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Material Properties

Characteristic strength of concrete, f_{cu} / f_{ck} | f_c' (f_{cu} ≤ 60N/mm^2; H!	35	28	N/mm^2	OK
Yield strength of longitudinal steel, f_y	Higher	500	N/mm^2	
Yield strength of shear link steel, f_{yv}	Higher	500	N/mm^2	
Type of concrete and density, ρ_c	Normal Weight	25	kN/m^3	

Slab Parameters

Shorter span (defined as in x), l_x (number affects slab l_x mo	Multi Span	10.000	m	
Longer span (defined as in y), l_y (number affects slab l_y mor	Multi Span	10.000	m	OK
Slab support conditions (affects moments, shear, deflection	Continuous - Continuous End			
Panel (affects moments, shear, supports for edge beam)	Interior			
Overall slab depth, h_{slab} (l/27 s/s; l/36 cont; l/7-l/10 cant)		400	mm	
Cover to all reinforcement, cover (usually MAX(25, ϕ) internal; 40 external)		25	mm	
Effective depth to sagging steel in x, $d_{x,s}$ = h_{slab} - cover - ϕ_{sy} - ϕ_{sx}/2		351	mm	
Effective depth to CS sagging steel in y, $d_{y,s,c}$ = h_{slab} + d_{dp} - cover - ϕ_{sy}/2		367	mm	
Effective depth to MS sagging steel in y, $d_{y,s,m}$ = h_{slab} - cover - ϕ_{sy}/2		367	mm	
Effective depth to CS hogging steel in x, $d_{x,h,c}$ = h_{slab} + d_{dp} - cover - ϕ_{link} - ϕ_{hy} -		335	mm	
Effective depth to MS hogging steel in x, $d_{x,h,m}$ = h_{slab} - cover - ϕ_{link} - ϕ_{hy} - ϕ_{hx}/2		335	mm	
Effective depth to CS hogging steel in y, $d_{y,h,c}$ = h_{slab} +d_{dp} - cover - ϕ_{link} - ϕ_{hy}/2		355	mm	
Effective depth to MS hogging steel in y, $d_{y,h,m}$ = h_{slab} - cover - ϕ_{link} - ϕ_{hy}/2		355	mm	

Note that the column strip reinforcement diameters have been assumed for the effective depth calcs, this effectively enforcing the simplicity of common planes of reinforcement for each of the layers.

Note that d_{dp} only incorporated in the above column strip hogging effective depth calcs if w_{dp} >= l_x/3.

Note that d_{dp} only incorporated in the above column strip sagging effective depth calcs if w_{dp} >= l_x/3 and bar

	MS	CS	
Sagging steel reinforcement diameter in x, ϕ_{sx}	12	16	mm
Sagging steel reinforcement pitch for resistance in x, p_{sx}	150	150	mm
Sagging steel area provided in x, $A_{s,prov,x,s}$ = (π.ϕ_{sx}^2/4)/p_{sx}	754	1340	mm^2/m
Sagging steel reinforcement diameter in y, ϕ_{sy}	12	16	mm
Sagging steel reinforcement pitch for resistance in y, p_{sy}	150	150	mm
Sagging steel area provided in y, $A_{s,prov,y,s}$ = (π.ϕ_{sy}^2/4)/p_{sy}	754	1340	mm^2/m
Hogging steel reinforcement diameter in x, ϕ_{hx}	12	20	mm
Hogging steel reinforcement pitch for resistance in x, p_{hx}	150	150	mm
Hogging steel area provided in x, $A_{s,prov,x,h}$ = (π.ϕ_{hx}^2/4)/p_{hx}	754	2094	mm^2/m
Hogging steel reinforcement diameter in y, ϕ_{hy}	12	20	mm
Hogging steel reinforcement pitch for resistance in y, p_{hy}	150	150	mm
Hogging steel area provided in y, $A_{s,prov,y,h}$ = (π.ϕ_{hy}^2/4)/p_{hy}	754	2094	mm^2/m

Shear link diameter, ϕ_{link}						10	mm
Pitch of links, $s_{v,2/3}$					225	225	mm
No. of links, $n_{l,2/3}$	74	101	76	120	57	57	
Area of links, $A_{sv,prov,2/}$	5812	7933	5969	9425	4477	4477	mm^2
Pitch of links, $s_{v,4/5}$					225	225	mm
No. of links, $n_{l,4/5}$	129	157	163	207	57	57	
Area of links, $A_{sv,prov,4/}$	10132	12331	12802	16258	4477	4477	mm^2

Slab Drop and Slab Band and Column Head Concepts — Note

*Note **slab drops** (valid only if w_{dp} ≥ l_x/3) enhance the punching shear capacity and the column strip hogging moment capacity in x and y enabling thinner slabs beyond the drops.*

***Slab bands** differ from slab drops in the sense that the column strip sagging moment capacity in y (but not in x) is also enhanced on top of the benefits of slab drops.*

***Column heads** enhance punching shear capacity only.*

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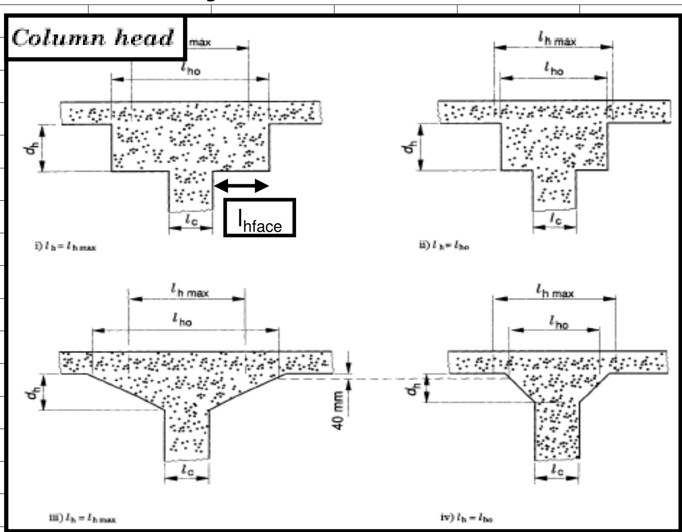
				BS8110
Slab Loading (Plan Loading)	Elastic Moments Effects			
(Internal elev load must be checked on effective widths [span/(5 or 7.14)] within slab depth)				
Live load, LL		5.00	kPa	
Superimposed dead load, SDL_{plan}		1.00	kPa	
Dead load of slab, DL = $h_{slab}.\rho_c + [w_{dp}^2$ or $w_{dp}.l_y]/(l_x.l_y).d_{dp}.\rho_c$		10.00	kPa	
ULS slab loading, $\omega_{ULS,slab}$ (a.k.a. n) = $1.4(DL+SDL_{plan})+1.6LL$	22.90	**23.40**	kPa	**OK**
Edge Loading (Elevation Loading)	Elastic Moments Effects			
Superimposed dead load on edge beam spanning in x direction, $SDL_{elev,x}$		0.00	kN/m	
Superimposed dead load on edge beam spanning in y direction, $SDL_{elev,y}$		0.00	kN/m	
Column Parameters				
Section type	Rectangular			
Depth, h (rectangular) or diameter, D (circular)	Parallel to Edge	800	mm	
Width, b (rectangular) or N/A (circular)	Perpendicular to Edge	800	mm	
Limitations of Moment Transfer Into Edge Column				
Width of edge column strip for moment transfer, $b_{e,a}$		1500	mm	
Slab Drop and Slab Band Parameters				
Slab drop depth, d_{dp} (excluding slab depth h_{slab})	Not Banded	0	mm	
Slab drop width, w_{dp} (usually >= $l_x/3$ when employed)		0.000	m	**N/A**
Note that the self weight of the slab drop is accounted in the design of the slab.				
Column Head Parameters				
Column head effective depth (rectangular), $l_{h,h}$ = MIN ($l_{h0,h}$, $l_{hmax,h}$) or effective		800	mm	
Column head effective width (rectangular), $l_{h,b}$ = MIN ($l_{h0,b}$, $l_{hmax,b}$) or N/A (circ		800	mm	
Column head dimension beyond column face, l_{hface}		0	mm	
Column head depth, d_h		0	mm	
Column head actual depth (rectangular), $l_{h0,h}$ = h + (1 or 2).l_{hface} o		800	mm	
Column head actual width (rectangular), $l_{h0,b}$ = b + (1 or 2).l_{hface} or		800	mm	
Column head maximum depth (rectangular), $l_{hmax,h}$ = h + 2.(d_h-40		720	mm	
Column head maximum width (rectangular), $l_{hmax,b}$ = b + 2.(d_h-40)		720	mm	
Note that the self weight of the column head is to be accounted in the design of the column, not the slab.				



$$l_{h\,max} = l_c + 2(d_h - 40)$$

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Parameters of Edge Beam Spanning in x Direction

Downstand edge beam ?		No	
(obtains relevant values from the two sections below)			
Width (no downstand), $b_{eff,x}$ or web width (with downstand), $b_{w,edge,x}$		700	mm
Dead load on edge beam downstand, $DL_{edge,x}$		0.00	kN/m
Sag moment edge beam, $M_{sag,edge,x}$		49	kNm
Hog moment edge beam, $M_{hog,edge,x}$		81	kNm
Shear edge beam, $V_{edge,x}$		54	kN
Span (for effective width and deflection calcs) = l_x		10.000	m
Available beam spacing (effective width calcs) = $l_y/2$		5.000	m
Sag section type		Rect - continuous	
Hog section type		Rect - continuous	
Overall depth, $h_{edge,x}$		400	mm
Effective width, $b_{eff,x}$ = span/10 if single span, span/14.29 if multi-span		700	mm
Dead load excluding downstand, $\omega_{edge,DL,x}$ = ($b_{eff,x}$ or $b_{w,edge,x}$ + $b_{eff,x}$).h_{slab}.ρ_c		7.00	kN/m

With Downstand Depth

Downstand depth of edge beam (excluding slab), $h_{d,edge,x}$	200	mm
Width of edge beam, $b_{w,edge,x}$	300	mm
Dead load on edge beam downstand, $DL_{edge,x}$ = $h_{d,edge,x} b_{w,edge,x} \rho_c$	1.50	kN/m
Sag section type	L - continuous	
Hog section type	Rect - continuous	
Overall depth, $h_{edge,x}$ (downstand + slab)	600	mm

Without Downstand Depth

Downstand depth of edge beam (excluding slab), $h_{d,edge,x}$ = 0.0	0	mm
Width of edge beam, $b_{w,edge,x}$ = 0.0	0	mm
Dead load on edge beam downstand, $DL_{edge,x}$ = 0.0	0.00	kN/m
Sag section type	Rect - continuous	
Hog section type	Rect - continuous	
Overall depth, $h_{edge,x}$ (slab)	400	mm

For sagging: tension steel diameter, $\phi_{t,sag}$ and number	16	6	
For sagging: compression steel diameter, $\phi_{c,sag}$ and number	None	6	
For sagging: add cover to compression steel, $cover_{add,c,sag}$ = ϕ_{hy}		20	mm
For hogging: tension steel diameter, $\phi_{t,hog}$ and number	16	6	
For hogging: add cover to tensile steel, $cover_{add,t,hog}$ = $cover_{add,c,sag}$		20	mm
For hogging: compression steel diameter, $\phi_{c,hog}$ and number	None	6	
Link diameter ϕ_{link}, number and pitch 10	2	200	mm
For sagging: number of layers of tensile steel, $n_{layers,tens,sag}$		1	layer(s)
For sagging: number of layers of compression steel, $n_{layers,comp,sag}$		1	layer(s)
Ratio β_b=1.2 (sagging) or 0.8 (hogging) unless single span or conti	1.0	1.0	
For hogging: number of layers of tensile steel, $n_{layers,tens,hog}$		1	layer(s)
For hogging: number of layers of compression steel, $n_{layers,comp,hog}$		1	layer(s)

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Parameters of Edge Beam Spanning in y Direction

Downstand edge beam ?	No	
(obtains relevant values from the two sections below)		
Width (no downstand), $b_{eff.y}$ or web width (with downstand), $b_{w,edge,y}$	700	mm
Dead load on edge beam downstand, $DL_{edge,y}$	0.00	kN/m
Sag moment edge beam, $M_{sag,edge,y}$	49	kNm
Hog moment edge beam, $M_{hog,edge,y}$	81	kNm
Shear edge beam, $V_{edge,y}$	54	kN
Span (for effective width and deflection calcs) = l_y	10.000	m
Available beam spacing (effective width calcs) = $l_x/2$	5.000	m
Sag section type	Rect - continuous	
Hog section type	Rect - continuous	
Overall depth, $h_{edge,y}$	400	mm
Effective width, $b_{eff,y}$ = span/10 if single span, span/14.29 if multi-span	700	mm
Dead load excluding downstand, $\omega_{edge,DL,y}$ = ($b_{eff,y}$ or $b_{w,edge,y}$ + $b_{eff,y}$).h_{slab}.ρ_c	7.00	kN/m

With Downstand Depth

Downstand depth of edge beam (excluding slab), $h_{d,edge,y}$	200	mm
Width of edge beam, $b_{w,edge,y}$	300	mm
Dead load on edge beam downstand, $DL_{edge,y}$ = $h_{d,edge,y} b_{w,edge,y} \rho_c$	1.50	kN/m
Sag section type	L - continuous	
Hog section type	Rect - continuous	
Overall depth, $h_{edge,y}$ (downstand + slab)	600	mm

Without Downstand Depth

Downstand depth of edge beam (excluding slab), $h_{d,edge,y}$ = 0.0	0	mm
Width of edge beam, $b_{w,edge,y}$ = 0.0	0	mm
Dead load on edge beam downstand, $DL_{edge,y}$ = 0.0	0.00	kN/m
Sag section type	Rect - continuous	
Hog section type	Rect - continuous	
Overall depth, $h_{edge,y}$ (slab)	400	mm

