	3.5	0.9	0	0.5	0.5
C_1'	F_3	3.1	0	2.4	2.4	1.1	0	0.8	0.8
C_1'	F_4	3.1	0	2.2	2.2	1.1	0	0	0
C_1'	F_5	3.1	0	0	0	1.1	0	0	0
C_2'	F_1	2.3	2.7	0.8	0.8	0.8	0.8	0.2	0.2
C_2'	F_2	2.3	2.6	0.8	0.8	0.9	0	−0.2	−0.2
C_2'	F_3	3.1	0	0	0	1.0	0	0	0
C_2'	F_4	3.1	0	0	0	1.0	0	0	0
C_2'	F_5	3.1	0	0	0	1.0	0	0	0

$C_1' = 1$ unit of live load combination in combination C_1 i.e. $C_1' = 1\,\text{kN/m}$ of B_2, B_3 and B_4 and 9/30 kN/m of B_5, B_6 and B_7.
Full compliment of B_2, B_3 and B_4 is 30 kN/m and of B_5, B_6 and B_7 is 9 kN/m.

Plastic moment at joint 5 is fixed at 172 kNm
Plastic moment at joint 6 is fixed at 103.2 kNm
Dead load moment at joint 6 = 97.5 kNm

Each unit of combination C_1 produces 1.1 kNm at joint 6 for frame type F_1. Therefore

units of live load in combination C_1 required to form first plastic hinges at joint 6 and joint 9 in members 12 and 14

$$= \frac{103.2 - 97.5}{1.1} = 5 \text{ units, say}$$

Frame type F_2 has joints released at joints 6 and 9 for members 12 and 14. After 5 units of combination C_1 the bending moments at joints are as follows:

Frame type F_1

Member 9	Joint 2	62.7 + 5 × 2.0 = 72.7 kNm	
	Joint 5	102.5 + 5 × 3.7 = 121.0 kNm	
Member 10	Joint 5	95.5 + 5 × 3.4 = 112.5 kNm	
Member 12	Joint 3	44.7 + 5 × 0.7 = 48.2 kNm	
	Joint 6	97.5 + 5 × 1.1 = 103 kNm	*plastic
Member 13	Joint 6	91.6 + 5 × 1.0 = 96.6 kNm	

Units of live load in combination C_1 to form second plastic hinges at joints 5 and 8 in members 9 and 11

$$= \frac{172 - 121}{3.6} = 14 \text{ units of combination } C_1$$

Total number of units of C_1 to cause plastic hinges at joints 5 and 8 in members 9 and 11 is 19.
After 19 units of combination C_1, the bending moments at joints are as follows:

Frame type F_2

Member 9	Joint 2	72.7 + 14 × 2.1 = 102.1 kNm	
	Joint 5	121 + 14 × 3.6 = 171.4 kNm	*plastic
Member 10	Joint 5	112.5 + 14 × 3.5 = 161.5 kNm	
Member 12	Joint 3	48.2 + 14 × 0.9 = 60.8 kNm	
	Joint 6	plastic = 103 kNm	*plastic
Member 13	Joint 6	96.6 + 14 × 0.5 = 103.6 kNm	*plastic

Joint 5 of member 9 and joint 6 of member 13 have gone plastic simultaneously at 19 units of combination C_1. Therefore *frame type F_3 is not considered*.

After 24 units of combination C_1, the bending moments at joints are as follows:

Frame type F_4

Member 9	Joint 2	102.1 + 5 × 3.1	= 117.6 kNm	
	Joint 5	plastic	= 171.4 kNm	*plastic
Member 10	Joint 5	161.5 + 5 × 2.2	= 172.5 kNm	*plastic
Member 12	Joint 3	60.8 + 5 × 1.1	= 66.3 kNm	
	Joint 6	plastic	= 103 kNm	*plastic
Member 13	Joint 6	plastic	= 103.6 kNm	*plastic

Frame type F_5

After 30 units of combination C_1, the bending moments at joints are as follows:

Member 9	Joint 2	117.6 + 6 × 3.1	= 136.2 kNm
	Joint 5		= 171.4 kNm
Member 10	Joint 5		= 172.5 kNm
Member 12	Joint 3	66.3 + 6 × 1.1	= 72.9 kNm
	Joint 6		= 103.0 kNm
Member 13	Joint 6		= 103.6 kNm

Formula for calculating midspan bending moment and shear

C_7 + 5 units of C_1' (F_1) + 14 units of C_1' (F_2) + 5 units of C_1' (F_4) + 6 units of C_1' (F_5)

Member 9 – combination C_1 (20% redistribution)

Midspan moment = 56.3 + 2.2 × 5 + 2.2 × 14 + 5 × 3.5 + 6 × 3.5 = 136.6 kNm

End shear, joint 2 = 111.1 + 3.7 × 5 + 3.7 × 14 + 4.6 × 5 + 4.6 × 6 = 232.0 kN

End shear, joint 5 = 126.9 + 4.3 × 5 + 4.3 × 14 + 3.4 × 5 + 3.4 × 6 = 246.0 kN

Similarly

Member 10 – combination C_1 (20% redistribution)

Midspan moment = 115.7 kNm
End shear = 239 kN

Member 12 – combination C_1 (20% redistribution)

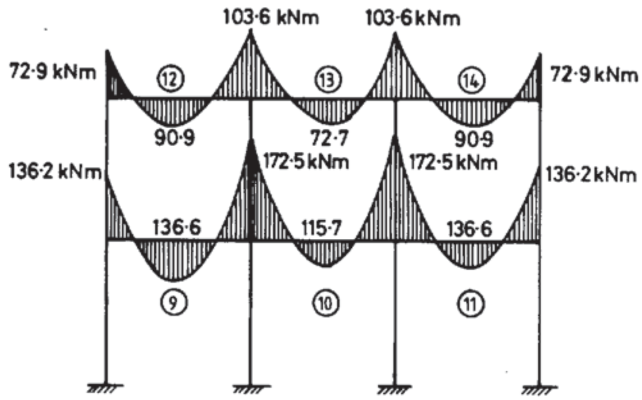
Midspan moment = 90.9 kNm
End shear 3 = 134.9 kN
End shear 6 = 147.1 kN

Member 13 – combination C_1 (20% redistribution)

Midspan moment = 72.7 kNm
End shear 6 = 141 kN

From computer output:
Member midspan moments (kNm) and shears (kN) for various frame types under unit loading.

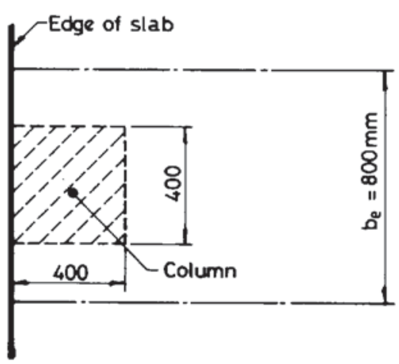
Combination	Frame type	Member 9			Member 10			Member 12			Member 13		
		Midspan moment	Shear 2	Shear 5	Midspan moment	Shear 5	Shear 8	Midspan moment	Shear 3	Shear 6	Midspan moment	Shear 6	Shear 9
C_7	F_1	56.3	111.1	126.9	42.7	119.0	119.0	61.5	94.4	115.6	39.7	105.0	105.0
C_1'	F_1	2.2	3.7	4.3	1.6	4.0	4.0	0.6	1.1	1.3	0.5	1.2	1.2
C_1'	F_2	2.2	3.7	4.3	1.5	4.0	4.0	1.1	1.4	1.0	1.0	1.2	1.2
C_1'	F_3	3.5	4.6	3.4	2.6	4.0	4.0	1.0	1.4	1.0	0.7	1.2	1.2
C_1'	F_4	3.5	4.6	3.4	2.8	4.0	4.0	1.0	1.4	1.0	1.5	1.2	1.2
C_1'	F_5	3.5	4.6	3.4	5.0	4.0	4.0	1.0	1.4	1.0	1.5	1.2	1.2



SK 9/31 Bending moment diagram combination C_1 (20% redistribution).

Note: Only one combination C_1 has been fully analysed to demonstrate the procedure for redistribution of moments in a frame structure. In practice all combinations of loads should be similarly processed to get an envelope of moments and shears. For all combinations of loads the plastic hinges will form at the same moment, i.e. 172 kNm at first floor level and 103.2 kNm at roof level.

Step 2 Check moment connection to edge column



SK 9/32 Effective width of slab for moment transfer.

$M_{t,\ max} = 0.15 b_e d^2 f_{cu}$

$b_e = C_x + C_y = 400 + 400 = 800\,\text{mm}$

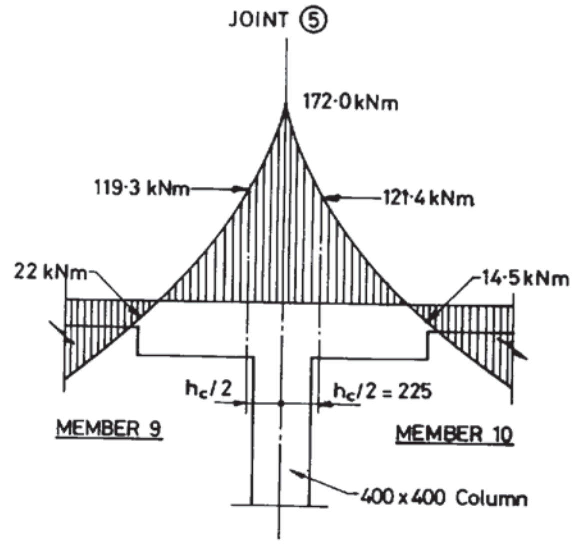
$d = 175\,\text{mm}$ assumed

$f_{cu} = 40\,\text{N/mm}^2$

$$M_{t,\ max} = 0.15 \times 800 \times 175^2 \times 40$$
$$= 147\,\text{kNm} > 136.2\,\text{kNm} \quad \text{(member 9, joint 2)}$$

The column slab connection at the edge can transfer the applied moment and no further redistribution is necessary. It is conservatively assumed in this analysis that the depth of the slab at the column is 200 mm, ignoring the drop. The moment $M_{t,\ max}$ is greater than the design moment obtained from an equivalent frame analysis.

Step 3 Find bending moments at $h_c/2$ and at edge of drop



SK 9/33 Bending moments at critical points – combination C_1 (20% redistribution).

Member 9 – combination C_I (20% redistribution)

Joint 5:
Bending moment = 172 kNm
Shear = 246 kN
Dead load: 1.4 × 40 = 56 kN/m near support
Live load: 1.6 × 30 = 48 kN/m near support
$h_c/2 = 0.225$ m

$$\text{Bending moment at } h_c/2 = 172 - 246 \times 0.225 + \frac{(56 + 48) \times 0.225^2}{2}$$

$$= 119.3\,\text{kNm} \quad \text{(top tension)}$$

Edge of drop = 1000 mm from centreline of column

$$\text{Bending moment at edge of drop} = 172 - 246 \times 1 + \frac{(56 + 48) \times 1^2}{2}$$

$$= -22\,\text{kNm} \quad \text{(bottom tension)}$$

Joint 2, similarly:

$$\text{Bending moment at } h_c/2 = 136.2 - 232 \times 0.225 + \frac{(56 + 48) \times 0.225^2}{2}$$

$$= 86.6\,\text{kNm} \quad \text{(top tension)}$$

Bending moment at edge of drop = −43.8 kNm (bottom tension)

Member 10 – combination C_I (20% redistribution)

Joint 5:
Bending moment at $h_c/2$ = 121.4 kNm (top tension)
Bending moment at edge of drop = −14.5 kNm (bottom tension)

Member 12 – combination C_1 (20% redistribution)

Joint 3:
Bending moment at $h_c/2$

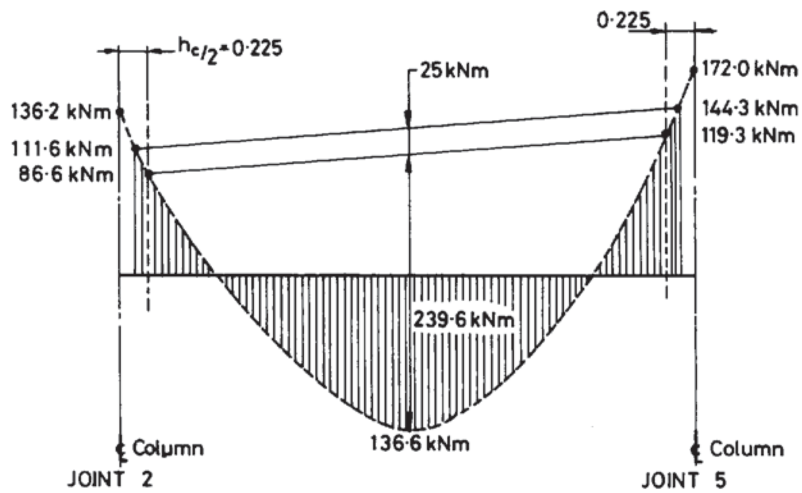
$$= 72.9 - 134.9 \times 0.225 + \frac{(1.4 \times 30 + 1.6 \times 9) \times 0.225^2}{2} = 44.0\,\text{kNm}$$

Joint 6:
Bending moment at $h_c/2$ = 71.3 kNm (top tension)

Member 13 – combination C_1 (20% redistribution)

Joint 6:
Bending moment at $h_c/2$ = 73.3 kNm (top tension)

Step 4 Check limitation of negative design moment



SK 9/34 Member 9 – combination C_1 (20% redistribution): limitation of negative design moment.

$$M' = \left(\frac{nl_2}{8}\right)\left(l_1 - \frac{2h_c}{3}\right)^2$$

where n = loading per unit area on slab.