For sagging: tension steel diameter, $\phi_{t,sag}$ and number		16	6	
For sagging: compression steel diameter, $\phi_{c,sag}$ and number		None	6	
For hogging: tension steel diameter, $\phi_{t,hog}$ and number		16	6	
For hogging: compression steel diameter, $\phi_{c,hog}$ and number		None	6	
Link diameter ϕ_{link}, number and pitch	10	2	200	mm
For sagging: number of layers of tensile steel, $n_{layers,tens,sag}$			1	layer(s)
For sagging: number of layers of compression steel, $n_{layers,comp,sag}$			1	layer(s)
Ratio β_b=1.2 (sagging) or 0.8 (hogging) unless single span or conti		1.0	1.0	
For hogging: number of layers of tensile steel, $n_{layers,tens,hog}$			1	layer(s)
For hogging: number of layers of compression steel, $n_{layers,comp,hog}$			1	layer(s)

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Utilisation Summary (Slab)

Item				UT	Remark
$M_{sag,lx,m}$				96%	OK
$M_{sag,lx,c}$				66%	OK
$M_{hog,lx,m}$				93%	OK
$M_{hog,lx,c}$				104%	NOT OK
$M_{sag,ly,m}$				92%	OK
$M_{sag,ly,c}$				63%	OK
$M_{hog,ly,m}$				87%	OK
$M_{hog,ly,c}$				97%	OK
% Min sag reinforcement $l_{x,m}$ utilisation				69%	OK
% Min sag reinforcement $l_{x,c}$ utilisation				39%	OK
% Min hog reinforcement $l_{x,m}$ utilisation				69%	OK
% Min hog reinforcement $l_{x,c}$ utilisation				25%	OK
% Min sag reinforcement $l_{y,m}$ utilisation				69%	OK
% Min sag reinforcement $l_{y,c}$ utilisation				39%	OK
% Min hog reinforcement $l_{y,m}$ utilisation				69%	OK
% Min hog reinforcement $l_{y,c}$ utilisation				25%	OK
Limitations of moment transfer into edge column				53%	OK
Punching shear at colu	51%	49%	92%	92%	OK
Punching shear at firs	49%	98%	294%	294%	NOT OK
Punching shear at sec	37%	38%	95%	95%	OK
Punching shear at thir	36%	86%	70%	86%	OK
Punching shear at fou	82%	63%	57%	82%	OK
Deflection requirements				74%	OK
Total utilisation continuous slab				**294%**	**NOT OK**
Detailing requirements				**OK**	

Utilisation Summary (Beam)

Automatic design — **All Beams**

Item	UT	Detailing	Remark	
Edge beam x sagging	N/A	N/A	N/A	EBeamx Sag
Edge beam x hogging	N/A	N/A	N/A	EBeamx Hog
Edge beam y sagging	N/A	N/A	N/A	EBeamy Sag
Edge beam y hogging	N/A	N/A	N/A	EBeamy Hog

Overall Utilisation Summary

Overall utilisation	**294%**
Overall detailing requirements	**OK**

% Column strip sag reinforcement in x and y	0.34	0.34	%
% Column strip hog reinforcement x and y	0.52	0.52	%
% Middle strip sag reinforcement in x and y	0.19	0.19	%
% Middle strip hog reinforcement x and y	0.19	0.19	%
Estimated steel reinforcement quantity (130 – 220kg/m^3)		97	kg/m^3

[7.850 . ($A_{s,prov,x,s}$ +$A_{s,prov,y,s}$ +$A_{s,prov,x,h}$ +$A_{s,prov,x,h}$) / h_{slab}]; No curtailment; No laps; Links ignored;

Estimated steel reinforcement quantity (130 – 220kg/m^3)	136	kg/m^3	*IStructE*

[11.0 . ($A_{s,prov,x,s}$ +$A_{s,prov,y,s}$ +$A_{s,prov,x,h}$ +$A_{s,prov,x,h}$) / h_{slab}]; Curtailment; Laps; Links ignored; Averag

[Note that steel quantity in kg/m^3 can be obtained from 110.0 x % rebar];

Material cost:	concrete, c	180	units/m^3	steel, s	4500	units/tonne

Reinforced concrete material cost = [c+(est. rebar quant).s].h_{slab}	317	units/m^2

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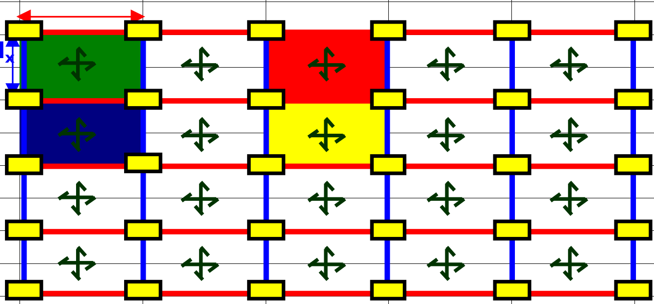
Plan Layout

Multi-Span l_x Multi-Span l_y Floor Plate



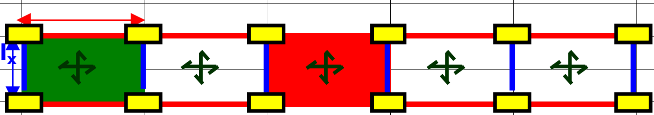
Relevant Panels

Interior	(yellow)
Edge for Span in x Direction	(red)
Edge for Span in y Direction	(blue)
Corner	(green)

Construction Type	***Support Conditions***
Continuous (Simple or Cont End)	*Continuous*

Number of spans in l_x	*Multi-span*
Number of spans in l_y	*Multi-span*

Single-Span l_x Multi-Span l_y Floor Plate



Relevant Panels

Interior	N/A
Edge for Span in x Direction	(red)
Edge for Span in y Direction	N/A
Corner	(green)

Construction Type	***Support Conditions***
Continuous (Simple or Cont End)	*Continuous*

Number of spans in l_x	*Single-span*
Number of spans in l_y	*Multi-span*

Average col and mid strips;

ge col and mid strips;

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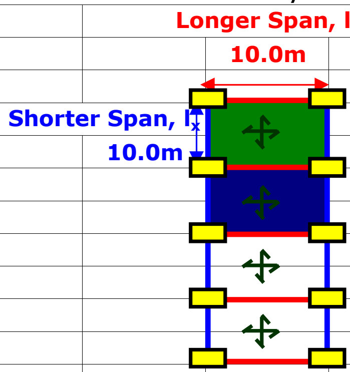
Multi-Span l_x Single-Span l_y Floor Plate

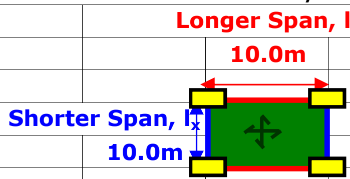


Relevant Panels	*Interior*	N/A
	Edge for Span in x Direction	N/A
	Edge for Span in y Direction	
	Corner	

Construction Type	***Support Conditions***
Continuous (Simple or Cont End)	*Continuous*

Number of spans in l_x	*Multi-span*
Number of spans in l_y	*Single-span*

Single-Span l_x Single-Span l_y Floor Plate



Relevant Panels	*Interior*	N/A
	Edge for Span in x Direction	N/A
	Edge for Span in y Direction	N/A
	Corner	

Construction Type	***Support Conditions***
Continuous (Simple or Cont End)	*Continuous*

Number of spans in l_x	*Single-span*
Number of spans in l_y	*Single-span*

Note that simple or continuous end slab support conditions refer to the end supports of multi-span continuous slabs and not single-span slabs, where the end connection is continuous.

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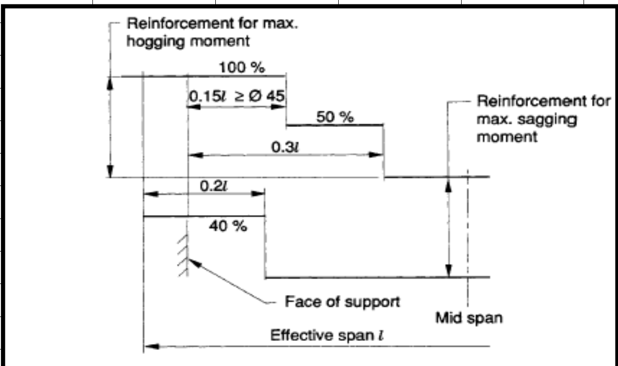
<u>*BS8110*</u>

Assumptions and Limitations

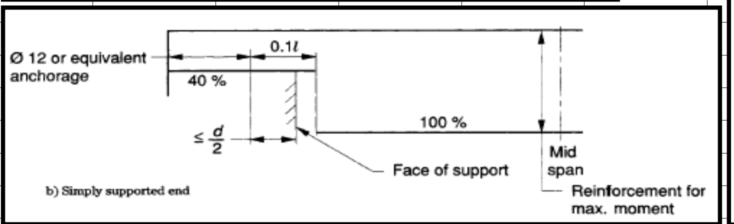
1 *Moment effects for slabs may be calculated based on redistributed effects or elastic effects.*
2 *Moment effects for beams may be calculated based on redistributed effects or elastic effects.*

Detailing Instructions

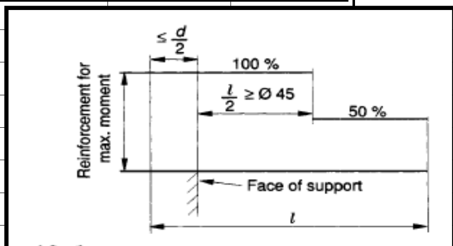


a) Continuous member (approximately equal spans using simplified load arrangement)

Note consideration should be given to the fact that although detailing rules allow the reduction of hogging steel from 100% to 50% beyond 0.15L, punching shear requirements may dictate as the % of tensile steel may have to be maintained at 100% until 0.3L or further to provide the required shear strength over all the shear perimeters checked. This indeed is the assumption herewith;



b) Simply supported end



c) Cantilever

Detailing Steel Positions

Note that the main slab reinforcement in x is assumed to be interior to main slab reinforcement in y;
Note that the main edge beam in x reinforcement is assumed to be interior to main slab reinforcement in y;
Note that the main edge beam in y reinforcement is assumed to be at same level as main slab reinforcement in

Note the same cover to all reinforcement used for the slab is used for the beam;

Note that where relevant, the additional cover has been added to only the top surface of the edge beam, in effect assuming that there is a downstand edge beam.

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Structural Analysis Slab

Design ULS total load for one panel, F = n.l_x.l_y **2340** kN

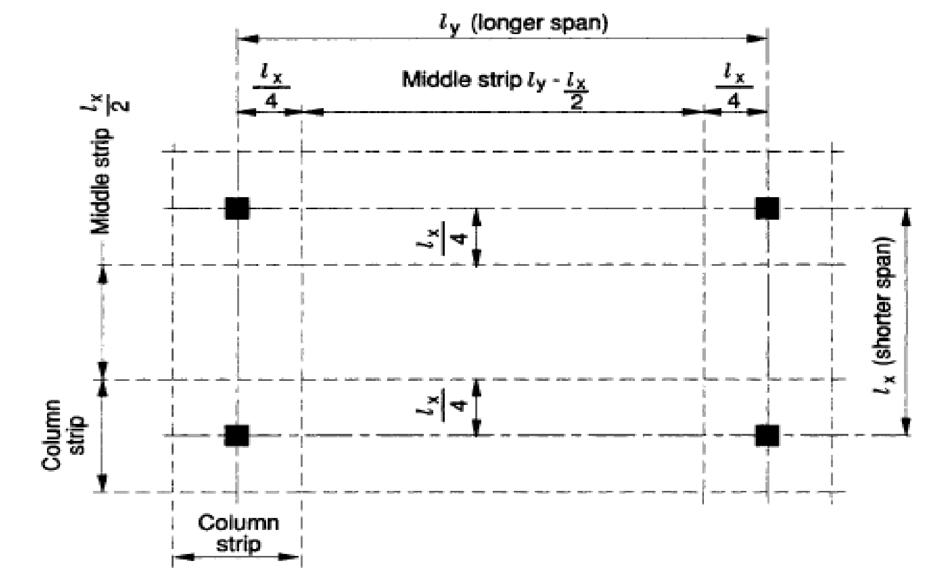
Table 11 Ultimate bending moment and shear force in flat slabs

	End support/slab connection				At first interior support	Middle of interior spans	Interior supports
	Simple		Continuous				
	At outer support	Near middle of end span	At outer support	Near middle of end span			
Moment	0	0.086*Fl*	- 0.04*Fl**	0.075*Fl* †	- 0.086*Fl*	0.063*Fl*	- 0.063*Fl*
Shear	0.4*F*	–	0.46*F*	–	0.6*F*	–	0.5*F*
Total column moments	0.04*Fl*	–	–	–	0.022*Fl*	–	0.022*Fl*

where F is the total design ultimate load on a panel bounded by four columns and *l* is the effective span.
Note: The moments at supports taken from this table may be reduced by 0.015Fh_c.
Allowance has been made for 20% redistribution as allowed in BS 8110.
*These moments may have to be reduced to be consistent with the capacity to transfer moments to the columns. The midspan moments † must then be increased correspondingly.

If a flat slab has at least three spans in each direction and the ratio of the longest span to the shortest does not exceed 1.2, the maximum values of the bending moments and shear forces may be obtained from Table 11.

y;



a) Slab without drops

Table 3.12 — Ultimate bending moment and shear forces in one-way spanning slabs

	End support/slab connection				At first interior support	Middle interior spans	Interior supports
	Simple		Continuous				
	At outer support	Near middle of end span	At outer support	Near middle of end span	–0.125Fl	0.050Fl#PL	–0.083Fl
Moment	0	0.086Fl	–0.04Fl	0.075Fl	– 0.086Fl	0.063Fl	–0.063Fl
Shear	0.4F	0.080Fl	0.46F	0.080Fl	0.6F	—	0.5F

NOTE F is the total design ultimate load (1.4G_k + 1.6Q_k);
l is the effective span.