Average n on span on first floor = 1.4 × 5 + 1.6 × 5 = 15 kN/m²

$l_1 = 5.0\,\text{m}$ $\quad l_2 = 6.0\,\text{m}$ $\quad h_c = 0.225\,\text{m}$

$$M' = \left(\frac{15 \times 6.0}{8}\right)\left(5.0 - \frac{2 \times 0.225}{3}\right)^2$$

$$= 264.6\,\text{kNm} \quad \text{(floor slab)}$$

Average n on span on roof = 1.4 × 5 + 1.6 × 1.5 = 9.4 kN/m^2

$$M' = \left(\frac{9.4 \times 6.0}{8}\right)\left(5.0 - \frac{2 \times 0.225}{3}\right)^2$$

= 165.8 kNm (roof slab)

Check negative moment limitation

Member 9
Joint 2 at $h_c/2 = M_2$ = 86.6 kNm (see Step 3)
Joint 5 at $h_c/2 = M_5$ = 119.3 kNm
Midspan moment = 136.6 kNm (see Step 1)

Average of M_2 and M_5 plus midspan moment
= 0.5 (86.6 + 119.3) + 136.6
= 239.6 kNm < M' = 264.6 kNm

The negative moments will have to be increased by (264.6 − 239.6) = 25.0 kNm

Revised negative moments:
Joint 2: 86.6 + 25.0 = 111.6 kNm
Joint 5: 119.3 + 25.0 = 144.3 kNm

Member 10
Joint 5 at $h_c/2 = M_5$ = 121.4 kNm
Midspan moment = 115.7 kNm
Average of negative and positive = 121.4 + 115.7 = 237.1 kNm < M' = 264.6 kNm

The negative moments will have to be increased by (264.6 − 237.1) = 27.5 kNm

Revised negative moments:
Joint 5: 121.4 + 27.5 = 148.9 kNm

Member 12
Joint 3 at $h_c/2 = M_3$ = 44.0 kNm
Joint 6 at $h_c/2 = M_6$ = 71.3 kNm
Midspan moment = 90.9 kNm

Average of negative and positive = 0.5 (44.0 + 71.3) + 90.9 = 148.55 kNm < M' = 165.8 kNm

The negative moments will have to be increased by (165.8 − 148.6) = 17.2 kNm

Revised negative moments:
Joint 3: 44.0 + 17.2 = 61.2 kNm
Joint 6: 71.3 + 17.2 = 88.5 kNm

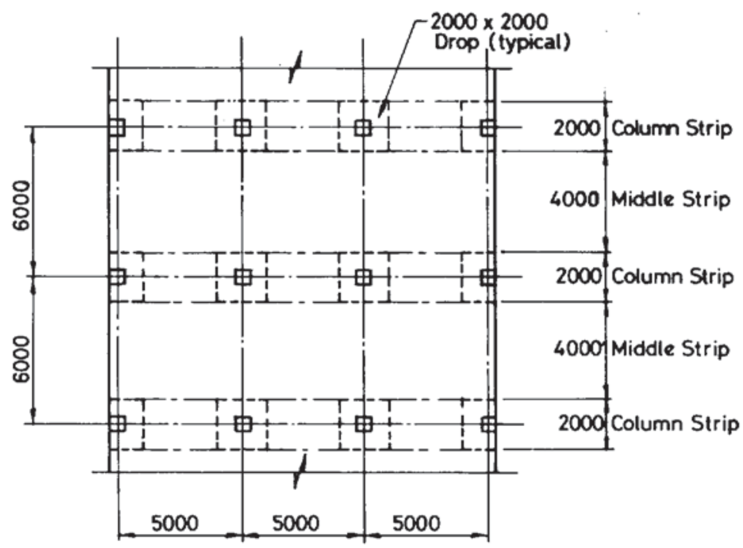
Member 13
Joint 6 at $h_c/2 = M_6$ = 73.3 kNm
Midspan moment = 72.7 kNm

Average of negative and positive = 73.3 + 72.7 = 146 kNm < M' = 165.8 kNm

The negative moments will have to be increased by (165.8 − 146) = 19.8 kNm

Revised negative moments:
Joint 6: 73.3 + 19.8 = 93.1 kNm

Step 5 Carry out division of panels



SK 9/35 Plan of floor slab – division of strips.

First floor slab

Size of drop = 2000 mm

$$\frac{l_x}{3} = \frac{5000}{3} = 1667\,\text{mm} < 2000\,\text{mm}$$

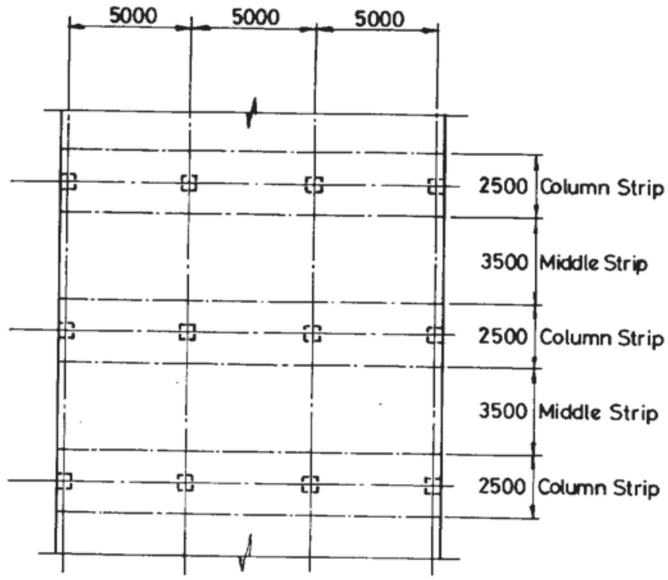
Column strip = 2000 mm

With drop middle strip = 5000 − 2000 = 3000 mm (y–y)
and = 6000 − 2000 = 4000 mm (x–x)

Middle strip in a slab without drop = 6000 − $l_x/2$ = 6000 − 2500 = 3500 mm

$$\text{Proportion of middle strip with drop and without drop} = \frac{4000}{3500} = 1.14$$

Roof slab



SK 9/36 Plan of roof slab – division of strips.

$$\text{Column strip} = \frac{l_x}{2} = \frac{5000}{2} = 2500\,\text{mm}$$

$$\text{Middle strip} = 6000 - 2500 = 3500\,\text{mm} \quad (x\text{–}x)$$
$$= 5000 - 2500 = 2500\,\text{mm} \quad (y\text{–}y)$$

Step 6 ***Divide moments between column strip and middle strip***

For slabs without drops,

Negative moments – 75% column strip
25% middle strip
Positive moments – 55% column strip
45% middle strip

Floor slab: design moments

Member 9: negative moments

Joint 2: 111.6 kNm (see Step 4)
Joint 5: 144.3 kNm (see Step 4)

Middle strip moments

Joint 2: 111.6 × 0.25 × 1.14 (see Step 4) = 31.8 kNm (top tension)

Edge of drop = 43.8 × 0.25 × 1.14 = 12.5 kNm (top tension)

Joint 5: 144.3 × 0.25 × 1.14 = 41.1 kNm (top tension)

Edge of drop = 22.0 × 0.25 × 1.14 = 6.3 kNm (top tension)

Column strip moments

Joint 2: 111.6 − 31.8 = 79.8 kNm (top tension)
Edge of drop: 43.8 − 12.5 = 31.3 kNm (top tension)
Joint 5: 144.3 − 41.1 = 103.2 kNm (top tension)
Edge of drop: 22 − 6.3 = 15.7 kNm (top tension)

Member 9: positive moments

Design midspan moment = 136.6 kNm (bottom tension)

Middle strip moments

Midspan: 136.6 × 0.45 × 1.14 = 70.1 kNm (bottom tension)

Column strip moments

Midspan: 136.6 − 70.1 = 66.5 kNm (bottom tension)
Member 10: negative moments
Joint 5: 148.9 kNm (see Step 4)

Middle strip moments

Joint 5: 148.9 × 0.25 × 1.14 = 42.4 kNm (top tension)
Edge of drop: 14.5 × 0.25 × 1.14 = 4.1 kNm (top tension)

Column strip moments

Joint 5: 148.9 − 42.4 = 106.3 kNm (top tension)
Edge of drop: 14.5 − 4.1 = 10.4 kNm (top tension)

Member 10: positive moments

Design midspan moment = 115.7 kNm (bottom tension)

Middle strip moments

Midspan: 115.7 × 0.45 × 1.14 = 59.4 kNm (bottom tension)

Column strip moments

Midspan: 115.7 − 59.4 = 56.3 kNm (bottom tension)

Note: Similarly calculate moments in column strips and middle strips in roof slab.

Step 7 ***Determine cover to reinforcement***
See Step 3 of Section 3.3.

Step 8 ***Design for flexure***
See Step 4 of Section 3.3.
The increased slab thickness at drops may be considered for the determination of reinforcement provided all reinforcement is properly anchored. Check reinforcement also at edge of drop.

Note: In this example the reinforcement is found for the flat slab spanning in the short direction only. Exactly the same method of analysis and design should be used to find the reinforcement in the long direction.

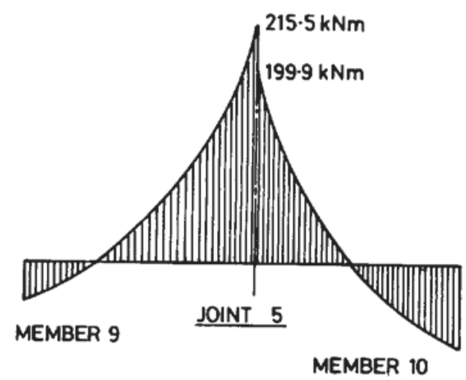
Step 9 ***Detailing of reinforcement***

Two-thirds of the negative support reinforcement in the column strip should be placed in half the width of the column strip centred over the column.

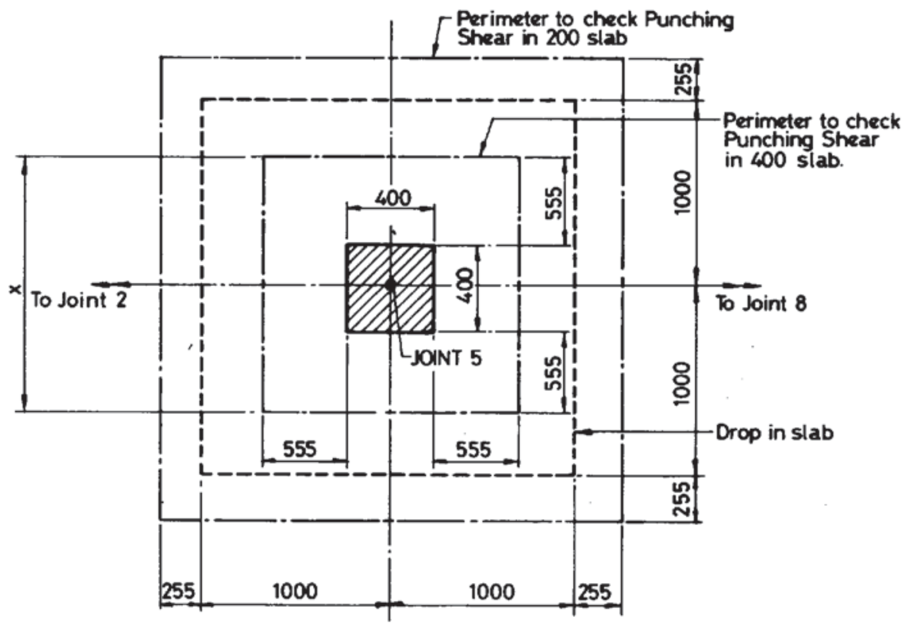
Step 10 ***Calculate punching shear and shear stress***

Punching shear at floor slab

Check joint 5.



SK 9/37 Moment transfer to column for punching shear calculation.



SK 9/38 Punching shear perimeters – plan of floor slab.

Use results of elastic analysis of frame before redistribution.

Maximum column moment at
joint 5 = 215.5 − 199.9 = 15.6 kNm
$M_t = 15.6$ kNm

A 30% reduction is allowed if frame analysis is carried out.

$\therefore \quad M_t = 0.7 \times 15.6 = 10.9$ kNm

$V_t = 258.2 + 239.0 = 497.2$ kN

Punching shear perimeter at 1.5d from face of column.

$d = 400 - 30 = 370$ mm
$1.5d = 1.5 \times 370 = 555$ mm
$x = 400 + 2 \times 555 = 1510$ mm $= 1.510$ m

$$V_{eff} = V_t\left(1 + \frac{1.5M_t}{V_t x}\right) = 497.2\left(1 + \frac{1.5 \times 10.9}{497.2 \times 1.510}\right)$$
$$= 508\text{ kN}$$

Maximum shear stress at column perimeter ($U_o = 4 \times 400$)

$$= \frac{V_{eff}}{U_o d} = \frac{508 \times 10^3}{4 \times 400 \times 370} = 0.86\text{ N/m}^2 < 5\text{ N/m}^2 \quad \text{OK}$$

Shear stress, $v = \dfrac{V_{eff}}{Ud}$

$U = 4 \times 1510 = 6040$

$\therefore \quad v = \dfrac{508 \times 10^3}{6040 \times 370} = 0.23\text{ N/mm}^2 < v_c$ for minimum percentage of reinforcement

No shear reinforcement is required in slab with drop.
For slab outside drop consider that loaded area is perimeter of drop.

$V_{eff} = 508$ kN − (load on the area of drop)
$= 508 - 4 \times 22$
$= 420$ kN

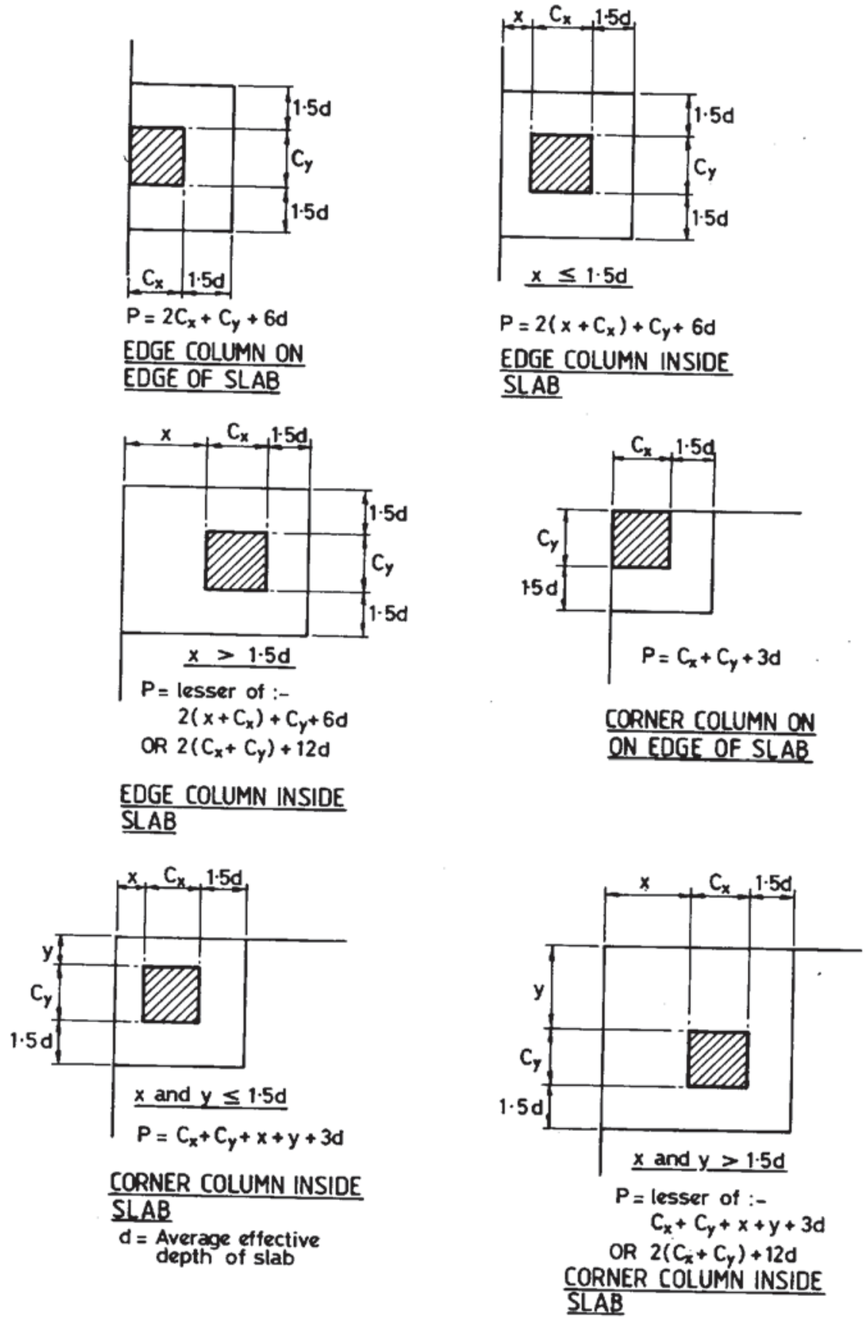
Perimeter of slab at 1.5d ($d = 170$ mm)
$= 4 \times (2000 + 3 \times 170) = 10040$ mm

Shear stress, $v = \dfrac{420 \times 10^3}{10040 \times 170}$

$= 0.25\text{ N/mm}^2 < v_c$ for minimum percentage of reinforcement

No shear reinforcement is required at internal columns of floor slab.
Similarly check for an external column and a corner column.