Note elastic moment effects. #PL

Note allowance has been made in this ta

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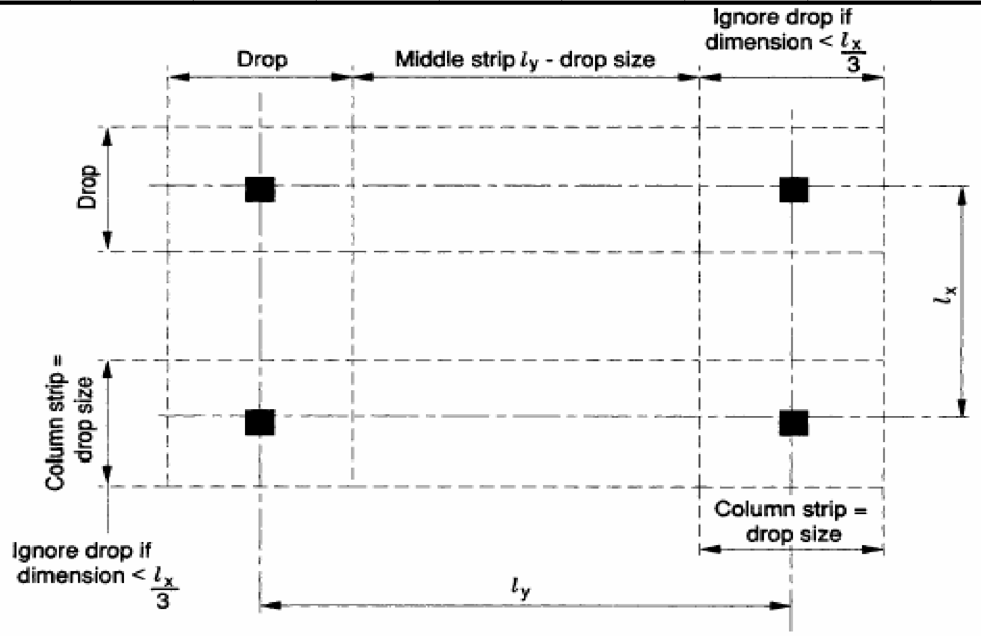

b) Slab with drops

Shorter Span, l_x

Width of column strip, $w_{1,lx}$ = IF (w_{dp}<l_x/3, l_x/2, w_{dp})	5.000	m
Width of middle strip, $w_{2,lx} = l_y - w_{1,lx}$	5.000	m
Sag moment for s/s case = 0.125F.l_x *(single span)*	2925	kNm
Hog moment for s/s case = 0.0625F.l_x *(single span continuous - simple or continuous end)*	1463	kNm
Shear for s/s case = 0.5F *(single span)*	1170	kN
Column moment for s/s case = 0.04Fl_x *(single span)*	936	kNm

	Simple End		**Continuous End**		**Interior**			
	At outer support	Near middle of end span	At outer support	Near middle of end span	At first interior support	Middle of interior spans	Interior supports	
Moment	0	1872	936	1872	2925	1170	1942	kNm
Shear	936	N/A	1076	N/A	1404	N/A	1170	kN
Col Mnt	936	N/A	N/A	N/A	515	N/A	515	kNm

Note that for edge panels, the shear force has been calculated for the less critical outer support instead of the first interior support because the SDL will be more critical here due to the external cladding.
Note that l above refers to l_x.

Sag moment, $M_{sag,lx}$	**1170**	kNm
Hog moment, $M_{hog,lx}$	**1942**	kNm
Shear, V_{lx}	**1170**	kN
Column moment, $M_{col,lx}$	**515**	kNm
Sag moment mid strip, $M_{sag,lx,m}=0.45w_{2,lx}/(l_y-l_x/2).M_{sag,lx}$	**527**	kNm
Sag moment col strip, $M_{sag,lx,c}=[1-0.45w_{2,lx}/(l_y-l_x/2)].M_{sag,lx}$	**644**	kNm
Hog moment mid strip, $M_{hog,lx,m}=0.25w_{2,lx}/(l_y-l_x/2).M_{hog,lx}$	**486**	kNm
Hog moment col strip, $M_{hog,lx,c}=[1-0.25w_{2,lx}/(l_y-l_x/2)].M_{hog,lx}$	**1457**	kNm

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Longer Span, l_y

Width of column strip, $w_{1,ly}$ = IF (w_{dp}<l_x/3, l_x/2, w_{dp}) 5.000 m

Width of middle strip, $w_{2,ly} = l_x - w_{1,ly}$ 5.000 m

Sag moment for s/s case = 0.125F.l_y *(single span)* 2925 kNm

Hog moment for s/s case = 0.0625F.l_y *(single span continuous - simple or continuous end)* 1463 kNm

Shear for s/s case = 0.5F *(single span)* 1170 kN

Column moment for s/s case = 0.04Fl_y *(single span)* 936 kNm

	Simple End		Continuous End		Interior			
	At outer support	Near middle of end span	At outer support	Near middle of end span	At first interior support	Middle of interior spans	Interior supports	
Moment	0	1872	936	1872	2925	1170	1942	kNm
Shear	936	N/A	1076	N/A	1404	N/A	1170	kN
Col Mnt	936	N/A	N/A	N/A	515	N/A	515	kNm

Note that for edge panels, the shear force has been calculated for the less critical outer support instead of the first interior support because the SDL will be more critical here due to the external cladding.

Note that l above refers to l_y.

Sag moment, $M_{sag,ly}$ **1170** kNm

Hog moment, $M_{hog,ly}$ **1942** kNm

Shear, V_{ly} **1170** kN

Column moment, $M_{col,ly}$ **515** kNm

Sag moment mid strip, $M_{sag,ly,m}=0.45w_{2,ly}/(l_x/2).M_{sag,ly}$ **527** kNm

Sag moment col strip, $M_{sag,ly,c}=[1-0.45w_{2,ly}/(l_x/2)].M_{sag,ly}$ **644** kNm

Hog moment mid strip, $M_{hog,ly,m}=0.25w_{2,ly}/(l_x/2).M_{hog,ly}$ **486** kNm

Hog moment col strip, $M_{hog,ly,c}=[1-0.25w_{2,ly}/(l_x/2)].M_{hog,ly}$ **1457** kNm

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Structural Analysis Edge Beam Spanning in x Direction

Slab UDL on edge beam, $\omega_{edge,x}$ = assumed 1.4 x $\omega_{edge,DL,x}$	10	kN/m
ULS beam, $\omega_{ULS,edge,x}$ = $\omega_{edge,x}$+1.4SDL$_{elev,edge,x}$+1.4DL$_{edge,x}$	10	kN/m

Table 3.5 — Design ultimate bending moments and shear forces

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0	0.09Fl	–0.11Fl	0.07Fl	–0.08Fl –0.083Fl
Shear	0.45F	0.08Fl	0.6F –0.125Fl	0.05Fl[#PL]	0.55F

NOTE l is the effective span; F is the total design ...
No redistribution of the mo...

Note elastic moment effects. [#PL] *pattern loading factor 1.2;*

Note allowance has been made in this table for 20% moment redistribution;

Interior or End Beam ? Interior Beam

Note that the coefficients above are appropriate to the interior or edge panel as follows.

	Sag x	*Hog x*	*Shear x*
Interior Edge Beam	*0.050*	*0.083*	*0.550*
End Edge Beam	*0.080*	*0.125*	*0.600*
Single Span Edge Beam	*0.125*	*0.063*	*0.500*

Note that the beams are always continuous (unless single span) since monolithic with columns, and the slab is also always continuous (unless single span) in flat slabs.

Sag moment edge beam, $M_{sag,edge,x}$ = coeff.($\omega_{ULS,edge,x}$. l_x)l_x	**49**	kNm
Hog moment edge beam, $M_{hog,edge,x}$ = coeff.($\omega_{ULS,edge,x}$. l_x)l_x	**81**	kNm
Shear edge beam, $V_{edge,x}$ = coeff.($\omega_{ULS,edge,x}$. l_x)	**54**	kN

Structural Analysis Edge Beam Spanning in y Direction

Slab UDL on edge beam, $\omega_{edge,y}$ = assumed 1.4 x $\omega_{edge,DL,y}$	10	kN/m
ULS beam, $\omega_{ULS,edge,y}$ = $\omega_{edge,y}$+1.4SDL$_{elev,edge,y}$+1.4DL$_{edge,y}$	10	kN/m

Table 3.5 — Design ultimate bending moments and shear forces

	At outer support	Near middle of end span	At first interior support	At middle of interior spans	At interior supports
Moment	0	0.09Fl	–0.11Fl	0.07Fl	–0.08Fl –0.083Fl
Shear	0.45F	0.08Fl	0.6F –0.125Fl	0.05Fl[#PL]	0.55F

NOTE l is the effective span; F is the total design ...
No redistribution of the mo...

Note elastic moment effects. [#PL] *pattern loading factor 1.2;*

Note allowance has been made in this table for 20% moment redistribution;

Interior or End Beam ? Interior Beam

Note that the coefficients above are appropriate to the interior or edge panel as follows.

	Sag y	*Hog y*	*Shear y*
Interior Edge Beam	*0.050*	*0.083*	*0.550*
End Edge Beam	*0.080*	*0.125*	*0.600*
Single Span Edge Beam	*0.125*	*0.063*	*0.500*

Note that the beams are always continuous (unless single span) since monolithic with columns, and the slab is also always continuous (unless single span) in flat slabs.

Sag moment edge beam, $M_{sag,edge,y}$ = coeff.($\omega_{ULS,edge,y}$. l_y)l_y	**49**	kNm
Hog moment edge beam, $M_{hog,edge,y}$ = coeff.($\omega_{ULS,edge,y}$. l_y)l_y	**81**	kNm
Shear edge beam, $V_{edge,y}$ = coeff.($\omega_{ULS,edge,y}$. l_y)	**54**	kN

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Slab Moment Design

	M (kNm)		M/b (kNm/m)	
Sag moment, $M_{sag,lx,m}$	527	kNm	105	kNm/m
Sag moment, $M_{sag,lx,c}$	644	kNm	129	kNm/m
Hog moment, $M_{hog,lx,m}$	486	kNm	97	kNm/m
Hog moment, $M_{hog,lx,c}$	1457	kNm	291	kNm/m
Sag moment, $M_{sag,ly,m}$	527	kNm	105	kNm/m
Sag moment, $M_{sag,ly,c}$	644	kNm	129	kNm/m
Hog moment, $M_{hog,ly,m}$	486	kNm	97	kNm/m
Hog moment, $M_{hog,ly,c}$	1457	kNm	291	kNm/m

Ensure singly reinforced $K = M/bd^2f_{cu}$ $z = d\left\{0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)}\right\}$ z <=0.95d $A_s = \frac{M}{(0.95f_y)z}$

$K' = 0.156$ $K' = 0.402(\beta_b - 0.4) - 0.18(\beta_b - 0.4)^2$

Limit K' (sagging span x)	0.156	Limit K' (sagging span y)	0.156
Limit K' (hogging span x)	0.156	Limit K' (hogging span y)	0.156

	b	d	K	z	A_s/b	$A_{s,prov}$	UT	
$M_{sag,lx,m}$	5000	351	0.024	333	723	754	96%	OK
$M_{sag,lx,c}$	5000	351	0.030	333	883	1340	66%	OK
$M_{hog,lx,m}$	5000	335	0.025	318	698	754	93%	OK
$M_{hog,lx,c}$	5000	335	0.074	305	2188	2094	104%	NOT OK
$M_{sag,ly,m}$	5000	367	0.022	349	691	754	92%	OK
$M_{sag,ly,c}$	5000	367	0.027	349	845	1340	63%	OK
$M_{hog,ly,m}$	5000	355	0.022	337	659	754	87%	OK
$M_{hog,ly,c}$	5000	355	0.066	327	2041	2094	97%	OK

Note unless single span or continuous elastic whereby β_b = 1.00 and K' = 0.156, K' calculated with β_b = 1.20 (sagging) or 0.80 (hogging), however K' for $\beta_b \geq 0.90$ truncated at 0.156.
If K > K', then UT = 999%. Note that A_s/b and $A_{s,prov}$ above are in units of mm^2/m. Note that b is taken as the relevant middle or columns widths. Note that $A_{s,prov}$ is specified for both the middle and column strips.

3.7.3.1 *Column and middle strips*

The column and middle strips should be designed to withstand the design moments obtained from **3.7.2.** In general, two-thirds of the amount of reinforcement required to resist the negative design moment in the column strip should be placed in a width equal to half that of the column strip and central with the column.

% Min sag reinforcement $l_{x,m}$ (>= 0.0024bh G250; >= 0.0013bh G460)	0.19	%	
% Min sag reinforcement $l_{x,m}$ utilisation	69%		OK
% Min sag reinforcement $l_{x,c}$ (>= 0.0024bh G250; >= 0.0013bh G460)	0.34	%	
% Min sag reinforcement $l_{x,c}$ utilisation	39%		OK
% Min hog reinforcement $l_{x,m}$ (>= 0.0024bh G250; >= 0.0013bh G460)	0.19	%	
% Min hog reinforcement $l_{x,m}$ utilisation	69%		OK
% Min hog reinforcement $l_{x,c}$ (>= 0.0024b(h+d_{dp}) G250; >= 0.0013b(h+d_{dp})	0.52	%	
% Min hog reinforcement $l_{x,c}$ utilisation	25%		OK
% Min sag reinforcement $l_{y,m}$ (>= 0.0024bh G250; >= 0.0013bh G460)	0.19	%	
% Min sag reinforcement $l_{y,m}$ utilisation	69%		OK
% Min sag reinforcement $l_{y,c}$ (>= 0.0024b(h+d_{dp}) G250; >= 0.0013b(h+d_{dp}) (	0.34	%	
% Min sag reinforcement $l_{y,c}$ utilisation	39%		OK
% Min hog reinforcement $l_{y,m}$ (>= 0.0024bh G250; >= 0.0013bh G460)	0.19	%	
% Min hog reinforcement $l_{y,m}$ utilisation	69%		OK
% Min hog reinforcement $l_{y,c}$ (>= 0.0024b(h+d_{dp}) G250; >= 0.0013b(h+d_{dp})	0.52	%	
% Min hog reinforcement $l_{y,c}$ utilisation	25%		OK

Note that d_{dp} only incorporated in column strip hogging above if w_{dp} >= l_x/3.
Note that d_{dp} only incorporated in column strip sagging above if w_{dp} >= l_x/3 and banded.