Rules for calculation of perimeters of external and corner columns



SK 9/39 Punching shear perimeters for flat slab.

The illustrations show the different column configurations with respect to a free edge and the corresponding perimeters for the calculation of punching shear stresses.

When the column face is more than $1.5d$ away from a free edge of slab, then there are two alternative perimeters possible as illustrated. Take the least of these two alternatives for the calculation of punching shear stress.

Punching shear at roof slab
Check joint 3 – edge column.

$V_t = 130\,\text{kN}$

The frame action considered is in the $x-x$ direction as explained in Section 9.3.5.

$V_{eff} = 1.25V_t = 1.25 \times 130 = 162.5\,\text{kN}$

$d = 170\,\text{mm}$ assumed
$1.5d = 1.5 \times 170 = 255\,\text{mm}$

Shear stress at column perimeter ($U_o = 3 \times 400 = 1200$)

$$= \frac{V_{eff}}{U_o d} = \frac{162.5 \times 10^3}{1200 \times 170} = 0.8\,\text{N/mm}^2 < 5\,\text{N/mm}^2 \quad \text{OK}$$

$$\text{Shear stress at } 1.5d = v = \frac{V_{eff}}{Ud}$$

$$U = [2 \times (400 + 255)] + (400 + 510)$$
$$= 2220$$

$$\therefore \quad v = \frac{162.5 \times 10^3}{2220 \times 170}$$
$$= 0.43\,\text{N/mm}^2$$

Assume minimum percentage of tensile reinforcement in slab.

$v_c = 0.48\,\text{N/mm}^2$ for Grade 40 concrete and an effective depth of 170 mm.

No shear reinforcement is necessary.

Note: The punching shear check should also be carried out for the flat slab spanning in the long direction and the worst result should be used.

Step 11 Check span/effective depth ratio
Follow Step 11 of Section 3.3

Step 12 Curtailment of bars
Follow Step 12 of Section 3.3.

Step 13 Spacing of bars
Follow Step 13 of Section 3.3.

Step 14 Check early thermal cracking
Follow Step 14 of Section 3.3.

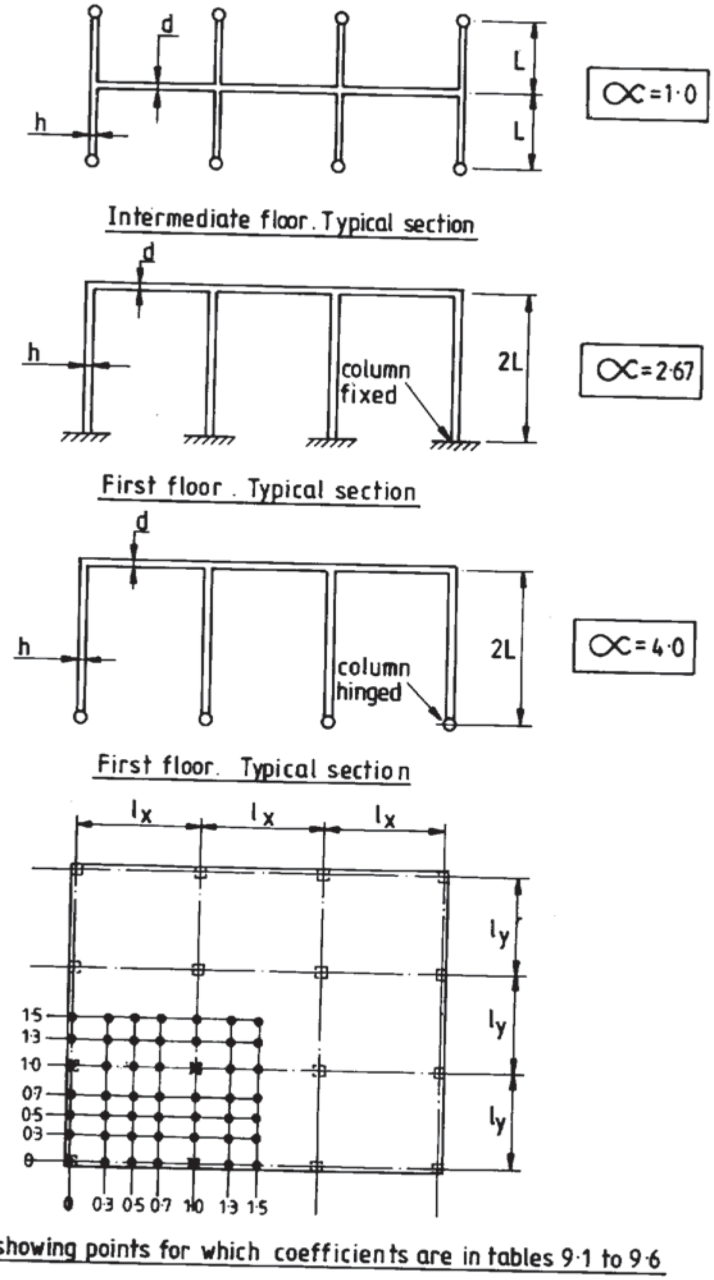
Step 15 Calculate minimum reinforcement
Follow Step 9 and Step 15 of Section 3.3.

Step 16 Calculate flexural crack width
Follow Step 16 of Section 3.3.

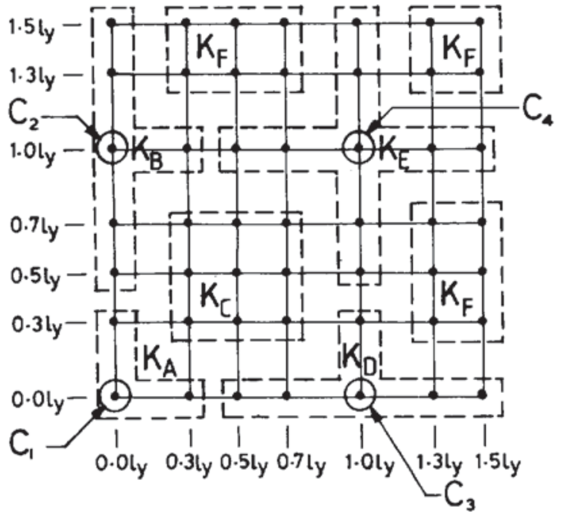
Step 17 Design of connections
Follow Chapter 11

9.6 TABLES AND GRAPHS FOR CHAPTER 9

How to use Tables 9.1 to 9.6



SK 9/40 Sketches to be used in conjunction with Tables 9.1–9.6.



SK 9/41 Zones of stiffness correction factors to be applied to points of interest.

Points to note:

(1) The flat slab system should have at least 3 spans in the l_x direction and 3 spans in the l_y direction.

(2) The coefficients are valid for equal spans in the l_x and l_y direction. They may be used up to a maximum variation of 20% in the span lengths.

(3) The tables can be used only for uniformly distributed loads with all spans loaded simultaneously with the maximum load.

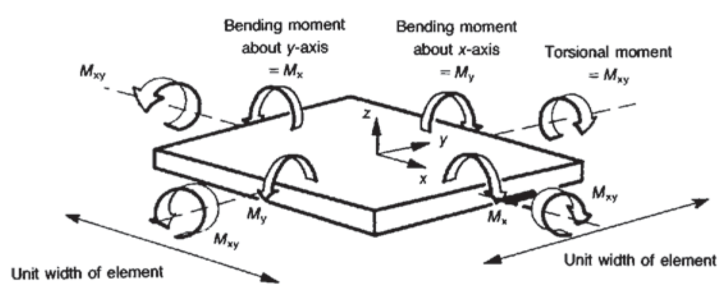
(4) To account for the possible increase in moment due to variation of live loads in different panels of the flat slab, no redistribution of moments should be carried out.

(5) For horizontal loading a separate frame analysis should be carried out and the appropriate moments will be combined with the vertical load moments. In general, flat slab construction should be fully braced and the horizontal load should be carried entirely by a shear-wall system.

(6) The coefficients are applicable to a corner panel, an edge panel with a free edge in the l_x direction, an edge panel with a free edge in the l_y direction and an internal panel.

(7) The moment triad (M_x, M_y and M_{xy}) obtained by this method of analysis should be combined using the Wood–Armer method as described in Section 1.12.

Sign convention for bending moments (+ ve in directions shown)



Note: A positive moment denotes hogging. This sign convention is opposite to the Wood–Armer convention. Reverse the signs of the moments before carrying out Wood–Armer combination as per Section 1.12.

Step-by-step analysis procedure

Step 1: Determine value of α (see SK 9/40 and find l_x/l_y).

Step 2: Assume d is the thickness of slab and h is the dimension of a side of a square column.

Step 3: From available L, as shown in SK 9/40, determine $S = \alpha d^3 L/h^4$.

Step 4: Select a point of interest from SK 9/40 or SK 9/41 where the moments have to be found. Corresponding to the zone of influence, find appropriate stiffness connection factor K depending on S from Graphs 9.1 to 9.18.

Step 5: Find moment coefficients from Tables 9.1 to 9.3 corresponding to l_x/l_y and the location of the point of interest.

Step 6: Find the ultimate uniformly distributed load on the flat slab = $n\,\text{kN/m}^2$.

Step 7: Find moment triads:

$$M_x = nC_xK_xl_y^2\,\text{kNm/m}$$
$$M_y = nC_yK_yl_y^2\,\text{kNm/m}$$
$$M_{xy} = nC_{xy}K_{xy}l_y^2\,\text{kNm/m}$$

Step 8: Carry out Wood–Armer combination.

Step 9: Find column reactions corresponding to l_x/l_y (see SK 9/41 for column locations).

Step 10: The moments obtained using these coefficients are in kNm/m. Find the effective width b_e as in Section 9.3.4. Multiply the moment obtained by analysis at edge and corner columns with the effective width b_e to find the slab-to-column connection moment. This transfer moment should be less than $M_{t,max}$ as defined in Section 9.3.4.

Reaction at column $C_1 = nC_1l_xl_y$
Reaction at column $C_2 = nC_2l_xl_y$, etc.

Note: If in the zone of K_A (as shown in SK 9/41) the point of interest is located, then to find M_x use stiffness correction factor K_{AX} corresponding to S and l_x/l_y as in Graph 19.1. Similarly if the point of interest lies in the zone of K_F then to find M_x use K_{FX} as in Graph 9.16.

The benefits of using these tables and graphs are that the analysis can be done very quickly and the necessity of carrying out the two analyses for the two orthogonal directions may be avoided. These tables can also be used for the analyses of raft foundation where the loading n may be assumed to be uniformly distributed over an inverted flat slab. The total loads from a structure will be assumed uniformly distributed at the underside of the raft.

Step 11: Calculate total column moments using Table 9.7 and divide the total moment between the columns at the junction depending on their relative stiffness. The stiffer the column, the more moment it will carry. The stiffness of a column may be calculated as I/l where I is the moment of inertia and l is the effective height.

Table 9.1 Bending moment coefficients for design of flat slabs ($l_x/l_y = 1.0$).

Location of point of interest	$0l_x$	$0.3l_x$	$0.5l_x$	$0.7l_x$	$1.0l_x$	$1.3l_x$	$1.5l_x$	Moment coefficient, C
$1.5l_y$	−0.011 84	−0.053 70	−0.056 97	−0.024 96	+0.026 33	−0.005 54	−0.022 30	C_x
	−0.049 87	−0.033 00	−0.027 94	−0.033 01	−0.043 58	−0.028 44	−0.022 30	C_y
	0.005 41	0.002 96	0.001 00	0.004 00	0.001 59	0.003 80	0.001 13	C_{xy}
$1.3l_y$	−0.010 48	−0.057 39	−0.062 49	−0.026 20	+0.046 90	−0.007 57	−0.028 44	C_x
	−0.027 76	−0.016 67	−0.013 22	−0.012 65	−0.019 54	−0.007 57	−0.005 54	C_y
	0.020 30	0.011 03	0.003 38	0.014 76	0.007 44	0.014 20	0.003 80	C_{xy}
$1.0l_y$	+0.054 82	−0.070 14	−0.075 60	−0.036 47	+0.156 56	−0.019 54	−0.043 58	C_x
	+0.115 71	+0.027 04	+0.015 21	−0.042 47	+0.156 56	+0.046 90	+0.026 33	C_y
	0.031 89	0.006 19	0.001 59	0.008 19	0.042 91	0.007 44	0.001 59	C_{xy}
$0.7l_y$	−0.010 49	−0.057 49	−0.063 57	−0.029 41	+0.042 47	−0.012 65	−0.033 01	C_x
	−0.041 32	−0.032 18	−0.029 93	−0.029 41	−0.036 47	−0.026 20	−0.024 96	C_y
	0.025 24	0.012 14	0.003 95	0.017 90	0.008 19	0.014 76	0.004 00	C_{xy}
$0.5l_y$	−0.001 57	−0.052 45	−0.057 18	−0.029 93	+0.015 21	−0.013 22	−0.027 94	C_x
	−0.076 03	−0.060 63	−0.057 18	−0.063 57	−0.075 60	−0.062 49	−0.056 97	C_y
	0.005 62	0.002 62	0.001 13	0.003 95	0.001 59	0.003 38	0.001 00	C_{xy}
$0.3l_y$	−0.010 59	−0.054 23	−0.060 63	−0.032 18	+0.027 04	−0.016 67	−0.033 00	C_x
	−0.065 15	−0.054 23	−0.052 45	−0.057 49	−0.070 14	−0.057 39	−0.053 70	C_y
	0.014 19	0.007 44	0.002 62	0.012 14	0.006 19	0.011 03	0.002 96	C_{xy}
$0l_y$	+0.063 70	−0.065 15	−0.076 03	−0.041 32	+0.115 71	−0.027 76	−0.049 87	C_x
	+0.063 70	−0.010 59	−0.011 57	−0.010 49	+0.054 82	−0.010 48	−0.011 84	C_y
	0.012 77	0.014 19	0.005 62	0.025 24	0.031 89	0.020 30	0.005 41	C_{xy}

$C_1 = 0.194\,355 \quad C_2 = 0.451\,53 \quad C_3 = 0.451\,53 \quad C_4 = 1.152\,63$

Table 9.2 Bending moment coefficients for design of flat slabs ($l_x/l_y = 1.2$).