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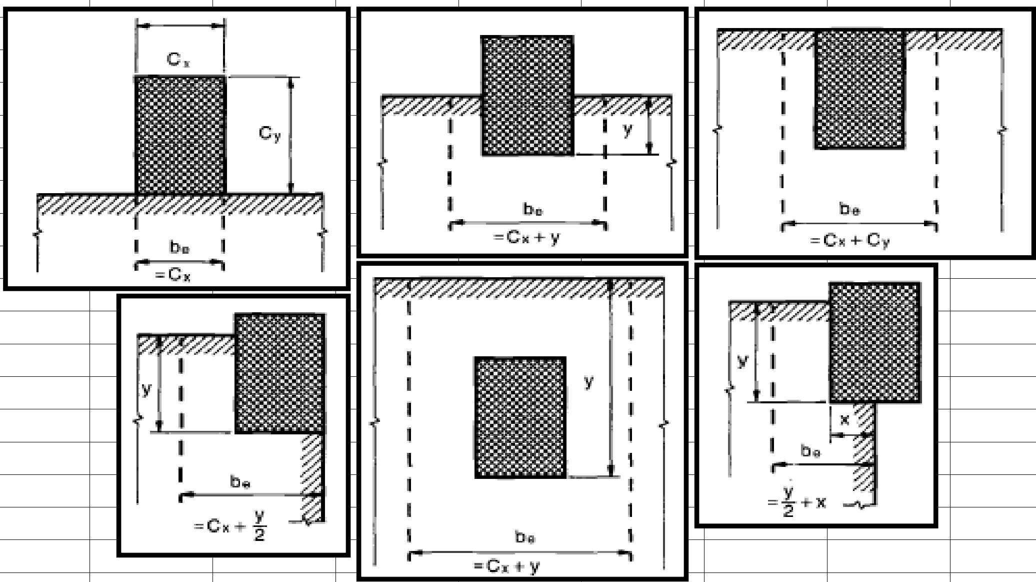
Limitations of Moment Transfer Into Edge Column

Maximum design moment that can be transferred to column by column strip

$M_{t\,max} = 0.15 b_e d^2 f_{cu}$	*Note $f_{cu} \leq 60N/mm^2$*	884	kNm

where

Width of strip, $b_{e,a}$	1500	mm
Adopted width of strip, b_e = MIN ($b_{e,a}$, $w_{1,lx}$, $w_{1,ly}$)	1500	mm



Effective depth for top steel in col strip, d = MIN ($d_{x,h,c}$, $d_{y,h,c}$)			335	mm
Bending moment to transfer into column at edges, $M_{t,act}$ = 0.04F.MAX(l_x,l_y)			936	kNm
Ensure moment transfer $M_{t,max}$ >= 0.5$M_{t,act}$	884	>=	468	kNm
Moment transfer utilisation 1-($M_{t,max}$-(0.5$M_{t,act}$))/$M_{t,max}$			53%	OK
Need to increase end span sag moment ? MAX(0, ($M_{t,act}$ - $M_{t,max}$)/2)			26	kNm

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Punching Shear (BS8110)

ULS design punching shear into column, V_t	2340	kN	
Note V_t = F (internal), F/2+SDL $_{elev,x/y}$.$l_{x/y}$ (edge), F/4+(SDL $_{elev,x}$.l_x +SDL $_{elev,y}$.l_y)/2 (corner).			
Area of column section, $A_{c1} = l_{h,b}.l_{h,h}$ (rectangular) or $\pi l_{h,D}{}^2/4$ (circular)	0.64	m^2	
Average effective depth of both rebar layers, $d = (d_{x,h,c} + d_{y,h,c})/2$ · Include	345	mm	
Note conservatively that d_{dp} should not be incorporated above when incorporated within $d_{x,h,c}$ and $d_{y,h,c}$ to cater for the reduced effective depth at shear perimeters beyond the slab drop widths.			
Area of tensile steel reinforcement provided, $A_{s,prov,x,h,c}$	2094	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,h,c}$	2094	mm^2/m	
Average area of tensile steel reinforcement provided, $A_{s,prov,h,c}$	2094	mm^2/m	
$\rho_w = 100A_{s,prov,h,c}/(1000.d)$	0.61	%	
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$	0.62	N/mm^2	cl.3.4.5.4
$\rho_w = 100A_{s,prov,h,c}/(1000.d) \leq 3$	0.61	%	cl.3.4.5.4
$f_{cu} = f_{cu} \leq 40N/mm^2$	35	N/mm^2	cl.3.4.5.4
$(400/d)^{1/4} \geq 0.67$	1.04		cl.3.4.5.4

Values of design concrete shear strength, v_c (N/mm^2) (table 3.8 of BS 8110)

$100A_s / b_v d$	Effective depth (mm) 125	150	175	200	225	250	300	400
≤ 0.15	0.45	0.43	0.41	0.40	0.39	0.38	0.36	0.34
0.25	0.53	0.51	0.49	0.47	0.46	0.45	0.43	0.40
0.50	0.67	0.64	0.62	0.60	0.58	0.56	0.54	0.50
0.75	0.77	0.73	0.71	0.68	0.66	0.65	0.62	0.57
1.00	0.84	0.81	0.78	0.75	0.73	0.71	0.68	0.63
1.50	0.97	0.92	0.89	0.86	0.83	0.81	0.78	0.72
2.00	1.06	1.02	0.98	0.95	0.92	0.89	0.86	0.80
≥ 3.00	1.22	1.16	1.12	1.08	1.05	1.02	0.98	0.91

For characteristic concrete strengths greater than 25 N/mm^2, the values in this table may be multiplied by $(f_{cu}/25)^{1/2}$. The value of f_{cu} should not be taken as greater than 40.

Column Face Perimeter

Shear force at column face, $V_1 = V_t - n.A_{c1}$	2325	kN	cl.3.7.6.1
Eff. shear force, $V_{eff,1}$ = (1.15 int., 1.40 edge column) . V_1	2674	kN	cl.3.7.6
Column face perimeter, u_1	3200	mm	cl.3.7.6.1

	Rectangular		Circular		
Internal column:	*$2.(l_{h,b}+l_{h,h})$*	*3200*	*$\pi.l_{h,D}$*	*N/A*	mm
Edge column:	*$2l_{h,b}+l_{h,h}$ or $2l_{h,h}+l_{h,b}$*	*2400*	*$\pi/4(\pi.l_{h,D})$*	*N/A*	mm
Corner column:	*$(l_{h,b}+l_{h,h})$*	*1600*	*$\pi.l_{h,D}/2$*	*N/A*	mm

Shear stress at column face perimeter, $v_1 = V_{eff,1} / u_1 d$ (< $0.8f_{cu}{}^{0.5}$ & $5N/mm^2$)	2.42	N/mm^2	cl.3.7.7.2
Ultimate shear strength, MIN{$0.8f_{cu}{}^{0.5}$ & $5.0N/mm^2$}	4.73	N/mm^2	cl.3.7.7.2
Ultimate shear stress utilisation	51%		OK

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					<u>*BS8110*</u>
First Shear Perimeter	*@1.50d*	***518***	*to*	*@0.00d*	***0*** mm	*cl.3.7.7.6*

Shear force 1.5d from column face, $V_2 = V_t$ - n.(A_{c2} ≤ {$l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l_y$			2261	kN	*cl.3.7.7.6*
	Rectangular		*Circular*		
Internal column:	*$(l_{h,b}+3d).(l_{h,h}+3d)$*	*3.37*	*$(l_{h,D}+3d)^2$*	*N/A*	m^2
Edge colum$_{h,b}$+1.5d).$(l_{h,h}+3d)$ or $(l_{h,h}+1.5d).(l_{h,b}+3d)$		*2.42*	*.$(l_{h,D}+3d)$*	*N/A*	m^2
Corner column:	*$(l_{h,b}+1.5d).(l_{h,h}+1.5d)$*	*1.74*	*$_{,D}+1.5d)^2$*	*N/A*	m^2
Eff. shear force, $V_{eff,2}$ = (1.15 int., 1.40 edge column) . V_2				**2600** kN	*cl.3.7.6*
Column first perimeter, u_2 ≤ {$2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2$}				7340 mm	*cl.3.7.7.6*
	Rectangular		*Circular*		
Internal column:	*$2.(l_{h,b}+l_{h,h})+12d$*	*7340*	*$4l_{h,D}+12d$*	*N/A*	mm
Edge column:	*$2l_{h,b}+l_{h,h}+6d$ or $2l_{h,h}+l_{h,b}+6d$*	*4470*	*$3l_{h,D}+6d$*	*N/A*	mm
Corner column:	*$(l_{h,b}+l_{h,h})+3d$*	*2635*	*$2l_{h,D}+3d$*	*N/A*	mm
Shear stress at column first perimeter, $v_2 = V_{eff,2} / u_2d$				**1.03** N/mm^2	*cl.3.7.7.3*

(Shear capacity enhancement by calculating v_d at 1.5d from support and comparing against unenhanced v_c as clause 3.7.7.6 BS8110 employed instead of calculating v_d at support and comparing against enhanced v_c within 1.5d of the support as clause 3.7.7.4 BS8110;)

Case $v_2 < v_c$		**INVALID**	**0.62**	*cl.3.7.7.6*
No links required.				
Case $v_c < v_2 < 1.6v_c$	**0.62**	**N/A**	**0.99**	*cl.3.7.7.5*
$\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}$ N/A	>=	N/A	mm^2	*cl.3.7.7.5*
	Note $f_{yv} \leq 460N/mm^2$			*Foreword*
Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$				
Case $1.6v_c < v_2 < 2.0v_c$	**0.99**	**VALID**	**1.24**	*cl.3.7.7.5*
$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v-v_c)ud}{0.95f_{yv}}$ 5812	>=	2828	mm^2	*cl.3.7.7.5*
	Note $f_{yv} \leq 460N/mm^2$			*Foreword*
Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$				
Case $v_2 > 2.0v_c$	**1.24**	**N/A**		*cl.3.7.7.5*

First shear perimeter shear utilisation	**49%**	**OK**

							BS8110
Second Shear Perimeter	@2.25d	776	to	@0.75d	259	mm	cl.3.7.7.6
Shear force 2.25d from column face, $V_3 = V_t$ - n.(A_{c3} ≤ {$l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l$					2210	kN	cl.3.7.7.6
		Rectangular		Circular			
Internal column:	$(l_{h,b}+4.5d).(l_{h,h}+4.5d)$	5.53		$_{,D}+4.5d)^2$	N/A	m^2	
Edge column:	$5d).(l_{h,h}+4.5d)$ or $(l_{h,h}+2.25d).(l_{h,b}+4.5d)$	3.71		$l_{h,D}+4.5d)$	N/A	m^2	
Corner column:	$(l_{h,b}+2.25d).(l_{h,h}+2.25d)$	2.48		$+2.25d)^2$	N/A	m^2	
Eff. shear force, $V_{eff,3}$ = (1.15 int., 1.40 edge column) . V_3					2542	kN	cl.3.7.6
Column second perimeter, u_3 ≤ {$2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2$}					9410	mm	cl.3.7.7.6
		Rectangular		Circular			
Internal column:	$2.(l_{h,b}+l_{h,h})+18d$	9410		$4l_{h,D}+18d$	N/A	mm	
Edge column:	$2l_{h,b}+l_{h,h}+9d$ or $2l_{h,h}+l_{h,b}+9d$	5505		$3l_{h,D}+9d$	N/A	mm	
Corner column:	$(l_{h,b}+l_{h,h})+4.5d$	3153		$2l_{h,D}+4.5d$	N/A	mm	
Shear stress at column second perimeter, $v_3 = V_{eff,3} / u_3d$					0.78	N/mm^2	cl.3.7.7.3

Case $v_3 < v_c$				INVALID	0.62		cl.3.7.7.6
No links required.							
Case $v_c < v_3 < 1.6v_c$			0.62	VALID	0.99		cl.3.7.7.5
$\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}$		7933	>=	2972		mm^2	cl.3.7.7.5
			Note $f_{yv} \leq 460N/mm^2$				Foreword
Note	$\Sigma A_{sv}\sin\alpha$	>	$0.4ud/0.95f_{yv}$				
Case $1.6v_c < v_3 < 2.0v_c$			0.99	N/A	1.24		cl.3.7.7.5
$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v-v_c)ud}{0.95f_{yv}}$		N/A	>=	N/A		mm^2	cl.3.7.7.5
			Note $f_{yv} \leq 460N/mm^2$				Foreword
Note	$\Sigma A_{sv}\sin\alpha$	>	$0.4ud/0.95f_{yv}$				
Case $v_3 > 2.0v_c$			1.24	N/A			cl.3.7.7.5

Second shear perimeter shear utilisation	37%		OK

									BS8110
Third Shear Perimeter			@3.00d	1035	to	@1.50d	518	mm	cl.3.7.7.6
Shear force 3.0d from column face, $V_4 = V_t$ - n.(A_{c4} ≤ {$l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l_y$							2147	kN	cl.3.7.7.6
				Rectangular		Circular			
Internal column:				$(l_{h,b}+6d).(l_{h,h}+6d)$	8.24	$(l_{h,D}+6d)^2$	N/A	m^2	
Edge column:		$(l_{h,b}+3d).(l_{h,h}+6d)$ or $(l_{h,h}+3d).(l_{h,b}+6d)$			5.27	$.(l_{h,D}+6d)$	N/A	m^2	
Corner column:				$(l_{h,b}+3d).(l_{h,h}+3d)$	3.37	$(l_{h,D}+3d)^2$	N/A	m^2	
Eff. shear force, $V_{eff,4}$ = (1.15 int., 1.40 edge column) . V_4							2469	kN	cl.3.7.6
Column third perimeter, u_4 ≤ {$2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2$}							11480	mm	cl.3.7.7.6
				Rectangular		Circular			
Internal column:				$2.(l_{h,b}+l_{h,h})+24d$	11480	$4l_{h,D}+24d$	N/A	mm	
Edge column:		$2l_{h,b}+l_{h,h}+12d$ or $2l_{h,h}+l_{h,b}+12d$			6540	$3l_{h,D}+12d$	N/A	mm	
Corner column:				$(l_{h,b}+l_{h,h})+6d$	3670	$2l_{h,D}+6d$	N/A	mm	
Shear stress at column third perimeter, $v_4 = V_{eff,4} / u_4d$							0.62	N/mm^2	cl.3.7.7.3
	Case $v_4 < v_c$						INVALID	0.62	cl.3.7.7.6
	No links required.								
	Case $v_c < v_4 < 1.6v_c$					0.62	VALID	0.99	cl.3.7.7.5
		$\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}$			10132	>=	3625	mm^2	cl.3.7.7.5
						Note f_{yv} ≤460N/mm^2			Foreword
		Note	$\Sigma A_{sv}\sin\alpha$	>	$0.4ud/0.95f_{yv}$				
	Case $1.6v_c < v_4 < 2.0v_c$					0.99	N/A	1.24	cl.3.7.7.5
		$\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v-v_c)ud}{0.95f_{yv}}$			N/A	>=	N/A	mm^2	cl.3.7.7.5
						Note f_{yv} ≤460N/mm^2			Foreword
		Note	$\Sigma A_{sv}\sin\alpha$	>	$0.4ud/0.95f_{yv}$				
	Case $v_4 > 2.0v_c$					1.24	N/A		cl.3.7.7.5
Third shear perimeter shear utilisation							36%		OK

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Fourth Shear Perimeter	@3.75d	**1294**	to	@2.25d	**776**	mm	cl.3.7.7.6
Shear force 3.75d from column face, $V_5 = V_t - n.(A_{c5} \le \{l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l$					2071	kN	cl.3.7.7.6
		Rectangular		Circular			
Internal column:		$(l_{h,b}+7.5d).(l_{h,h}+7.5d)$	11.48	$_{,D}+7.5d)^2$	N/A	m^2	
Edge column:		$75d).(l_{h,h}+7.5d)$ or $(l_{h,h}+3.75d).(l_{h,b}+7.5d)$	7.09	$l_{h,D}+7.5d)$	N/A	m^2	
Corner column:		$(l_{h,b}+3.75d).(l_{h,h}+3.75d)$	4.38	$_{,}+3.75d)^2$	N/A	m^2	
Eff. shear force, $V_{eff,5}$ = (1.15 int., 1.40 edge column) . V_5					2382	kN	cl.3.7.6
Column fourth perimeter, $u_5 \le \{2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2\}$					13550	mm	cl.3.7.7.6
		Rectangular		Circular			
Internal column:		$2.(l_{h,b}+l_{h,h})+30d$	13550	$4l_{h,D}+30d$	N/A	mm	
Edge column:		$2l_{h,b}+l_{h,h}+15d$ or $2l_{h,h}+l_{h,b}+15d$	7575	$3l_{h,D}+15d$	N/A	mm	
Corner column:		$(l_{h,b}+l_{h,h})+7.5d$	4188	$2l_{h,D}+7.5d$	N/A	mm	
Shear stress at column fourth perimeter, $v_5 = V_{eff,5} / u_5d$					0.51	N/mm^2	cl.3.7.7.3

Case $v_5 < v_c$			**VALID**	**0.62**	cl.3.7.7.6
No links required.					
Case $v_c < v_5 < 1.6v_c$		**0.62**	**N/A**	**0.99**	cl.3.7.7.5
$\Sigma A_{sv}\sin\alpha \ge \frac{(v-v_c)ud}{0.95f_{yv}}$	N/A	>=	N/A	mm^2	cl.3.7.7.5
		Note $f_{yv} \le 460N/mm^2$			*Foreword*
Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$					
Case $1.6v_c < v_5 < 2.0v_c$		**0.99**	**N/A**	**1.24**	cl.3.7.7.5
$\Sigma A_{sv}\sin\alpha \ge \frac{5(0.7v-v_c)ud}{0.95f_{yv}}$	N/A	>=	N/A	mm^2	cl.3.7.7.5
		Note $f_{yv} \le 460N/mm^2$			*Foreword*
Note $\Sigma A_{sv}\sin\alpha$ > $0.4ud/0.95f_{yv}$					
Case $v_5 > 2.0v_c$		**1.24**	**N/A**		cl.3.7.7.5

Fourth shear perimeter shear utilisation	**82%**	**OK**

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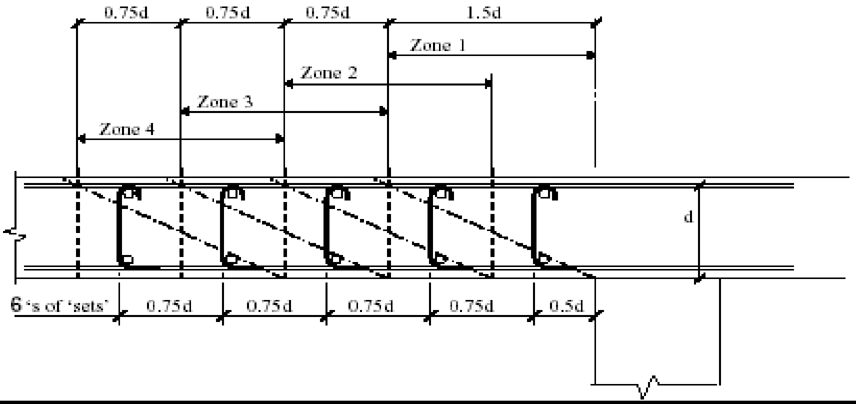
BS8110

Shear Reinforcement in Flat Slabs (BS 8110 Cl 3.7.7; EC2, Cl 5.4.3.3)

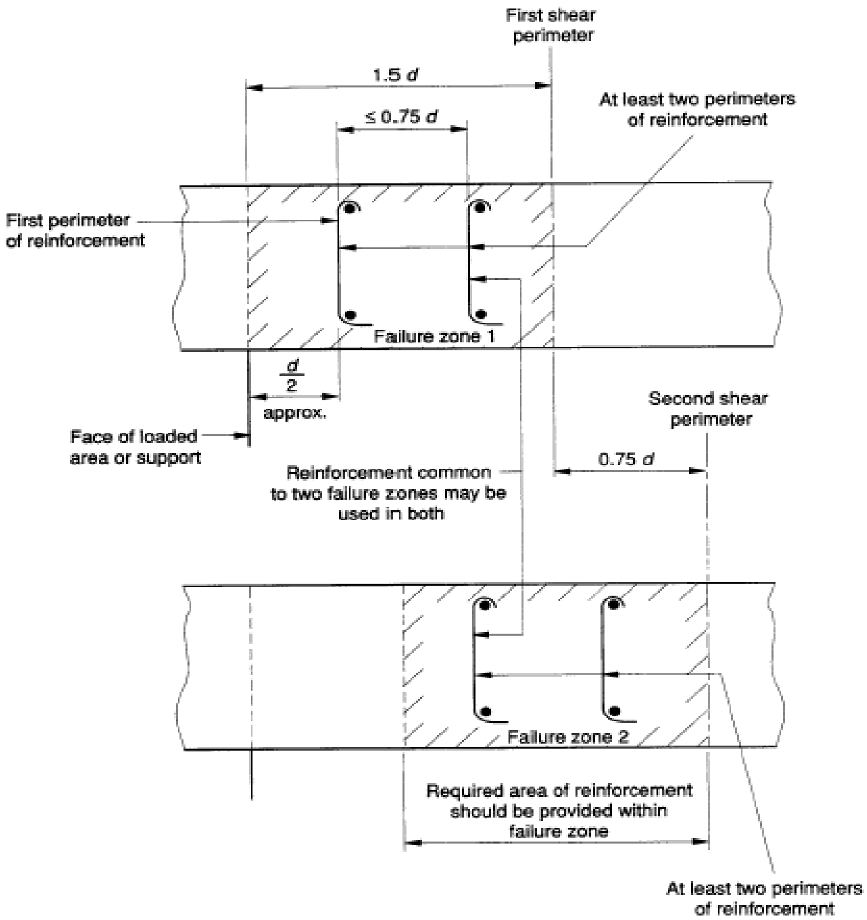
When shear reinforcement is required around columns it should be placed in rectangular perimeters. At least two sets of shear reinforcement should intersect the notional failure plane within the zone considered. (See Structures Notes 1988NST_5 and 1990NST_12).



The shear capacity is checked first on a perimeter 1.5d from the face of the loaded area. If the calculated shear stress does not exceed v_c then no further checks are needed.

If shear reinforcement is required, then it should be provided on at least two perimeters within the zone indicated in Figure 3.17. The first perimeter of reinforcement should be located at approximately 0.5d from the face of the loaded area and should contain not less than 40 % of the calculated area of reinforcement.

The spacing of perimeters of reinforcement should not exceed 0.75d and the spacing of the shear reinforcement around any perimeter should not exceed 1.5d. The shear reinforcement should be anchored round at least one layer of tension reinforcement. The shear stress should then be checked on successive perimeters at 0.75d intervals until a perimeter is reached which does not require shear reinforcement.



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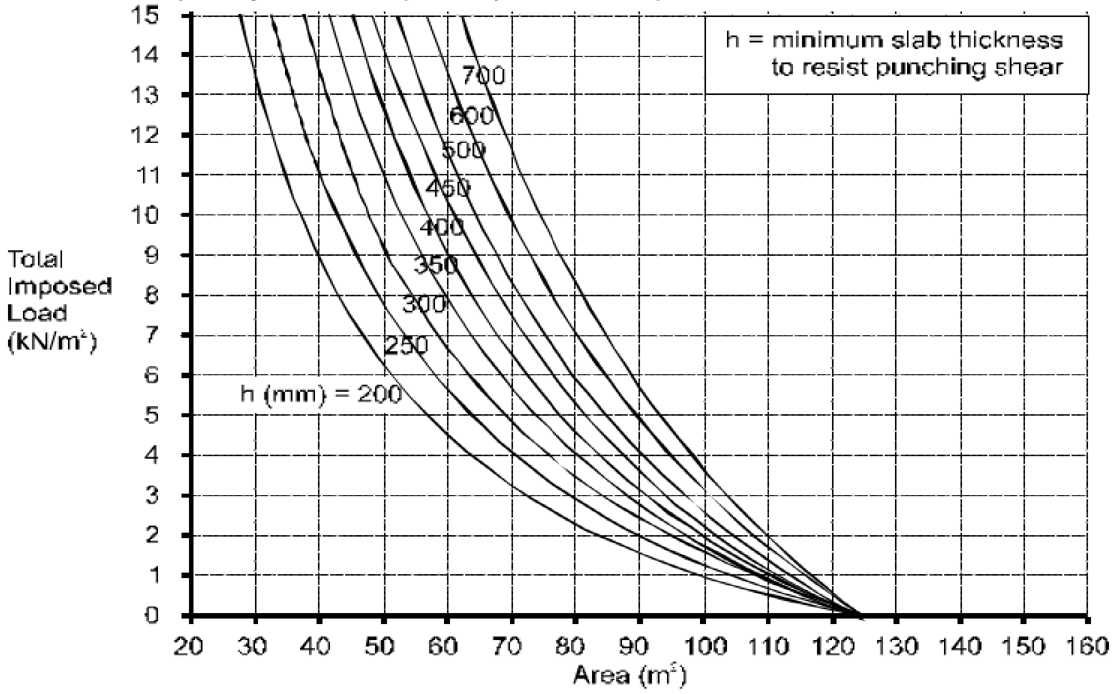
BS8110

Ultimate shear check at column face

Column (inc. head) 300 x 300

Note: For column sizes other than 300 x 300 the slab depth should be multiplied by the factor = (column perimeter/1200)



Notes:
1. f_{cu} = 35 N/mm²,
2. Dead load factor = 1.4,
3. Live load factor = 1.6,
4. The value of d/h is assumed to be 0.85,
5. The ratio of V_{eff}/V is assumed to be 1.15,