Location of point of interest	$0l_x$	$0.3l_x$	$0.5l_x$	$0.7l_x$	$1.0l_x$	$1.3l_x$	$1.5l_x$	Moment coefficient, C
$1.5l_y$	−0.016 57	−0.079 80	−0.085 86	−0.034 73	+0.053 40	−0.008 98	−0.037 84	C_x
	−0.057 58	−0.035 57	−0.028 28	−0.032 95	−0.047 54	−0.025 34	−0.017 80	C_y
	0.006 08	0.003 47	0.001 08	0.004 74	0.002 36	0.004 66	0.001 23	C_{xy}
$1.3l_y$	−0.014 14	−0.083 48	−0.090 97	−0.037 64	−0.077 68	−0.012 56	−0.043 71	C_x
	−0.032 35	−0.020 82	−0.017 16	−0.013 32	−0.019 30	−0.005 93	−0.005 43	C_y
	0.022 11	0.012 11	0.003 37	0.016 24	0.010 89	0.016 04	0.003 83	C_{xy}
$1.0l_y$	+0.072 62	−0.095 12	−0.101 76	−0.050 03	+0.195 83	−0.026 21	−0.056 75	C_x
	+0.127 06	+0.015 42	+0.001 64	−0.034 08	+0.182 74	+0.039 07	+0.014 80	C_y
	0.042 29	0.005 73	0.001 58	0.007 76	0.050 87	0.007 07	0.001 45	C_{xy}
$0.7l_y$	−0.013 99	−0.082 30	−0.090 65	−0.040 80	+0.071 60	−0.017 33	−0.046 90	C_x
	−0.044 65	−0.035 75	−0.033 78	−0.030 32	−0.036 56	−0.025 69	−0.026 23	C_y
	0.028 31	0.013 60	0.004 21	0.020 85	0.012 13	0.017 47	0.004 32	C_{xy}
$0.5l_y$	−0.016 04	−0.076 64	−0.083 90	−0.039 94	+0.039 19	−0.015 88	−0.041 09	C_x
	−0.081 82	−0.061 17	−0.055 95	−0.058 09	−0.080 95	−0.060 46	−0.053 11	C_y
	0.007 10	0.003 16	0.001 48	0.005 02	0.002 36	0.004 31	0.001 15	C_{xy}
$0.3l_y$	−0.013 65	−0.077 96	−0.086 93	−0.042 49	+0.053 16	−0.020 17	−0.045 58	C_x
	−0.069 07	−0.054 32	−0.051 11	−0.057 30	−0.074 82	−0.055 76	−0.050 69	C_y
	0.014 57	0.009 63	0.002 99	0.014 17	0.009 35	0.013 30	0.003 31	C_{xy}
$0l_y$	+0.090 04	−0.089 34	−0.102 61	−0.054 44	+0.152 00	−0.033 80	−0.062 65	C_x
	+0.064 76	−0.011 34	−0.011 69	−0.011 06	+0.059 45	−0.010 81	−0.011 76	C_y
	0.018 71	0.019 02	0.006 41	0.030 97	0.036 57	0.025 40	0.006 56	C_{xy}

$C_1 = 0.195\,075$ $C_2 = 0.448\,11$ $C_3 = 0.453\,375$ $C_4 = 1.153\,44$

Table 9.3 Bending moment coefficients for design of flat slabs ($l_x/l_y = 1.4$).

Location of point of interest	$0l_x$	$0.3l_x$	$0.5l_x$	$0.7l_x$	$1.0l_x$	$1.3l_x$	$1.5l_x$	Moment coefficient, C
$1.5l_y$	−0.02136	−0.10961	−0.11941	−0.04705	+0.08741	−0.01469	−0.05713	C_x
	−0.06491	−0.03952	−0.03062	−0.03282	−0.05018	−0.02241	−0.01520	C_y
	0.00639	0.00369	0.00100	0.00498	0.00320	0.01037	0.00113	C_{xy}
$1.3l_y$	−0.01752	−0.11318	−0.12362	−0.05114	+0.11466	−0.01923	−0.06200	C_x
	−0.03625	−0.02623	−0.02280	−0.01445	−0.01742	−0.00486	−0.00687	C_y
	0.02248	0.01222	0.00296	0.01615	0.01479	0.01609	0.00335	C_{xy}
$1.0l_y$	+0.09384	−0.12354	−0.13173	−0.06459	+0.23834	−0.03332	−0.07210	C_x
	+0.13729	+0.00397	−0.01129	+0.02551	+0.20889	+0.03142	+0.00431	C_y
	0.05628	0.00527	0.00170	0.00717	0.05755	0.00649	0.00125	C_{xy}
$0.7l_y$	−0.01723	−0.11078	−0.12186	−0.05410	+0.10672	−0.02342	−0.06343	C_x
	−0.04721	−0.03997	−0.03854	−0.03142	−0.03513	−0.02532	−0.02823	C_y
	0.03003	0.01418	0.00411	0.02227	0.01662	0.01867	0.00422	C_{xy}
$0.5l_y$	−0.02056	−0.10476	−0.11551	−0.05103	+0.07003	−0.02048	−0.05780	C_x
	−0.08747	−0.06227	−0.05539	−0.06281	−0.08560	−0.05789	−0.04970	C_y
	0.00842	0.00355	0.00170	0.00575	0.00318	0.00483	0.00116	C_{xy}
$0.3l_y$	−0.01606	−0.10580	−0.11820	−0.05479	+0.08586	−0.02520	−0.06155	C_x
	−0.07295	−0.05462	−0.04976	−0.05667	−0.07905	−0.05361	−0.04740	C_y
	0.01377	0.01071	0.00382	0.01612	0.01305	0.01473	0.00343	C_{xy}
$0l_y$	+0.12167	−0.11753	−0.13360	−0.06868	+0.19234	−0.04028	−0.07779	C_x
	+0.06593	−0.01189	−0.01158	−0.01135	+0.06438	−0.01080	−0.01135	C_y
	0.03124	0.02358	0.00741	0.03596	0.04237	0.02947	0.00739	C_{xy}

$C_1 = 0.19683 \quad C_2 = 0.445545 \quad C_3 = 0.45477 \quad C_4 = 1.1529$

Table 9.4 Bending moment coefficients for design of flat slabs ($l_x/l_y = 1.6$).

Location of point of interest	$0l_x$	$0.3l_x$	$0.5l_x$	$0.7l_x$	$1.0l_x$	$1.3l_x$	$1.5l_x$	Moment coefficient, C
$1.5l_y$	−0.025 96	−0.142 72	−0.156 90	−0.061 72	+0.127 85	−0.022 44	−0.079 38	C_x
	−0.071 31	−0.044 61	−0.034 96	−0.032 74	−0.051 06	−0.019 84	−0.014 75	C_y
	0.006 38	0.003 67	0.000 83	0.004 84	0.004 11	0.004 78	0.000 91	C_{xy}
$1.3l_y$	−0.020 28	−0.146 12	−0.160 10	−0.066 52	+0.157 46	−0.027 46	−0.083 06	C_x
	−0.039 24	−0.032 65	−0.029 81	−0.016 01	−0.013 75	−0.004 39	−0.009 69	C_y
	0.021 63	0.011 67	0.002 36	0.014 96	0.019 12	0.014 89	0.002 61	C_{xy}
$1.0l_y$	+0.118 58	−0.155 31	−0.165 70	−0.080 20	+0.284 62	−0.041 16	−0.090 17	C_x
	+0.146 26	−0.007 27	−0.023 53	+0.017 04	+0.234 86	+0.024 16	−0.005 11	C_y
	0.074 90	0.004 93	0.002 11	0.006 49	0.063 14	0.005 78	0.001 02	C_{xy}
$0.7l_y$	−0.019 88	−0.142 73	−0.157 13	−0.069 14	+0.147 68	−0.030 91	−0.082 71	C_x
	−0.049 01	−0.044 71	−0.043 92	−0.032 74	−0.032 31	−0.025 16	−0.030 77	C_y
	0.030 44	0.013 97	0.003 73	0.022 43	0.021 62	0.018 71	0.003 86	C_{xy}
$0.5l_y$	−0.024 79	−0.136 55	−0.151 62	−0.064 94	+0.107 49	−0.026 97	−0.077 73	C_x
	−0.092 66	−0.063 88	−0.055 66	−0.061 85	−0.089 34	−0.055 11	−0.047 13	C_y
	0.009 54	0.003 83	0.001 82	0.006 02	0.004 08	0.005 04	0.001 10	C_{xy}
$0.3l_y$	−0.017 37	−0.137 51	−0.154 19	−0.069 02	+0.124 93	−0.031 82	−0.080 87	C_x
	−0.076 49	−0.055 20	−0.048 68	−0.055 76	−0.082 63	−0.051 25	−0.044 33	C_y
	0.011 89	0.012 51	0.002 67	0.017 59	0.017 27	0.015 59	0.003 46	C_{xy}
$0l_y$	+0.158 59	−0.149 62	−0.169 14	−0.084 15	+0.237 12	−0.047 57	−0.095 90	C_x
	+0.066 99	−0.012 31	−0.011 39	−0.011 45	+0.069 50	−0.010 61	−0.010 82	C_y
	0.051 31	0.028 10	0.008 30	0.040 46	0.048 91	0.032 67	0.007 99	C_{xy}

$C_1 = 0.199\,125$ $C_2 = 0.443\,385$ $C_3 = 0.455\,76$ $C_4 = 1.151\,73$

Table 9.5 Bending moment coefficients for design of flat slabs ($l_x/l_y = 1.8$).

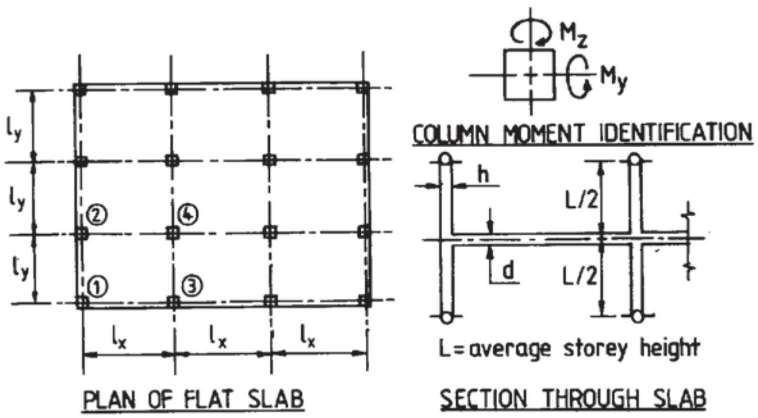
Location of point of interest	$0l_x$	$0.3l_x$	$0.5l_x$	$0.7l_x$	$1.0l_x$	$1.3l_x$	$1.5l_x$	Moment coefficient, C
$1.5l_y$	−0.030 10	−0.178 90	−0.197 96	−0.078 54	+0.174 36	−0.032 03	−0.104 20	C_x
	−0.076 52	−0.050 63	−0.041 05	−0.032 79	−0.050 01	−0.017 74	−0.016 33	C_y
	0.006 08	0.003 50	0.000 63	0.004 42	0.005 11	0.004 31	0.000 66	C_{xy}
$1.3l_y$	−0.022 10	−0.182 09	−0.200 23	−0.083 64	+0.205 82	−0.037 50	−0.106 77	C_x
	−0.041 20	−0.039 86	−0.037 92	−0.017 91	−0.008 29	−0.004 47	−0.013 68	C_y
	0.019 76	0.010 78	0.001 71	0.013 11	0.023 88	0.012 93	0.001 81	C_{xy}
$1.0l_y$	+0.146 87	−0.190 28	−0.203 70	−0.096 95	+0.335 05	−0.049 97	−0.111 26	C_x
	+0.154 02	−0.018 30	−0.035 10	+0.008 88	+0.260 64	+0.017 36	−0.013 59	C_y
	0.099 11	0.004 89	0.002 47	0.006 31	0.067 83	0.004 98	0.000 81	C_{xy}
$0.7l_y$	−0.021 83	−0.178 00	−0.196 42	−0.085 82	+0.194 37	−0.039 82	−0.104 88	C_x
	−0.050 07	−0.049 86	−0.049 70	−0.034 25	−0.028 18	−0.025 23	−0.033 68	C_y
	0.029 65	0.013 13	0.003 44	0.021 59	0.027 08	0.017 92	0.003 37	C_{xy}
$0.5l_y$	−0.028 40	−0.171 82	−0.191 96	−0.080 93	+0.151 36	−0.035 31	−0.100 74	C_x
	−0.097 18	−0.065 94	−0.056 75	−0.060 72	−0.092 04	−0.052 32	−0.045 52	C_y
	0.010 47	0.004 03	0.001 84	0.005 89	0.005 16	0.005 03	0.000 99	C_{xy}
$0.3l_y$	−0.017 16	−0.172 85	−0.194 64	−0.085 11	+0.170 18	−0.040 05	−0.103 47	C_x
	−0.079 44	−0.056 07	−0.048 04	−0.546 80	−0.085 44	−0.048 87	−0.041 77	C_y
	0.009 08	0.014 58	0.003 96	0.019 06	0.021 96	0.016 15	0.003 50	C_{xy}
$0l_y$	+0.200 72	−0.185 39	−0.209 19	−0.100 93	+0.286 59	−0.055 94	−0.117 31	C_x
	+0.067 85	−0.012 73	−0.011 21	−0.011 43	+0.074 73	−0.010 29	−0.010 27	C_y
	0.080 02	0.032 92	0.009 14	0.044 72	0.055 97	0.035 22	0.008 44	C_{xy}

$C_1 = 0.201\,78$ $C_2 = 0.441\,495$ $C_3 = 0.456\,39$ $C_4 = 1.150\,38$

Table 9.6 Bending moment coefficients for design of flat slabs ($l_x/l_y = 2.0$).