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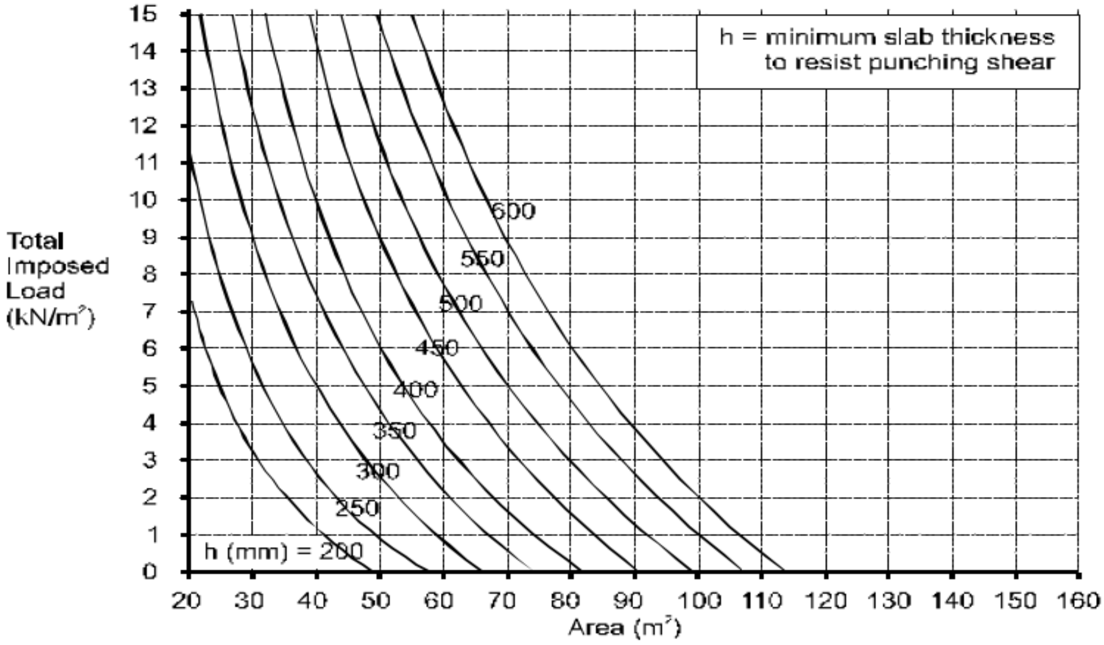
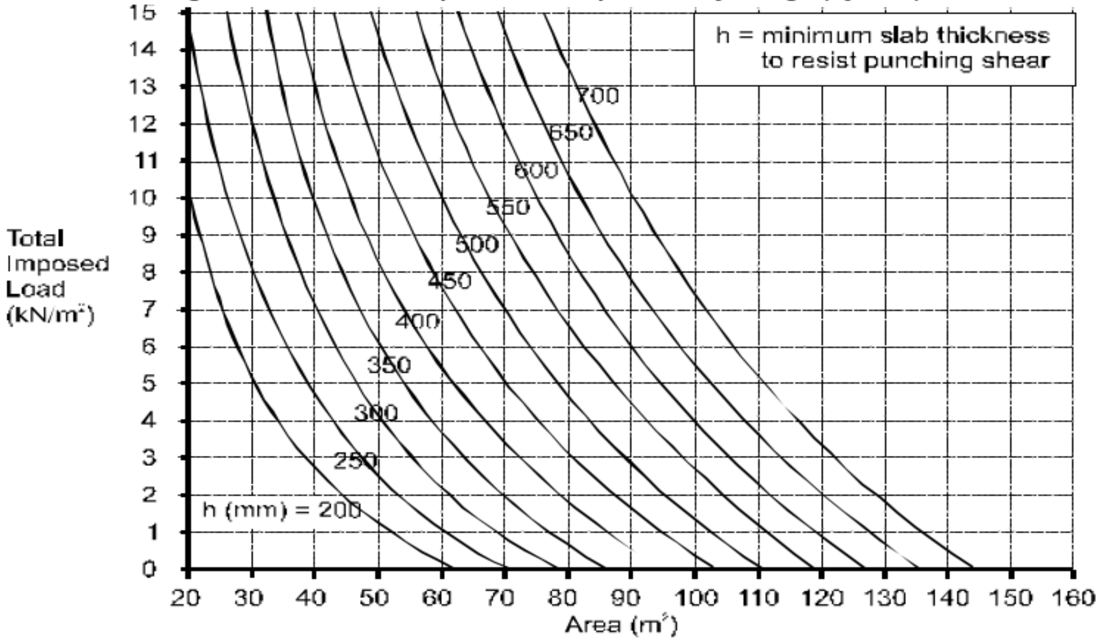
BS8110

Column 300 x 300
Punching shear check at first perimeter for preliminary design (v_c = 0.6)



Column 500 x 500
Punching shear check at first perimeter for preliminary design (v_c = 0.6)



Notes:
1. f_{cu} = 35 N/mm²,
2. Dead load factor = 1.4,
3. Live load factor = 1.6,
4. The value of d/h is assumed to be 0.85,
5. The ratio of V_{eff}/V is assumed to be 1.15,

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EC2

Punching Shear (EC2) Do Not Adopt Code Equivalence

ULS design punching shear into column, V_t	2340	kN	
Note V_t = F (internal), F/2+SDL $_{elev,x/y}$.$l_{x/y}$ (edge), F/4+(SDL $_{elev,x}$.l_x +SDL $_{elev,y}$.l_y)/2 (corner).			
Area of column section, $A_{c1} = l_{h,b}.l_{h,h}$ (rectangular) or $\pi l_{h,D}^2/4$ (circular)	0.64	m^2	
Average effective depth of both rebar layers, $d = (d_{x,h,c} + d_{y,h,c})/2$ · Include	345	mm	
Note conservatively that d_{dp} should not be incorporated above when incorporated within $d_{x,h,c}$ and $d_{y,h,c}$ to cater for the reduced effective depth at shear perimeters beyond the slab drop widths.			
Area of tensile steel reinforcement provided, $A_{s,prov,x,h,c}$	2094	mm^2/m	
Area of tensile steel reinforcement provided, $A_{s,prov,y,h,c}$	2094	mm^2/m	
Average area of tensile steel reinforcement provided, $A_{s,prov,h,c}$	2094	mm^2/m	
$\rho_w = 100A_{s,prov,h,c}/(1000.d)$	0.61	%	
$v_{Rd,c} = C_{Rd,c}.k(\rho_w f_{ck})^{1/3}+k_1.\sigma_{cp} \geq v_{min}+k_1.\sigma_{cp}$; $v_{min}=0.035k^{3/2}f_{ck}^{0.5}$, k_1=0.15, $f_{ck}\leq$5	0.54	N/mm^2	cl.6.2.2(1)
$C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$	0.12		cl.6.2.2(1)
$k = 1 + \sqrt{(200/d)} \leq 2.0$	1.76		cl.6.2.2(1)
$\rho_w = 100A_{s,prov,h,c}/(1000.d) \leq 2$	0.61	%	cl.6.2.2(1)
$\sigma_{cp} = F/(1000.h) \leq 0.2f_{cd}=0.2\alpha_{cc}.f_{ck}/\gamma_c=0.2(1.0.f_{ck}/1.5)$, $f_{ck}\leq$50N/m	0.00	N/mm^2	cl.6.2.2(1)

Table 8.2 Shear resistance of slabs without shear reinforcement $v_{Rd,c}$ N/mm^2 (Class C30/37 concrete)

$\rho_1 = A_s/bd$	Effective depth, d (mm)								
	≤ 200	225	250	300	350	400	500	600	750
0.25%	0.54	0.52	0.50	0.47	0.45	0.43	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.54	0.52	0.51	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.62	0.59	0.58	0.55	0.53	0.51
1.00%	0.75	0.72	0.71	0.68	0.65	0.64	0.61	0.59	0.57
1.25%	0.80	0.78	0.76	0.73	0.71	0.69	0.66	0.63	0.61
1.50%	0.85	0.83	0.81	0.78	0.75	0.73	0.70	0.67	0.65
2.00%	0.94	0.91	0.89	0.85	0.82	0.80	0.77	0.74	0.71
k	2.000	1.943	1.894	1.816	1.756	1.707	1.632	1.577	1.516

Table 8.3 Concrete strength modification factor

f_{ck} (N/mm^2)	25	30	35	40	45	50
Modification factor	0.94	1.00	1.05	1.10	1.14	1.19

Column Face Perimeter

Shear force at column face, $V_1 = V_t - n.A_{c1}$			2325	kN	
Eff. shear force, $V_{eff,1}$ = (1.15 int., 1.40 edge, 1.50 corner column) . V_1			2674	kN	cl.6.4.3(6)
Column face perimeter, u_1			3200	mm	cl.6.4.2(1)
	Rectangular		Circular		
Internal column:	*2.($l_{h,b}$+$l_{h,h}$)*	3200	*π.$l_{h,D}$*	N/A	mm
Edge column:	*2$l_{h,b}$+$l_{h,h}$ or 2$l_{h,h}$+$l_{h,b}$*	2400	*/4(π.$l_{h,D}$)*	N/A	mm
Corner column:	*($l_{h,b}$+$l_{h,h}$)*	1600	*π.$l_{h,D}$/2*	N/A	mm
Shear stress at column face perimeter, $v_1 = V_{eff,1} / u_1d$ (< $v_{Rd,max}$=0.20(1-f_{ck}/2			2.42	N/mm^2	cl.6.2.2(6)
Ultimate shear strength, $v_{Rd,max}$=0.20(1-f_{ck}/250).f_{ck}, $f_{ck}\leq$50N/mm^2)			4.97	N/mm^2	cl.6.2.2(6)
Ultimate shear stress utilisation			49%		OK

										EC2
First Shear Perimeter				*@2.00d*	***690***	*to*	*@0.00d*	***173***	mm	*cl.6.4.2(1)*
Shear force 2.0d from column face, $V_2 = V_t - n.(A_{c2} \le \{l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l_y$								2238	kN	**Note**
					Rectangular		Circular			
Internal column:		$2.(l_{h,b}.2d+l_{h,h}.2d)+A_{c1}+\pi.4d^2$			4.34		$(l_{h,D}+4d)^2$	N/A	m^2	
Edge column:	$(2.l_{h,h}.2d+l_{h,b}.2d)$ or $(2.l_{h,h}.2d+l_{h,b}.2d)]+A_{c1}+\pi.2d^2$				3.04		$(l_{h,D}+4d)^2$	N/A	m^2	
Corner column:		$(l_{h,b}.2d+l_{h,h}.2d)+A_{c1}+\pi.d^2$			2.12		$(l_{h,D}+4d)^2$	N/A	m^2	
Eff. shear force, $V_{eff,2}$ = (1.15 int., 1.40 edge, 1.50 corner column) . V_2								**2574**	kN	*cl.6.4.3(6)*
Column first perimeter, $u_2 \le \{2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2\}$								7535	mm	*cl.6.4.2(1)*
					Rectangular		Circular			
Internal column:		$2.(l_{h,b}+l_{h,h})+\pi.4d$			7535		$.(l_{h,D}+4d)$	N/A	mm	
Edge column:		$2l_{h,b}+l_{h,h}+\pi.2d$ or $2l_{h,h}+l_{h,b}+\pi.2d$			4568		$.(l_{h,D}+4d)$	N/A	mm	
Corner column:		$(l_{h,b}+l_{h,h})+\pi.d$			2684		$.(l_{h,D}+4d)$	N/A	mm	
Shear stress at column first perimeter, $v_2 = V_{eff,2} / u_2d$								**0.99**	N/mm^2	*cl.6.4.3(3)*
(Shear capacity enhancement by calculating v_d at 2.0d from support and comparing against										
unenhanced $v_{Rd,c}$ as clause 6.4.2(1) EC2 employed instead of calculating v_d at support and comparing										
against enhanced v_c within 2.0d of the support as clause 6.4.3(7) and clause 6.2.2(6) EC2 invalid;)										
	Case $v_2 < v_{Rd,c}$						**INVALID**	**0.54**		*cl.6.4.3(2)*
	No links required.									
	Case $v_{Rd,c} < v_2 < 0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5}$					**0.54**	**N/A**	**0.78**		*cl.6.4.3(2)*
		$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \le 50N/mm^2$					0.37		N/mm^2	*cl.9.2.2(5)*
		$A_{sv,prov,2} \ge A_{sv,nom,2}$			N/A	>=	N/A		mm^2	
		where $A_{sv,nom,2} > 3.(0.08/\gamma_s.f_{ck}^{0.5}).u_2s_{v,2}/(1.5f_{ywd,ef})$ *assuming 3 perimeters*								*cl.6.4.5(1)*
		where $f_{ywd,ef} = 250+0.25d \le f_{yv}/\gamma_s$, $f_{yv} \le 600N/mm^2$					336		N/mm^2	*cl.6.4.5(1)*
	Case $0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5} < v_2 < 2.0v_{Rd,c}$					**0.78**	**VALID**	**1.09**		*cl.6.4.3(2)*
		$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \le 50N/mm^2$					0.37		N/mm^2	*cl.9.2.2(5)*
		$A_{sv,prov,2} \ge A_{sv,2}$			5969	>=	5875		mm^2	
		where $A_{sv,2} > 3.(v_2-0.75v_{Rd,c}).u_2s_{v,2}/(1.5f_{ywd,ef})$ *assuming 3 perimeters*								*cl.6.4.5(1)*
		where $f_{ywd,ef} = 250+0.25d \le f_{yv}/\gamma_s$, $f_{yv} \le 600N/mm^2$					336		N/mm^2	*cl.6.4.5(1)*
	Case $v_2 > 2.0v_{Rd,c}$					**1.09**	**N/A**			*cl.8.1.1 MOSL*
First shear perimeter shear utilisation								**98%**		**OK**

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Second Shear Perimeter	@3.50d	1208	to	@2.00d	690 mm	cl.6.4.5(4)

Shear force 3.5d from column face, $V_3 = V_t - n.(A_{c3} \le \{l_x.l_y, l_x/2.l_y, l_x.l_y/2, l_x/2.l_y$ — 2127 kN — **Note**

	Rectangular		Circular			EC2
Internal column:	$2.(l_{h,b}.3.5d+l_{h,h}.3.5d)+A_{c1}+\pi.(7d)^2/4$	9.08	$(l_{h,D}+7d)^2$	N/A	m^2	
Edge column:	$(2.l_{h,h}.3.5d+l_{h,b}.3.5d)]+A_{c1}+\pi.(7d)^2/8$	5.83	$(l_{h,D}+7d)^2$	N/A	m^2	
Corner column:	$(l_{h,b}.3.5d+l_{h,h}.3.5d)+A_{c1}+\pi.(7d)^2/16$	3.72	$(l_{h,D}+7d)^2$	N/A	m^2	
Eff. shear force, $V_{eff,3}$ = (1.15 int., 1.40 edge, 1.50 corner column) . V_3				2447	kN	cl.6.4.3(6)
Column second perimeter, $u_3 \le \{2l_x+2l_y, l_x+l_y, l_x+l_y, l_x/2+l_y/2\}$				10787	mm	cl.6.4.5(4)
	Rectangular		Circular			
Internal column:	$2.(l_{h,b}+l_{h,h})+\pi.7d$	10787	$.(l_{h,D}+7d)$	N/A	mm	
Edge column:	$2l_{h,b}+l_{h,h}+\pi.7d/2$ or $2l_{h,h}+l_{h,b}+\pi.7d/2$	6193	$.(l_{h,D}+7d)$	N/A	mm	
Corner column:	$(l_{h,b}+l_{h,h})+\pi.7d/4$	3497	$.(l_{h,D}+7d)$	N/A	mm	
Shear stress at column second perimeter, $v_3 = V_{eff,3} / u_3d$				0.66	N/mm^2	cl.6.4.3(3)