Location of point of interest	$0l_x$	$0.3l_x$	$0.5l_x$	$0.7l_x$	$1.0l_x$	$1.3l_x$	$1.5l_x$	Moment coefficient, C
$1.5l_y$	−0.033 52	−0.217 98	−0.242 38	−0.097 27	+0.226 71	−0.043 31	−0.131 47	C_x
	−0.080 45	−0.057 33	−0.048 55	−0.033 00	−0.047 00	−0.016 14	−0.019 60	C_y
	0.005 53	0.003 26	0.000 44	0.003 86	0.006 21	0.003 68	0.000 41	C_{xy}
$1.3l_y$	−0.022 66	−0.220 93	−0.243 90	−0.102 37	+0.259 60	−0.048 25	−0.133 12	C_x
	−0.042 10	−0.047 64	−0.046 82	−0.020 07	−0.001 05	−0.005 02	−0.018 61	C_y
	0.017 07	0.009 81	0.001 13	0.010 98	0.029 01	0.010 64	0.001 07	C_{xy}
$1.0l_y$	+0.178 69	−0.228 30	−0.245 72	−0.114 92	+0.389 91	−0.059 93	−0.135 51	C_x
	+0.160 68	−0.029 09	−0.046 07	+0.001 16	+0.286 34	−0.011 05	−0.021 32	C_y
	0.129 76	0.005 37	0.002 75	0.006 33	0.071 81	0.004 36	0.001 02	C_{xy}
$0.7l_y$	−0.021 87	−0.216 36	−0.239 68	−0.104 05	+0.246 72	−0.050 17	−0.130 05	C_x
	−0.050 39	−0.055 33	−0.055 73	−0.035 91	−0.022 78	−0.025 54	−0.036 88	C_y
	0.027 77	0.011 79	0.003 31	0.020 00	0.032 92	0.016 56	0.002 83	C_{xy}
$0.5l_y$	−0.031 05	−0.210 43	−0.236 36	−0.098 83	+0.201 47	−0.045 40	−0.126 77	C_x
	−0.100 89	−0.068 37	−0.058 57	−0.059 54	−0.093 63	−0.049 68	−0.044 82	C_y
	0.011 21	0.004 18	0.001 78	0.005 88	0.006 35	0.004 88	0.000 90	C_{xy}
$0.3l_y$	−0.015 05	−0.211 63	−0.239 36	−0.102 99	+0.221 44	−0.049 92	−0.129 32	C_x
	−0.081 66	−0.057 19	−0.047 90	−0.053 50	−0.087 41	−0.046 57	−0.039 84	C_y
	0.005 49	0.017 03	0.004 46	0.020 71	0.027 05	0.016 63	0.003 61	C_{xy}
$0l_y$	+0.247 89	−0.224 65	−0.253 66	−0.119 07	+0.340 90	−0.065 57	−0.142 15	C_x
	+0.068 45	−0.013 16	−0.011 09	−0.011 31	+0.080 06	−0.009 91	−0.009 79	C_y
	0.118 41	0.038 47	0.010 02	0.049 00	0.063 41	0.037 30	0.008 82	C_{xy}

$C_1 = 0.204\,615 \quad C_2 = 0.439\,74 \quad C_3 = 0.456\,66 \quad C_4 = 1.149\,03$



SK 9/42 Column number identification.

Table 9.7 Bending moment coefficient for design of columns in flat slab construction.

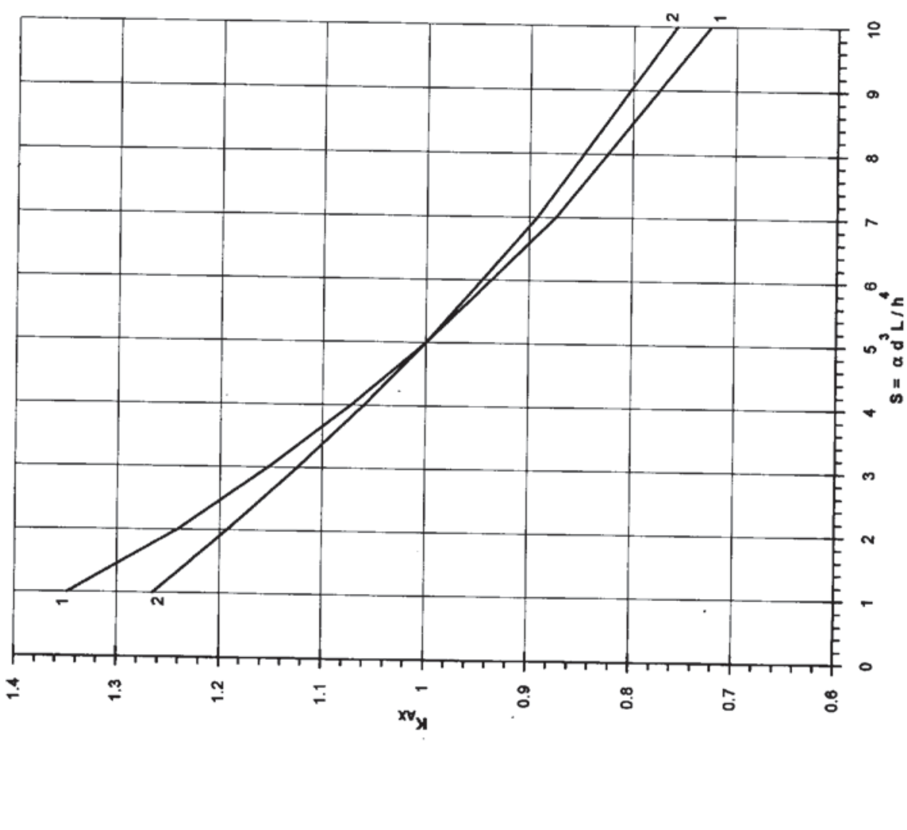
l_x/l_y Column no.	1	1.2	1.4	1.6	1.8	2	Moment coefficients
1	0.011 75	0.013 45	0.015 26	0.017 17	0.019 16	0.021 23	C_{cy}
1	−0.011 75	−0.017 92	−0.025 94	−0.036 03	−0.048 36	−0.063 07	C_{cz}
2	−0.002 44	−0.002 33	−0.002 24	−0.002 18	−0.002 16	−0.002 18	C_{cy}
2	−0.022 80	−0.032 71	−0.045 25	−0.060 73	−0.079 39	−0.101 47	C_{cz}
3	0.022 80	0.027 65	0.033 01	−0.038 85	0.045 13	0.051 84	C_{cy}
3	0.002 44	0.004 41	0.007 06	0.010 40	0.014 45	0.019 19	C_{cz}
4	−0.003 76	−0.003 93	−0.004 18	−0.004 54	−0.005 06	−0.005 71	C_{cy}
4	0.003 76	0.006 59	0.010 28	0.014 84	0.020 31	0.026 67	C_{cz}

Column total moment $= C_c \times R \times l_y^3 \times \eta$

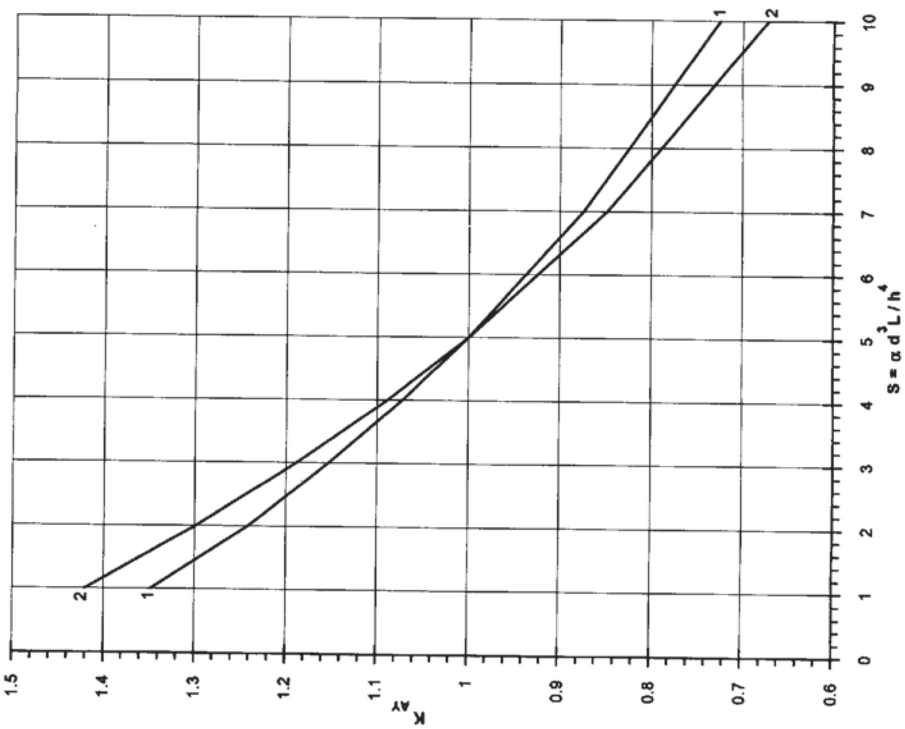
where η = load per unit area on slab.
See Graphs 9.19–9.26 for factors R_y and R_z.

Note: Divide total moment in column to top and bottom column in proportion to their stiffness.

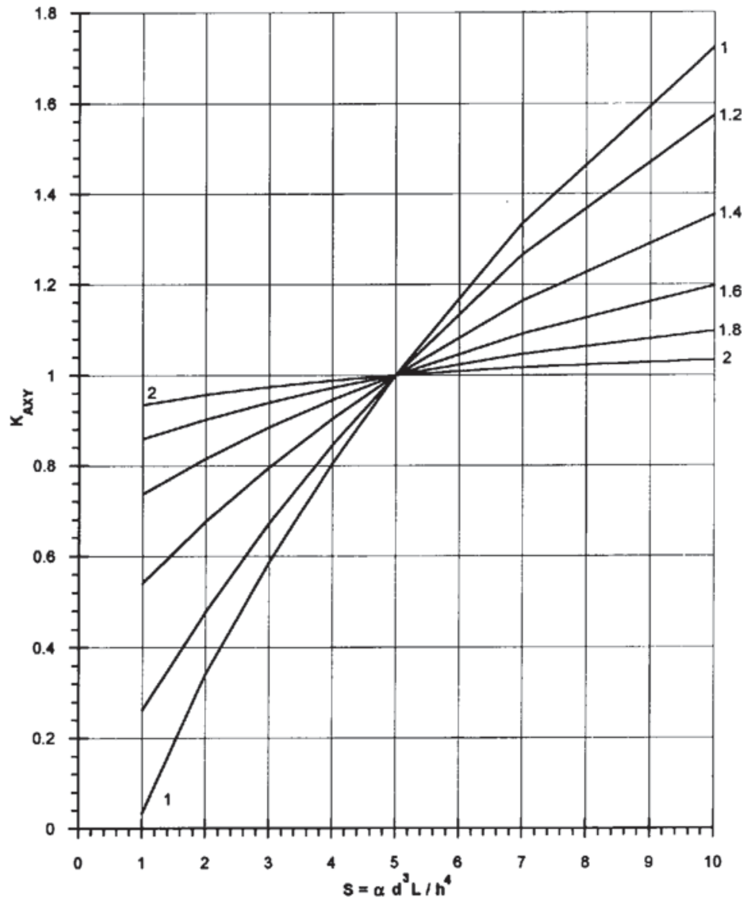
Graph 9.1 K_{Ax} for M_x. Stiffness correction coefficients for Zone A, curves for $l_x/l_y = 1$ and 2.



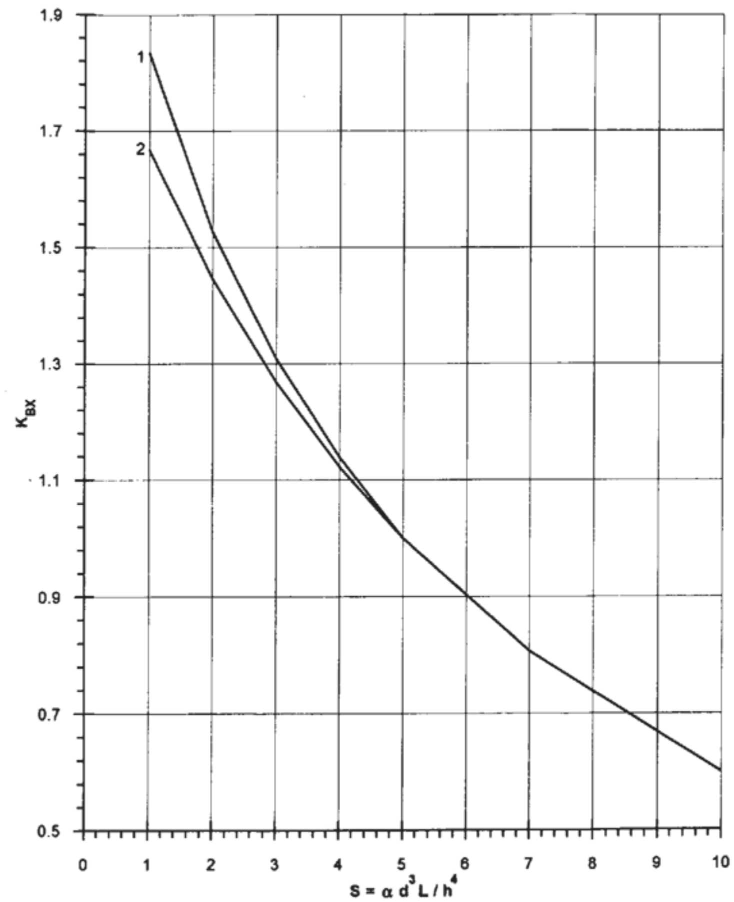
Graph 9.2 K_{Ay} for M_y. Stiffness correction coefficients for Zone A, curves for $l_x/l_y = 1$ and 2.



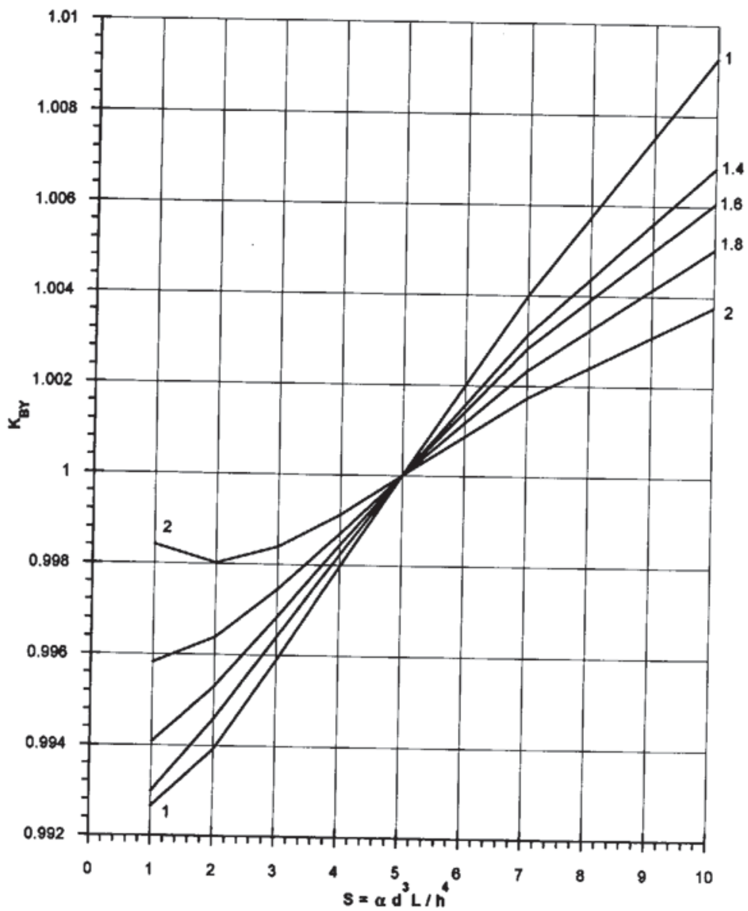
Graph 9.3 K_{Axy} for M_{xy}. Stiffness correction coefficients for Zone A, curves for $l_x/l_y = 1$, 1.2, 1.4, 1.6, 1.8 and 2.



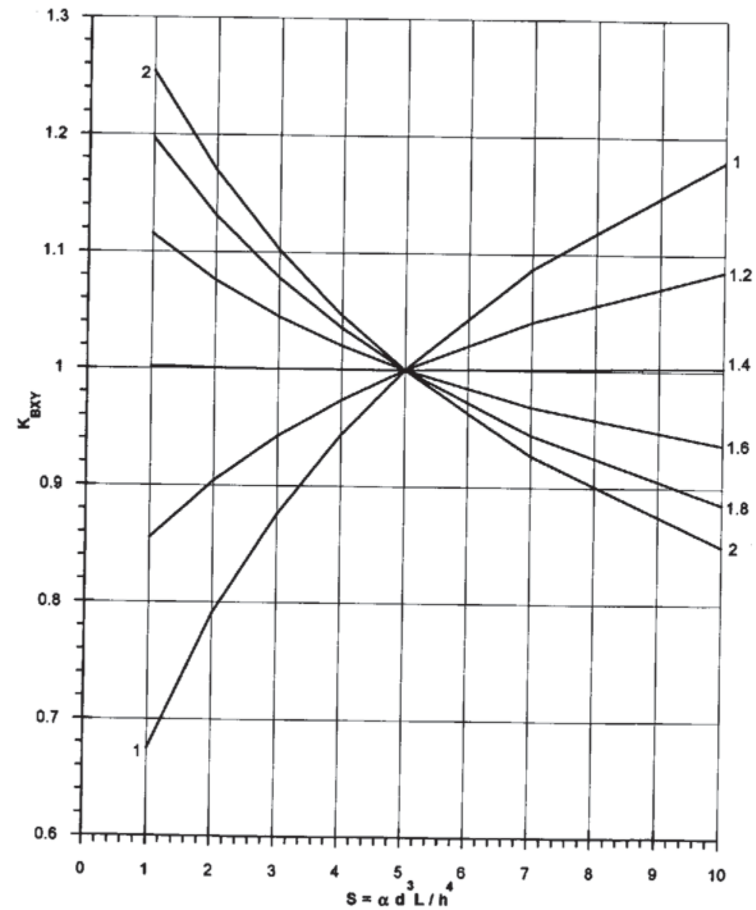
Graph 9.4 K_{Bx} for M_x. Stiffness correction coefficients for Zone B, curves for $l_x/l_y = 1$ and 2.

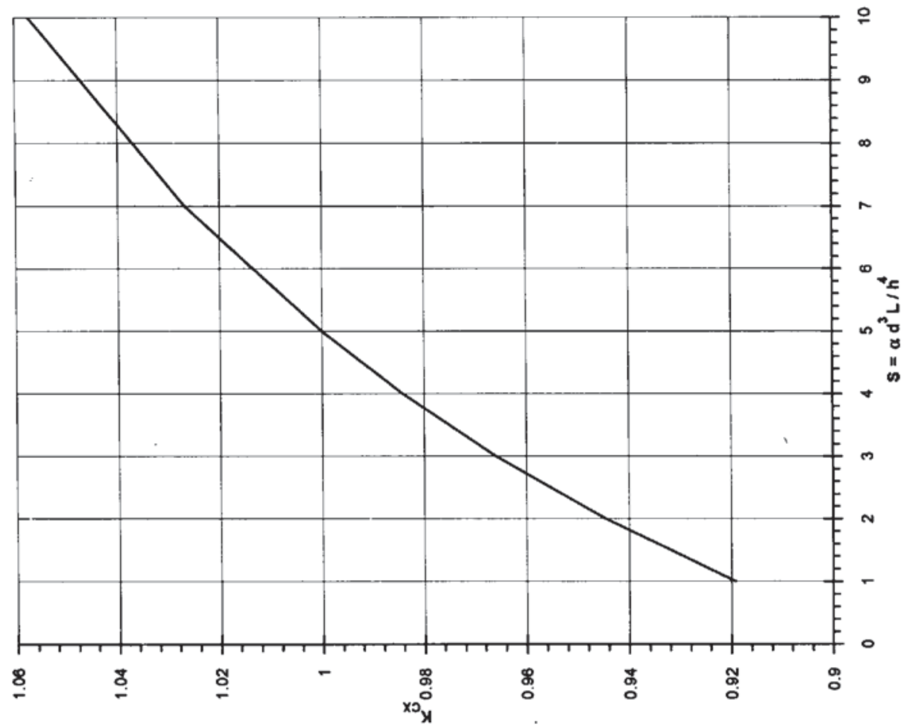


Graph 9.5 K_{By} for M_y. Stiffness correction coefficients for Zone B, curves for $l_x/l_y = 1$, 1.4, 1.6, 1.8 and 2.

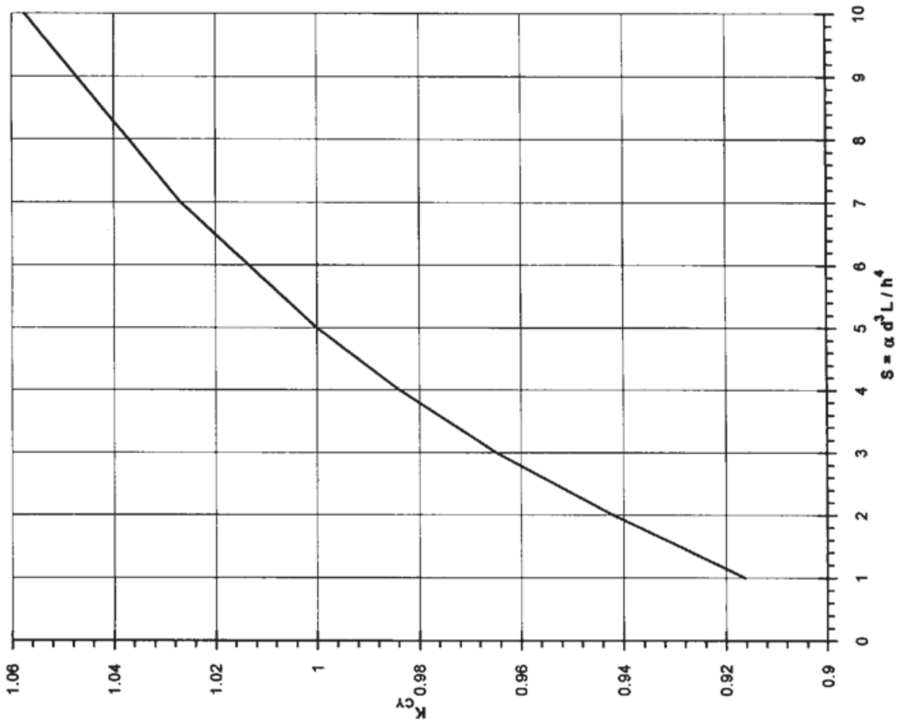


Graph 9.6 K_{Bxy} for M_{xy}. Stiffness correction coefficients for Zone B, curves for $l_x/l_y = 1$, 1.2, 1.4, 1.6, 1.8 and 2.



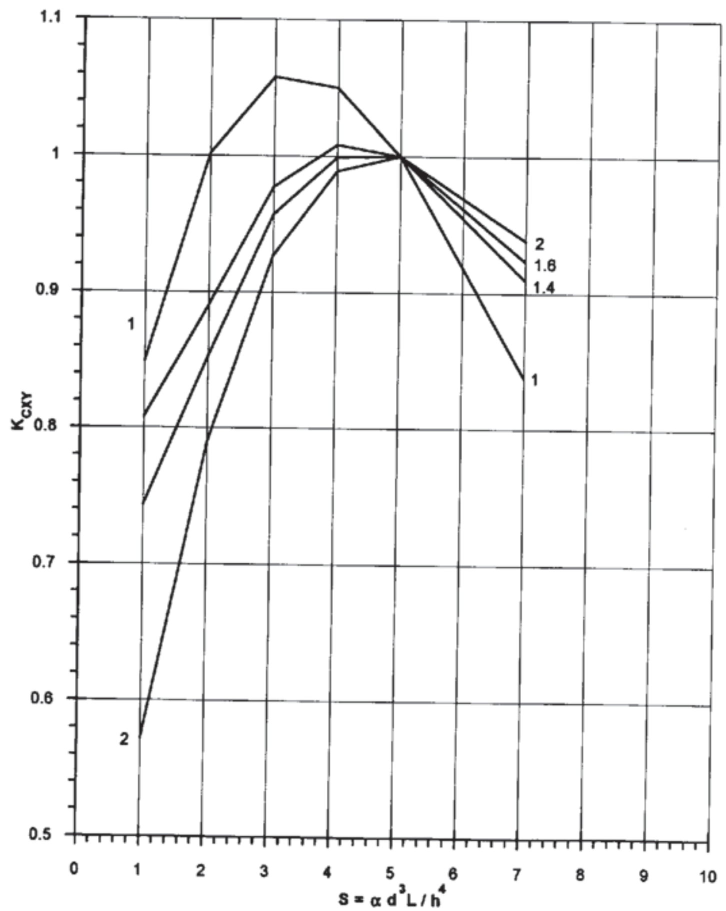


Graph 9.7 K_{Cx} for M_x. Stiffness correction coefficients for Zone C, curves for $l_x/l_y = 1$ and 2.

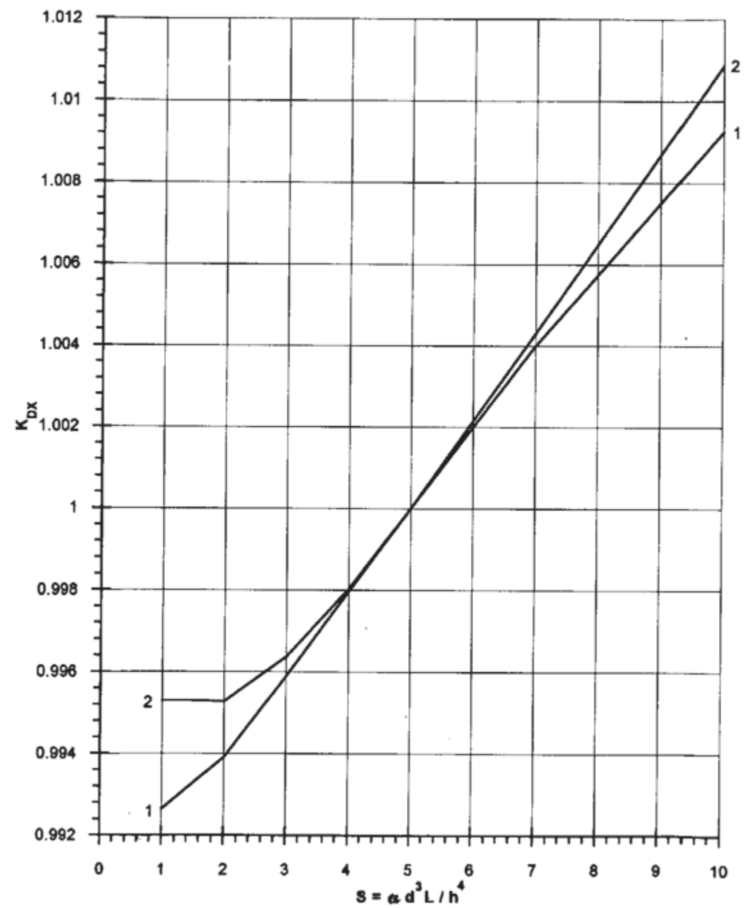


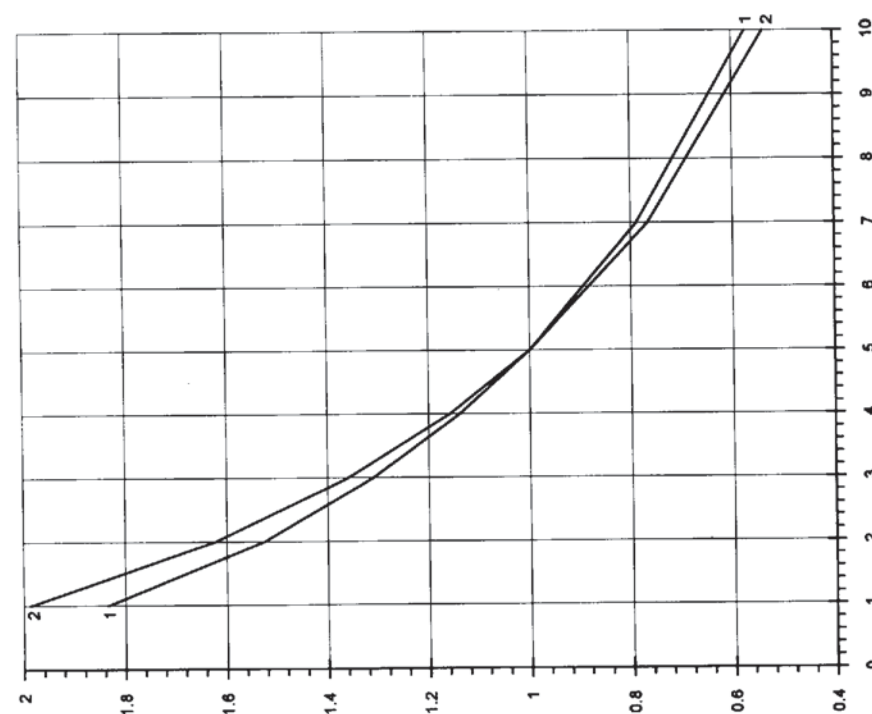
Graph 9.8 K_{Cy} for M_y. Stiffness correction coefficients for Zone C, curves for $l_x/l_y = 1$ and 2.

Graph 9.9 K_{Cxy} for M_{xy}. Stiffness correction coefficients for Zone C, curves for $l_x/l_y = 1$, 1.4, 1.6 and 2.

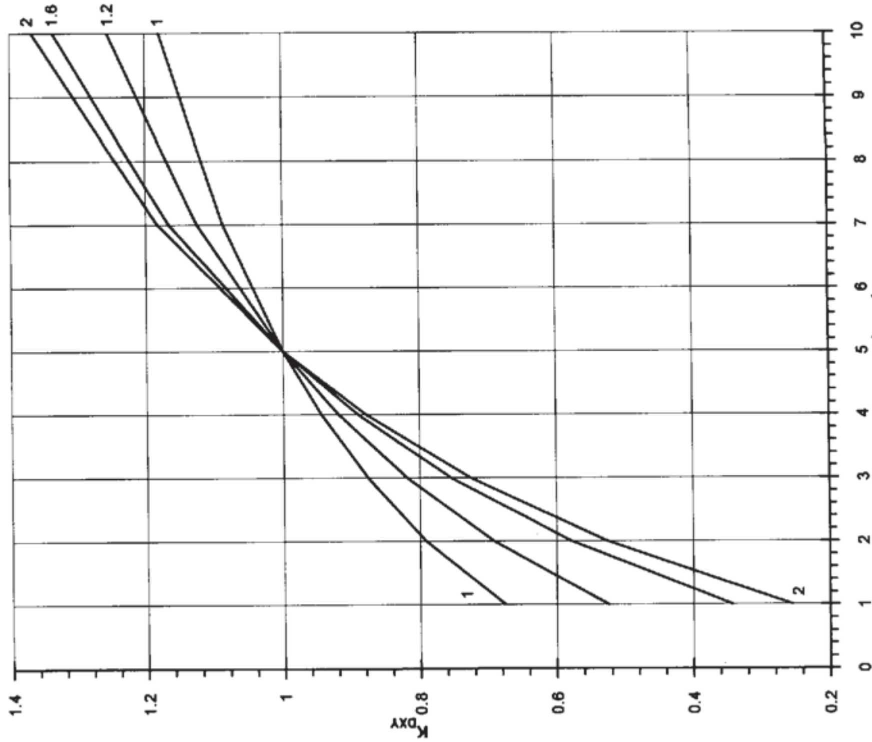


Graph 9.10 K_{Dx} for M_x. Stiffness correction coefficients for Zone D, curves for $l_x/l_y = 1$ and 2.