Case $v_3 < v_{Rd,c}$			INVALID	0.54	cl.6.4.3(2)
No links required.					
Case $v_{Rd,c} < v_3 < 0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5}$		0.54	VALID	0.78	cl.6.4.3(2)
$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \le 50N/mm^2$			0.37	N/mm^2	cl.9.2.2(5)
$A_{sv,prov,3} \ge A_{sv,nom,3}$	9425	>=	3543	mm^2	
where $A_{sv,nom,3} > 2.(0.08/\gamma_s.f_{ck}^{0.5}).u_3s_{v,3}/(1.5f_{ywd,ef})$ *assuming 2 perimeters*					cl.6.4.5(1)
where $f_{ywd,ef} = 250+0.25d \le f_{yv}/\gamma_s$, $f_{yv} \le 600N/mm^2$			336	N/mm^2	cl.6.4.5(1)
Case $0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5} < v_3 < 2.0v_{Rd,c}$		0.78	N/A	1.09	cl.6.4.3(2)
$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \le 50N/mm^2$			0.37	N/mm^2	cl.9.2.2(5)
$A_{sv,prov,3} \ge A_{sv,3}$	N/A	>=	N/A	mm^2	
where $A_{sv,3} > 2.(v_3-0.75v_{Rd,c}).u_3s_{v,3}/(1.5f_{ywd,ef})$ *assuming 2 perimeters*					cl.6.4.5(1)
where $f_{ywd,ef} = 250+0.25d \le f_{yv}/\gamma_s$, $f_{yv} \le 600N/mm^2$			336	N/mm^2	cl.6.4.5(1)
EY **Case $v_3 > 2.0v_{Rd,c}$**		1.09	N/A		cl.8.1.1 MOSL

Second shear perimeter shear utilisation	38%		OK

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.: jXXX	Sheet No.: 26	Rev.
		Member/Location	
Job Title	Member Design - Reinforced Concrete Flat Slab BS8110	Drg. Ref.	
Member Design - RC Flat Slab		Made by XX	Date 9/2/2024	Chd.

										EC2
Third Shear Perimeter			*@5.00d*	***1725***	*to*	*@3.50d*	***1208***	mm		*cl.6.4.5(4)*
Shear force 5.0d from column face, $V_4 = V_t - n.(A_{c4} \leq \{l_x.l_y, l_x/2.l_y, l_x.l_y/2, l_x/2.l_y$								1977	kN	**Note**
				Rectangular		Circular				
Internal column: $2.(l_{h,b}.5d+l_{h,h}.5d)+A_{c1}+\pi.(10d)^2/4$					15.51	$_{h,D}+10d)^2$	N/A	m^2		
Edge column: or $(2.l_{h,h}.5d+l_{h,b}.5d)]+A_{c1}+\pi.(10d)^2/8$					9.45	$_{h,D}+10d)^2$	N/A	m^2		
Corner column: $(l_{h,b}.5d+l_{h,h}.5d)+A_{c1}+\pi.(10d)^2/16$					5.74	$_{h,D}+10d)^2$	N/A	m^2		
Eff. shear force, $V_{eff,4}$ = (1.15 int., 1.40 edge, 1.50 corner column) . V_4							**2274**	kN		*cl.6.4.3(6)*
Column third perimeter, $u_4 \leq \{2l_x+2l_y, l_x+l_y, l_x+l_y, l_x/2+l_y/2\}$							14038	mm		*cl.6.4.5(4)*
				Rectangular		Circular				
Internal column:			*$2.(l_{h,b}+l_{h,h})+\pi.10d$*		14038	*$(l_{h,D}+10d)$*	N/A	mm		
Edge column: $2l_{h,b}+l_{h,h}+\pi.10d/2$ or $2l_{h,h}+l_{h,b}+\pi.10d/2$					7819	*$(l_{h,D}+10d)$*	N/A	mm		
Corner column:			*$(l_{h,b}+l_{h,h})+\pi.10d/4$*		4310	*$(l_{h,D}+10d)$*	N/A	mm		
Shear stress at column third perimeter, $v_4 = V_{eff,4} / u_4d$							**0.47**	N/mm^2		*cl.6.4.3(3)*
	Case $v_4 < v_{Rd,c}$						**VALID**	**0.54**		*cl.6.4.3(2)*
	No links required.									
	Case $v_{Rd,c} < v_4 < 0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5}$					**0.54**	**N/A**	**0.78**		*cl.6.4.3(2)*
		$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \leq 50N/mm^2$					0.37	N/mm^2		*cl.9.2.2(5)*
		$A_{sv,prov,4} \geq A_{sv,nom,4}$			N/A	>=	N/A	mm^2		
		where $A_{sv,nom,4} > 2.(0.08/\gamma_s.f_{ck}^{0.5}).u_4s_{v,4}/(1.5f_{ywd,ef})$ *assuming 2 perimeters*								*cl.6.4.5(1)*
		where $f_{ywd,ef} = 250+0.25d \leq f_{yv}/\gamma_s$, $f_{yv} \leq 600N/mm^2$					336	N/mm^2		*cl.6.4.5(1)*
	Case $0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5} < v_4 < 2.0v_{Rd,c}$					**0.78**	**N/A**	**1.09**		*cl.6.4.3(2)*
		$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \leq 50N/mm^2$					0.37	N/mm^2		*cl.9.2.2(5)*
		$A_{sv,prov,4} \geq A_{sv,4}$			N/A	>=	N/A	mm^2		
		where $A_{sv,4} > 2.(v_4-0.75v_{Rd,c}).u_4s_{v,4}/(1.5f_{ywd,ef})$ *assuming 2 perimeters*								*cl.6.4.5(1)*
		where $f_{ywd,ef} = 250+0.25d \leq f_{yv}/\gamma_s$, $f_{yv} \leq 600N/mm^2$					336	N/mm^2		*cl.6.4.5(1)*
EY	**Case $v_4 > 2.0v_{Rd,c}$**					**1.09**	**N/A**			*cl.8.1.1 MOSL*
Third shear perimeter shear utilisation							**86%**			**OK**

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	27	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Flat Slab BS8110	Drg. Ref.		
Member Design - RC Flat Slab		Made by XX	Date 9/2/2024	Chd.

						EC2
Fourth Shear Perimeter	*@6.50d*	***2243***	*to*	*@5.00d*	***1725*** mm	*cl.6.4.5(4)*

Shear force 5.0d from column face, $V_5 = V_t - n.(A_{c5} \leq \{l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l_y$			1787	kN	**Note**
	Rectangular	*Circular*			
Internal column:	$(l_{h,b}.6.5d+l_{h,h}.6.5d)+A_{c1}+\pi.(13d)^2/4$	23.61	$_{h,D}+13d)^2$ N/A	m²	
Edge column:	$(2.l_{h,h}.6.5d+l_{h,b}.6.5d)]+A_{c1}+\pi.(13d)^2/8$	13.92	$_{h,D}+13d)^2$ N/A	m²	
Corner column:	$(l_{h,b}.6.5d+l_{h,h}.6.5d)+A_{c1}+\pi.(13d)^2/16$	8.18	$_{h,D}+13d)^2$ N/A	m²	
Eff. shear force, $V_{eff,5}$ = (1.15 int., 1.40 edge, 1.50 corner column) . V_5			**2056**	kN	*cl.6.4.3(6)*
Column fourth perimeter, $u_5 \leq \{2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2\}$			17290	mm	*cl.6.4.5(4)*
	Rectangular	*Circular*			
Internal column:	$2.(l_{h,b}+l_{h,h})+\pi.13d$	17290	$(l_{h,D}+13d)$ N/A	mm	
Edge column:	$2l_{h,b}+l_{h,h}+\pi.13d/2$ or $2l_{h,h}+l_{h,b}+\pi.13d/2$	9445	$(l_{h,D}+13d)$ N/A	mm	
Corner column:	$(l_{h,b}+l_{h,h})+\pi.13d/4$	5123	$(l_{h,D}+13d)$ N/A	mm	
Shear stress at column fourth perimeter, $v_5 = V_{eff,5} / u_5d$			**0.34**	N/mm²	*cl.6.4.3(3)*

	Case $v_5 < v_{Rd,c}$		**VALID**	**0.54**	*cl.6.4.3(2)*
	No links required.				
	Case $v_{Rd,c} < v_5 < 0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5}$	**0.54**	**N/A**	**0.78**	*cl.6.4.3(2)*
	$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \leq 50N/mm^2$		0.37	N/mm²	*cl.9.2.2(5)*
	$A_{sv,prov,5} \geq A_{sv,nom,5}$ N/A	>=	N/A	mm²	
	where $A_{sv,nom,5} > 2.(0.08/\gamma_s.f_{ck}^{0.5}).u_5s_{v,5}/(1.5f_{ywd,ef})$ *assuming 2 perimeters*				*cl.6.4.5(1)*
	where $f_{ywd,ef} = 250+0.25d \leq f_{yv}/\gamma_s$, $f_{yv} \leq 600N/mm^2$		336	N/mm²	*cl.6.4.5(1)*
	Case $0.75v_{Rd,c}+0.08/\gamma_s.f_{ck}^{0.5} < v_5 < 2.0v_{Rd,c}$	**0.78**	**N/A**	**1.09**	*cl.6.4.3(2)*
	$0.08/\gamma_s.f_{ck}^{0.5}$, $f_{ck} \leq 50N/mm^2$		0.37	N/mm²	*cl.9.2.2(5)*
	$A_{sv,prov,5} \geq A_{sv,5}$ N/A	>=	N/A	mm²	
	where $A_{sv,5} > 2.(v_5-0.75v_{Rd,c}).u_5s_{v,5}/(1.5f_{ywd,ef})$ *assuming 2 perimeters*				*cl.6.4.5(1)*
	where $f_{ywd,ef} = 250+0.25d \leq f_{yv}/\gamma_s$, $f_{yv} \leq 600N/mm^2$		336	N/mm²	*cl.6.4.5(1)*
EY	**Case $v_5 > 2.0v_{Rd,c}$**	**1.09**	**N/A**		*cl.8.1.1 MOSL*

Fourth shear perimeter shear utilisation	**63%**		**OK**

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 28	Rev.
		Member/Location		
Job Title	Member Design - Reinforced Concrete Flat Slab BS8110	Drg. Ref.		
Member Design - RC Flat Slab		Made by XX	Date 9/2/2024	Chd.

ACI318

Punching Shear (ACI318) — Do Not Adopt Code Equivalence

ULS design punching shear into column, V_t					2340	kN	
Note V_t = F (internal), F/2+SDL$_{elev,x/y}$.l$_{x/y}$ (edge), F/4+(SDL$_{elev,x}$.l$_x$ +SDL$_{elev,y}$.l$_y$)/2 (corner).							
Area of column section, A_{c1} = $l_{h,b}.l_{h,h}$ (rectangular) or $\pi l_{h,D}^2/4$ (circular)					0.64	m^2	
Average effective depth of both rebar layers, d = $(d_{x,h,c} + d_{y,h,c})/2$				Include	345	mm	
Note conservatively that d_{dp} should not be incorporated above when incorporated within $d_{x,h,c}$ and $d_{y,h,c}$ to cater for the reduced effective depth at shear perimeters beyond the slab drop widths.							
$\phi v_{c,2}$ | $\phi v_{c,3}$ | $\phi v_{c,4}$ | $\phi v_{c,5}$ = [MIN(a-c)]	1.31	1.31	1.16		1.04	N/mm^2	

Table 22.6.5.2—Calculation of v_c for two-way shear

	v_c	
Least of (a), (b), and (c):	$0.33\lambda\sqrt{f_c'}$	(a)
	$0.17\left(1+\frac{2}{\beta}\right)\lambda\sqrt{f_c'}$	(b)
	$0.083\left(2+\frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'}$	(c)

Note: β is the ratio of long side to short side of the column, concentrated load, or reaction area and α_s is given in 22.6.5.3.