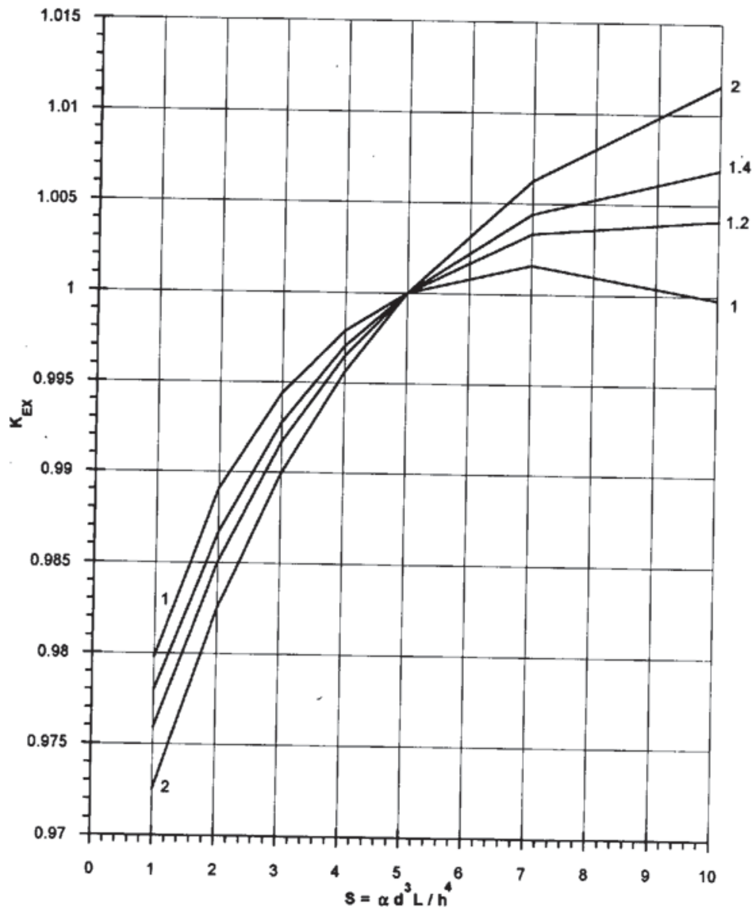


Graph 9.11 K_{Dy} for M_y. Stiffness correction coefficients for Zone D, curves for $l_x/l_y = 1$ and 2.

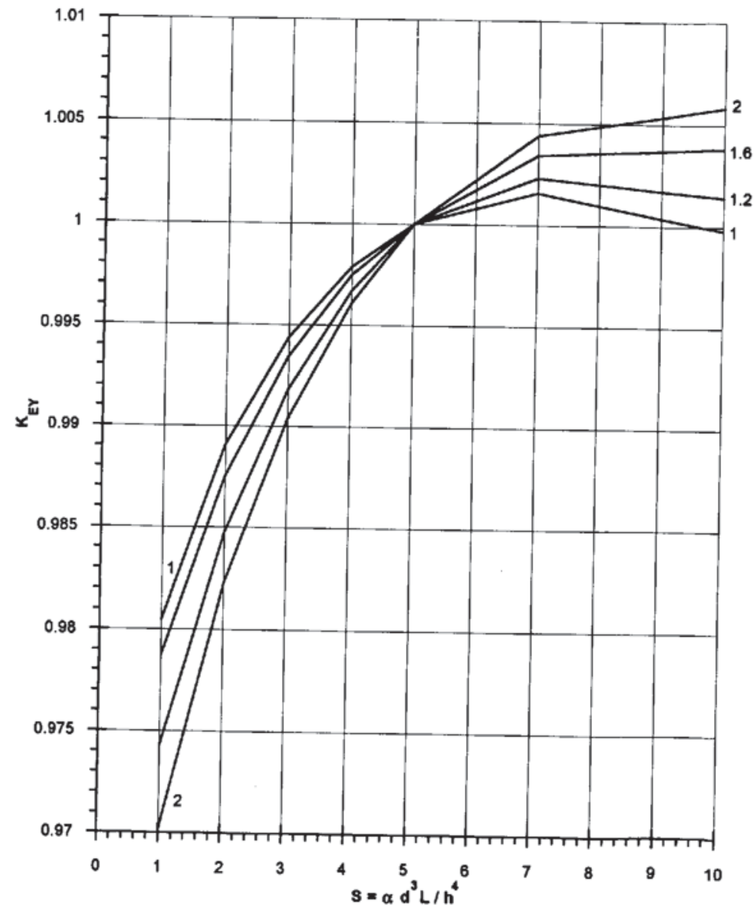


Graph 9.12 K_{Dxy} for M_{xy}. Stiffness correction coefficients for Zone D, curves for $l_x/l_y = 1, 1.2, 1.6$ and 2.

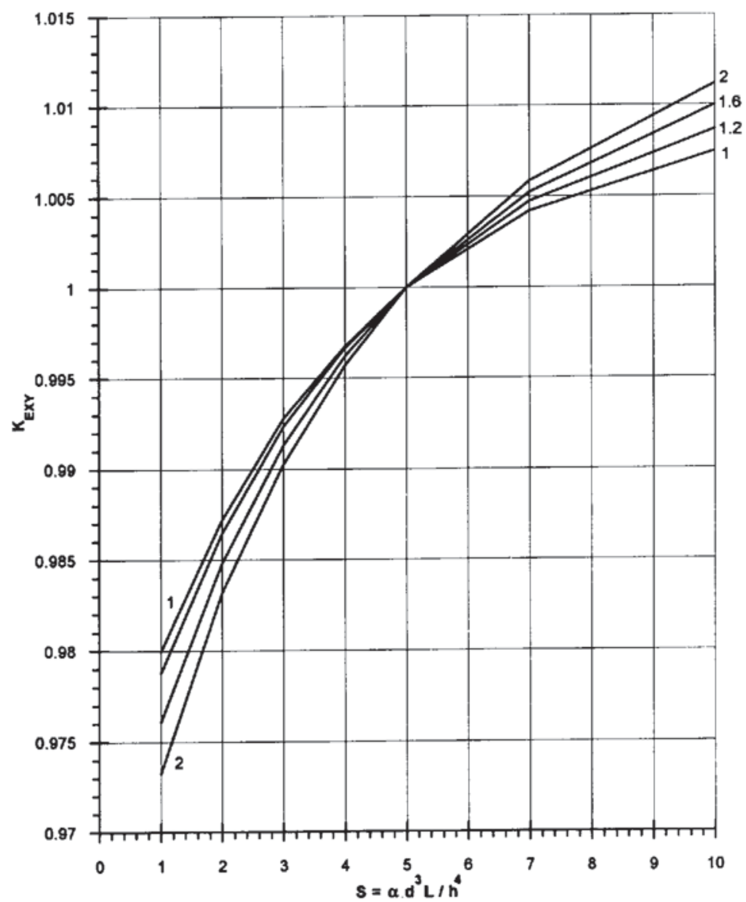
Graph 9.13 K_{Ex} for M_x. Stiffness correction coefficients for Zone E, curves for $l_x/l_y = 1$, 1.2, 1.4 and 2.



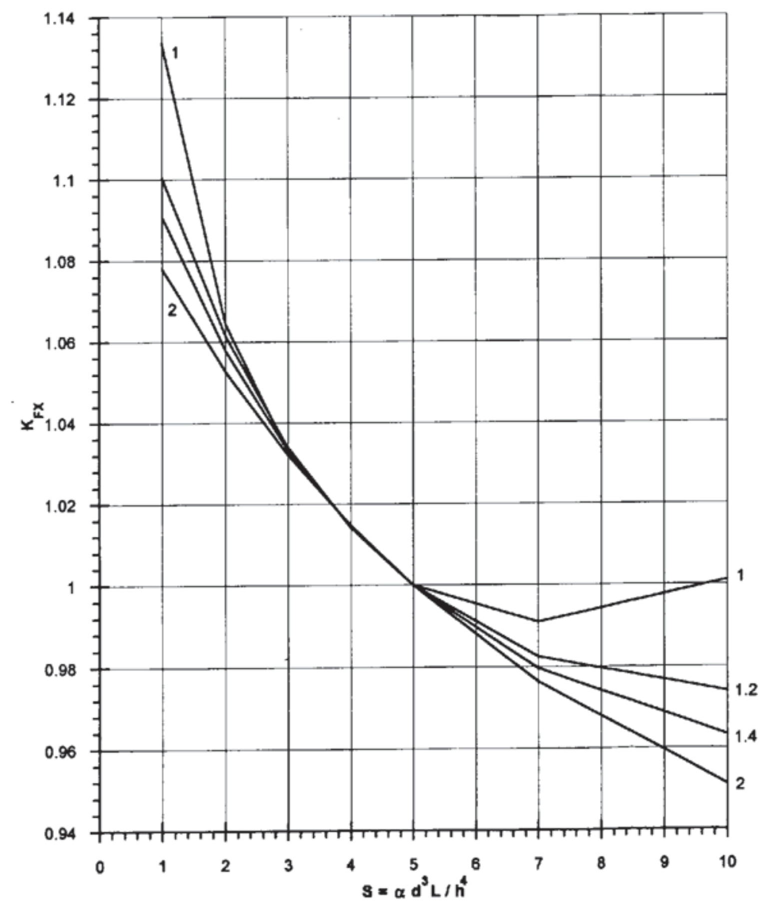
Graph 9.14 K_{Ey} for M_y. Stiffness correction coefficients for Zone E, curves for $l_x/l_y = 1$, 1.2, 1.6 and 2.



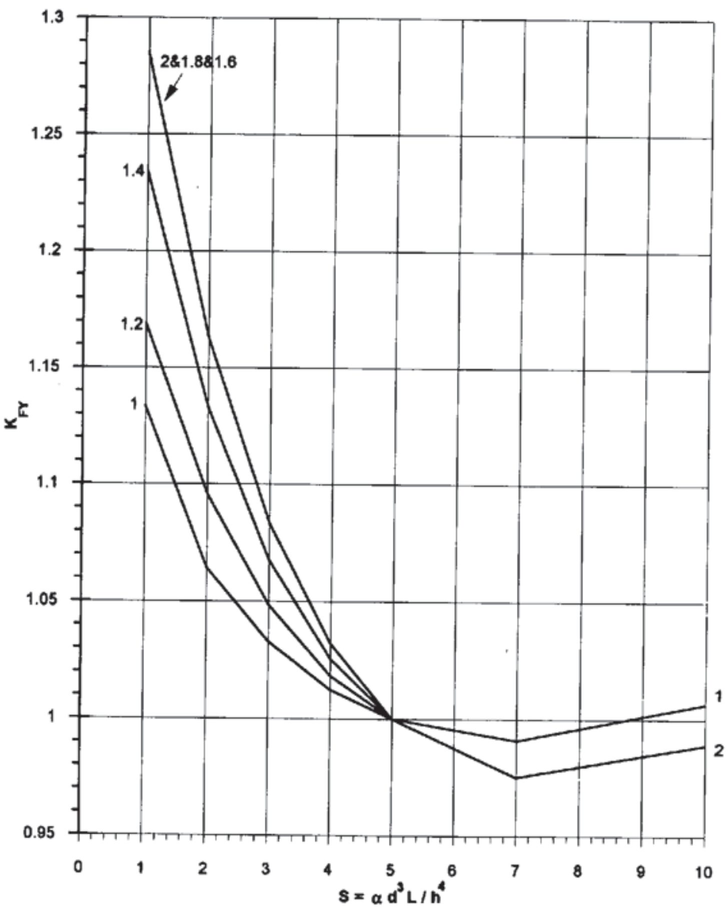
Graph 9.15 K_{Exy} for M_{xy}. Stiffness correction coefficients for Zone E, curves for $l_x/l_y = 1$, 1.2, 1.6 and 2.



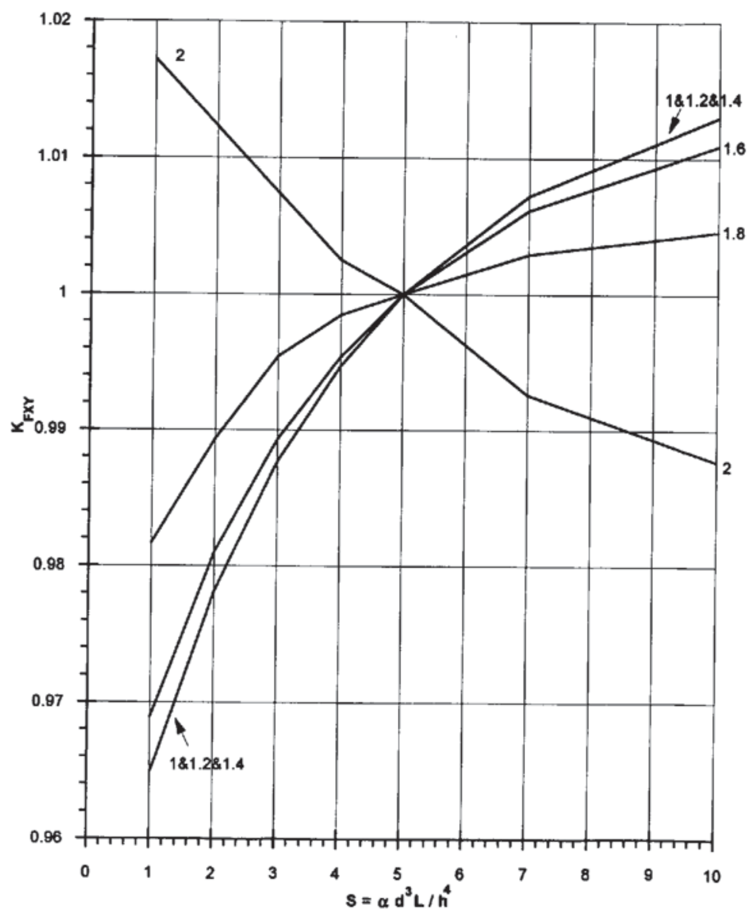
Graph 9.16 K_{Fx} for M_x. Stiffness correction coefficients for Zone F, curves for $l_x/l_y = 1$, 1.2, 1.4 and 2.



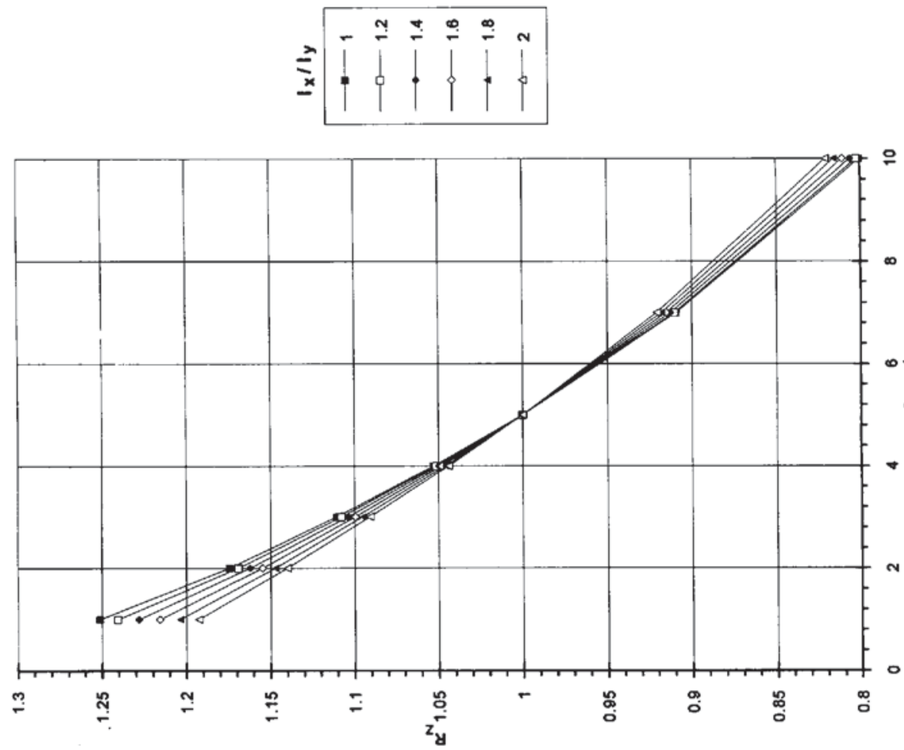
Graph 9.17 K_{Fy} for M_y. Stiffness correction coefficients for Zone F, curves for $l_x/l_y = 1$, 1.2, 1.4, 1.6, 1.8 and 2.



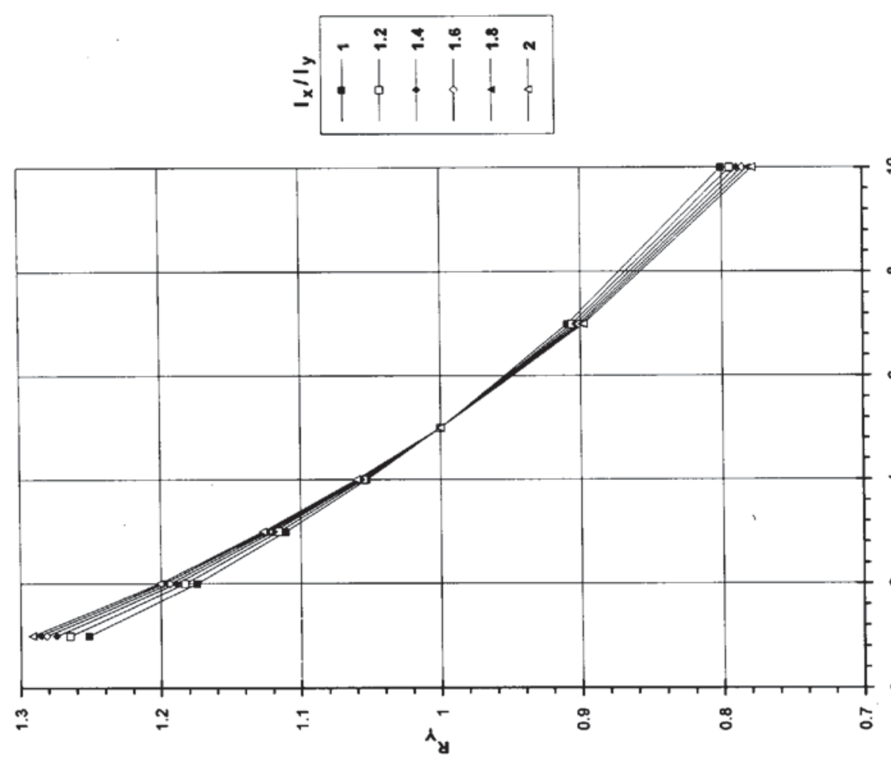
Graph 9.18 K_{Fxy} for M_{xy}. Stiffness correction coefficients for Zone F, curves for $l_x/l_y = 1$, 1.2, 1.4, 1.6, 1.8 and 2.

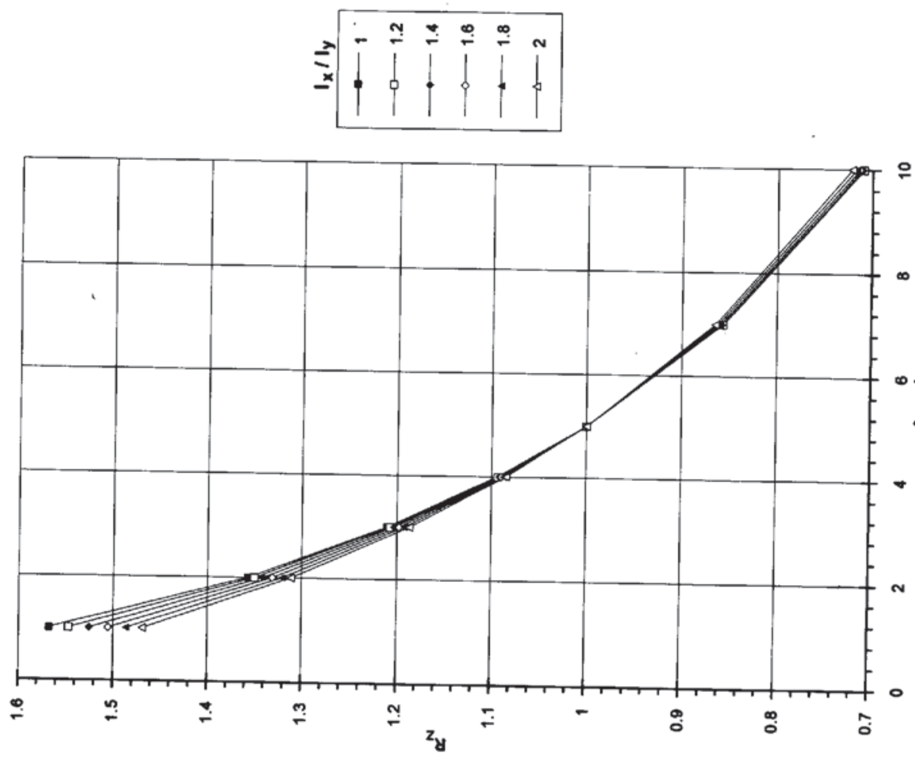


Graph 9.20 R_z for M_z. Stiffness correction coefficients for Column 1.

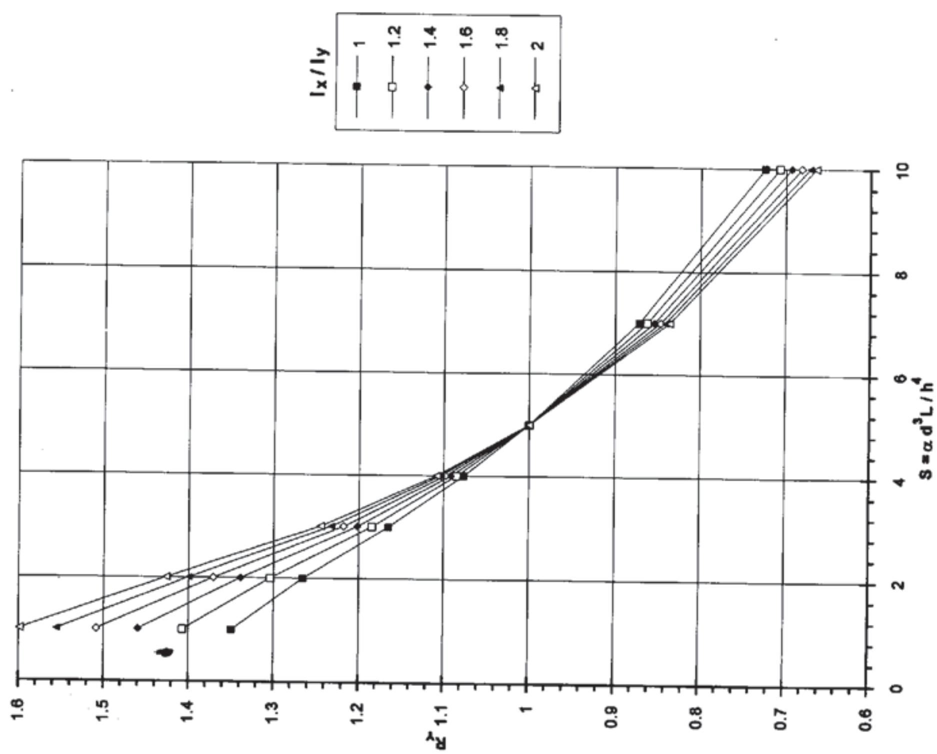


Graph 9.19 R_y for M_y. Stiffness correction coefficients for Column 1.



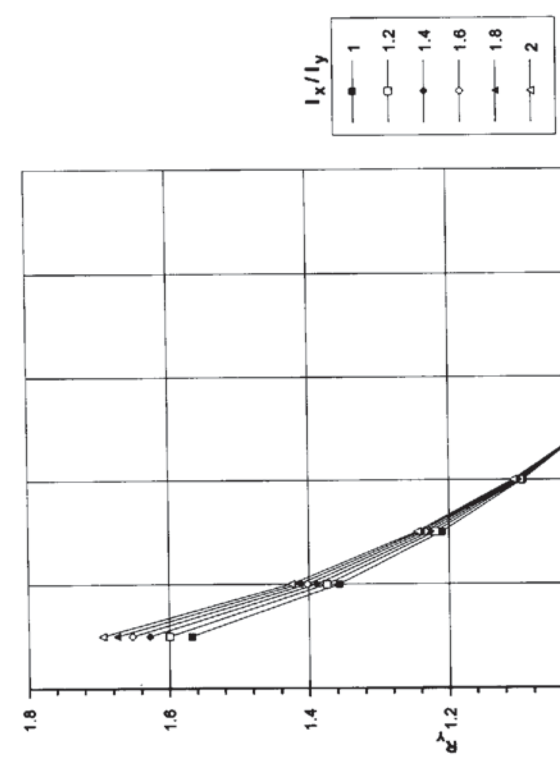


Graph 9.22 R_z for M_z. Stiffness correction coefficients for Column 2.

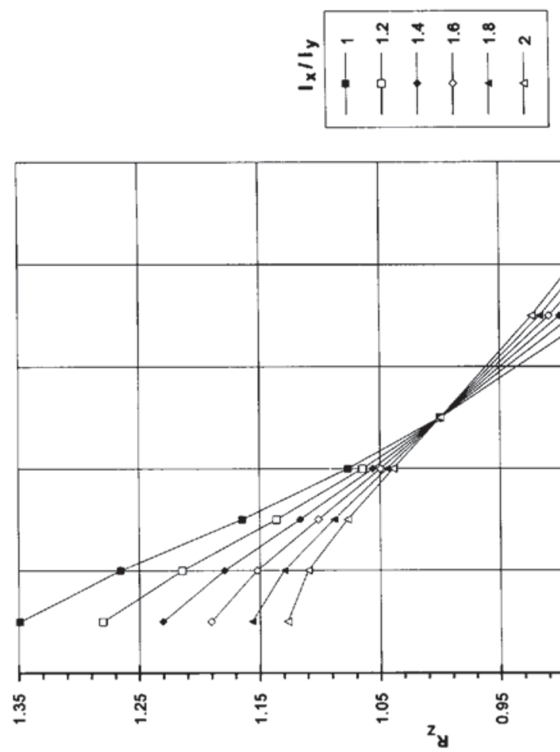


Graph 9.21 R_y for M_y. Stiffness correction coefficients for Column 2.

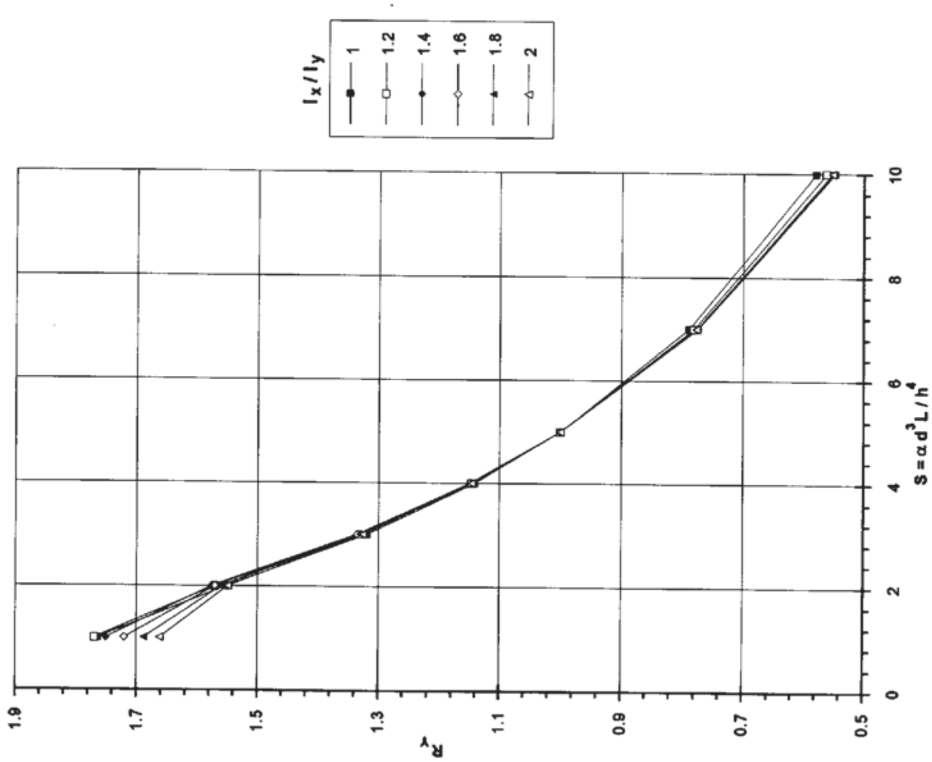
Graph 9.23 R_y for M_y. Stiffness correction coefficients for Column 3.



Graph 9.24 R_z for M_z. Stiffness correction coefficients for Column 3.



Graph 9.25 R_y for M_y. Stiffness correction coefficients for Column 4.



Graph 9.26 R_z for M_z. Stiffness correction coefficients for Column 4.