Note $f_c' \le 70N/mm^2$			
[(a)] ϕv_c =	1.31	N/mm^2	*5.5.2, cl.22*
[(b)] ϕv_c =	2.02	N/mm^2	*5.5.2, cl.22*
[(c)] $\phi v_{c,2}$ =	1.65	N/mm^2	*5.5.2, cl.22*
[(c)] $\phi v_{c,3}$ =	1.40	N/mm^2	*5.5.2, cl.22*
[(c)] $\phi v_{c,4}$ =	1.16	N/mm^2	*5.5.2, cl.22*
[(c)] $\phi v_{c,5}$ =	1.04	N/mm^2	*5.5.2, cl.22*

where λ = {1.00 NWC, 0.75 LWC}			Normal Weight	1.00		
where β = MAX (h,b) / MIN (h,b)				1.0		
where α_s = {40 interior, 30 edge, 20 corner} column				40		*cl.22.6.5.3*
where b_0 = {u_2 | u_3 | u_4 | u_5}	4580	6126	9055	11984	mm	
where φ = 0.75				0.75		*cl.21.2*

EY

Column Face Perimeter

Shear force at column face, $V_1 = V_t - n.A_{c1}$				2325	kN
Eff. shear force, $V_{eff,1}$ = (1.15 int., 1.40 edge column) . V_1				2674	kN
Column face perimeter, u_1				3200	mm
		Rectangular		*Circular*	
Internal column:	*$2.(l_{h,b}+l_{h,h})$*	*3200*	*$\pi.l_{h,D}$*	*N/A*	mm
Edge column:	*$2l_{h,b}+l_{h,h}$ or $2l_{h,h}+l_{h,b}$*	*2400*	*/4($\pi.l_{h,D}$)*	*N/A*	mm
Corner column:	*$(l_{h,b}+l_{h,h})$*	*1600*	*$\pi.l_{h,D}/2$*	*N/A*	mm

Shear stress at column face perimeter, $v_1 = V_{eff,1} / u_1 d$ (< $\phi 0.66\sqrt{f_c'}$), $f_c' \le 70N/$	2.42	N/mm^2	*5.6.2, cl.22*
Ultimate shear strength, $\phi 0.66\sqrt{f_c'}$, $f_c' \le 70N/mm^2$	2.62	N/mm^2	*5.6.2, cl.22*
Ultimate shear stress utilisation	92%		OK

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 29	Rev.
		Member/Location		
Job Title	Member Design - Reinforced Concrete Flat Slab BS8110	Drg. Ref.		
Member Design - RC Flat Slab		Made by **XX**	Date **9/2/2024**	Chd.

								<u>*ACI318*</u>
First Shear Perimeter		*@0.50d*	***173***	*to*	*@2.50d*	***863***	mm	*cl.22.6.4.1*
Shear force 0.5d from column face, $V_2 = V_t$ - n.($A_{c2} \le \{l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l_y$						2309	kN	**Note**
			Rectangular		*Circular*			
Internal column:			$(l_{h,b}+d).(l_{h,h}+d)$	*1.31*	$(l_{h,D}+d)^2$	*N/A*	m^2	
Edge column:			$(l_{h,b}+0.5d).(l_{h,h}+d)$ or $(l_{h,h}+0.5d).(l_{h,b}+d)$	*1.11*	$l).(l_{h,D}+d)$	*N/A*	m^2	
Corner column:			$(l_{h,b}+0.5d).(l_{h,h}+0.5d)$	*0.95*	$_{,D}+0.5d)^2$	*N/A*	m^2	
Eff. shear force, $V_{eff,2}$ = (1.15 int., 1.40 edge column) . V_2						**2656**	kN	*BS8110*
Column first perimeter, $u_2 \le \{2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2\}$						4580	mm	*cl.22.6.4.1*
			Rectangular		*Circular*			
Internal column:			$2.(l_{h,b}+l_{h,h})+4d$	*4580*	$4l_{h,D}+4d$	*N/A*	mm	
Edge column:			$2l_{h,b}+l_{h,h}+2d$ or $2l_{h,h}+l_{h,b}+2d$	*3090*	$3l_{h,D}+2d$	*N/A*	mm	
Corner column:			$(l_{h,b}+l_{h,h})+d$	*1945*	$2l_{h,D}+d$	*N/A*	mm	
Shear stress at column first perimeter, $v_2 = V_{eff,2} / u_2d$						**1.68**	N/mm^2	
.6.3.1								
.6.3.1								
.6.3.1								
	Case $v_2 < \phi v_{c,2}$					**INVALID**	**1.31**	*cl.22.6.5.2*
	No links required.							
	Case $\phi v_{c,2} < v_2 < \phi 0.17\lambda\sqrt{f_c'}+\phi NL$				**1.31**	**N/A**	**0.94**	*cl.22.6.6.1*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'},0.35\}, f_c' \le 70N/mm^2$				0.26	N/mm^2	
		$A_{sv,prov,2} \ge A_{sv,nom,2}$		N/A	>=	N/A	mm^2	
		where $A_{sv,nom,2} > 4.MAX\{0.062\sqrt{f_c'},0.35\}.u_2s_{v,2}/f_{yv}$						
		where λ={1.00 NWC, 0.75 LWC}, $f_{yv} \le 420N/mm^2$, $f_c' \le 70N/mm^2$						*cl.22.6.3.2*
	Case $\phi 0.17\lambda\sqrt{f_c'}+\phi NL < v_2 < \phi 0.5\sqrt{f_c'}$				**0.94**	**VALID**	**1.98**	*cl.22.6.6.2*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'},0.35\}, f_c' \le 70N/mm^2$				0.26	N/mm^2	
		$A_{sv,prov,2} \ge A_{sv,2}$		4477	<	13165	mm^2	*cl.22.6.1.3*
		where $A_{sv,2} > 4.(v_2-\phi 0.17\lambda\sqrt{f_c'}).u_2s_{v,2}/(\phi f_{yv})$ assuming 4 peripherals						*cl.22.6.6.1, cl.22*
		where ϕ=0.75, λ={1.00 NWC, 0.75 LWC}, $f_{yv} \le 420N/mm^2$						*cl.21.2, cl.22.6*
	Case $v_2 > \phi 0.5\sqrt{f_c'}$				**1.98**	**N/A**		*cl.22.6.6.2, cl.22*
First shear perimeter shear utilisation						**294%**		**NOT OK**
.6.3.1								
.6.3.1								

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 30	Rev.
		Member/Location		
Job Title	Member Design - Reinforced Concrete Flat Slab BS8110	Drg. Ref.		
Member Design - RC Flat Slab		Made by XX	Date 9/2/2024	Chd.

									ACI318
Second Shear Perimeter			@1.50d	518	to	@3.50d	1208	mm	cl.22.6.4.2
Shear force 1.5d from column face, $V_3 = V_t$ - n.(A_{c3} ≤ {$l_x.l_y, l_x/2.l_y, l_x.l_y/2, l_x/2.l_y$							2285	kN	**Note**
				Rectangular		Circular			
Internal column:				*($l_{h,b}$+2.12d).($l_{h,h}$+2.12d)*	*2.35*	*+2.12d)^2*	*N/A*	m^2	
Edge column:			*($l_{h,h}$+2.12d) or ($l_{h,h}$+1.06d).($l_{h,b}$+2.12d)*		*1.79*	*,D +2.12d)*	*N/A*	m^2	
Corner column:				*($l_{h,b}$+1.06d).($l_{h,h}$+1.06d)*	*1.36*	*+1.06d)^2*	*N/A*	m^2	
Eff. shear force, $V_{eff,3}$ = (1.15 int., 1.40 edge column) . V_3							2628	kN	*BS8110*
Column second perimeter, u_3 ≤ {$2l_x+2l_y, l_x+l_y, l_x+l_y, l_x/2+l_y/2$}							6126	mm	*cl.22.6.4.2*
				Rectangular		Circular			
Internal column:				*2.($l_{h,b}$+$l_{h,h}$)+8.48d*	*6126*	*h,D +8.48d*	*N/A*	mm	
Edge column:		*2$l_{h,b}$+$l_{h,h}$+4.24d or 2$l_{h,h}$+$l_{h,b}$+4.24d*			*3863*	*h,D +4.24d*	*N/A*	mm	
Corner column:				*($l_{h,b}$+$l_{h,h}$)+2.12d*	*2331*	*h,D +2.12d*	*N/A*	mm	
Shear stress at column second perimeter, $v_3 = V_{eff,3} / u_3d$							1.24	N/mm^2	

	Case $v_3 < \phi v_{c,3}$						VALID	1.31	*cl.22.6.5.2*
	No links required.								
	Case $\phi v_{c,3} < v_3 < 0.17\lambda\sqrt{f_c'}$+NL					1.31	N/A	0.94	*cl.22.6.6.1*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'},0.35\}, f_c' \le 70N/mm^2$					0.26	N/mm^2	
		$A_{sv,prov,3} \ge A_{sv,nom,3}$			N/A	>=	N/A	mm^2	
		where $A_{sv,nom,3} > 4.MAX\{0.062\sqrt{f_c'},0.35\}.u_3s_{v,3}/f_{yv}$							
		where λ={1.00 NWC, 0.75 LWC}, $f_{yv} \le 420N/mm^2$, $f_c' \le 70N/mm^2$							*cl.22.6.3.2*
cl.22.6.3.	**Case $\phi 0.17\lambda\sqrt{f_c'}+\phi NL < v_3 < \phi 0.5\sqrt{f_c'}$**					0.94	VALID	1.98	*cl.22.6.6.2*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'},0.35\}, f_c' \le 70N/mm^2$					0.26	N/mm^2	
		$A_{sv,prov,3} \ge A_{sv,3}$			4477	<	9955	mm^2	*cl.22.6.1.3*
.6.7.2		where $A_{sv,3} > 4.(v_3-\phi 0.17\lambda\sqrt{f_c'}).u_3s_{v,3}/(\phi f_{yv})$ assuming 4 peripherals							*cl.22.6.6.1, cl.22*
.3.2		where φ=0.75, λ={1.00 NWC, 0.75 LWC}, $f_{yv} \le 420N/mm^2$							*cl.21.2, cl.22.6*
.6.3.1	**Case $v_3 > \phi 0.5\sqrt{f_c'}$**					1.98	N/A		*cl.22.6.6.2, cl.22*
Second shear perimeter shear utilisation							95%		**OK**

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									ACI318
Third Shear Perimeter			*@3.00d*	***1035***	*to*	*@5.00d*	***1725***	mm	*cl.22.6.4.2*
Shear force 3.0d from column face, $V_4 = V_t - n.(A_{c4} \le \{l_x.l_y,l_x/2.l_y,l_x.l_y/2,l_x/2.l_y$								2220	kN	**Note**
				Rectangular		*Circular*			
Internal column:			$(l_{h,b}+4.24d).(l_{h,h}+4.24d)$		*5.12*	$+4.24d)^2$	*N/A*	m^2	
Edge column:			$(l_{h,h}+4.24d)$ or $(l_{h,b}+2.12d).(l_{h,b}+4.24d)$		*3.47*	$_{,D}+4.24d)$	*N/A*	m^2	
Corner column:			$(l_{h,b}+2.12d).(l_{h,h}+2.12d)$		*2.35*	$+2.12d)^2$	*N/A*	m^2	
Eff. shear force, $V_{eff,4}$ = (1.15 int., 1.40 edge column) . V_4							**2553**	kN	*BS8110*
Column third perimeter, $u_4 \le \{2l_x+2l_y,l_x+l_y,l_x+l_y,l_x/2+l_y/2\}$							9055	mm	*cl.22.6.4.2*
				Rectangular		*Circular*			
Internal column:			$2.(l_{h,b}+l_{h,h})+16.97d$		*9055*	$_{D}+16.97d$	*N/A*	mm	
Edge column:			$2l_{h,b}+l_{h,h}+8.49d$ or $2l_{h,h}+l_{h,b}+8.49d$		*5329*	$_{h,D}+8.49d$	*N/A*	mm	
Corner column:			$(l_{h,b}+l_{h,h})+4.24d$		*3063*	$_{h,D}+4.24d$	*N/A*	mm	
Shear stress at column third perimeter, $v_4 = V_{eff,4} / u_4d$							**0.82**	N/mm^2	
	Case $v_4 < \phi v_{c,4}$					**VALID**		**1.16**	*cl.22.6.5.2*
	No links required.								
	Case $\phi v_{c,4} < v_4 < 0.17\lambda\sqrt{f_c'}$+NL				**1.16**	**N/A**		**0.94**	*cl.22.6.6.1*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'},0.35\}$, $f_c' \le 70N/mm^2$				0.26		N/mm^2	
		$A_{sv,prov,4} \ge A_{sv,nom,4}$		N/A	>=	N/A		mm^2	
		where $A_{sv,nom,4} > 4.MAX\{0.062\sqrt{f_c'},0.35\}.u_4s_{v,4}/f_{yv}$							
		where λ={1.00 NWC, 0.75 LWC}, $f_{yv} \le 420N/mm^2$, $f_c' \le 70N/mm^2$							*cl.22.6.3.2*
cl.22.6.3.	**Case $\phi 0.17\lambda\sqrt{f_c'}+\phi NL < v_4 < \phi 0.5\sqrt{f_c'}$**				**0.94**	**N/A**		**1.98**	*cl.22.6.6.2*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'},0.35\}$, $f_c' \le 70N/mm^2$				0.26		N/mm^2	
		$A_{sv,prov,4} \ge A_{sv,4}$		N/A	>=	N/A		mm^2	*cl.22.6.1.3*
6.7.2		where $A_{sv,4} > 4.(v_4-\phi 0.17\lambda\sqrt{f_c'}).u_4s_{v,4}/(\phi f_{yv})$ assuming 4 peripherals							*cl.22.6.6.1, cl.22*
3.2		where ϕ=0.75, λ={1.00 NWC, 0.75 LWC}, $f_{yv} \le 420N/mm^2$							*cl.21.2, cl.22.6*
6.3.1	**Case $v_4 > \phi 0.5\sqrt{f_c'}$**				**1.98**	**N/A**			*cl.22.6.6.2, cl.22*
Third shear perimeter shear utilisation							**70%**		**OK**

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									ACI318
Fourth Shear Perimeter			*@4.50d*	***1553***	*to*	*@6.50d*	***2243***	mm	*cl.22.6.4.2*
Shear force 4.5d from column face, $V_5 = V_t - n.(A_{c5} \leq \{l_x.l_y, l_x/2.l_y, l_x.l_y/2, l_x/2.l_y$							2130	kN	**Note**
				Rectangular		*Circular*			
Internal column:		$(l_{h,b}+6.36d).(l_{h,h}+6.36d)$			*8.97*	$_{D}+6.36d)^2$	*N/A*	m^2	
Edge column:		$(l_{h,h}+6.36d)$ or $(l_{h,h}+3.18d).(l_{h,b}+6.36d)$			*5.68*	$_{h,D}+6.36d)$	*N/A*	m^2	
Corner column:		$(l_{h,b}+3.18d).(l_{h,h}+3.18d)$			*3.60*	$_{D}+3.18d)^2$	*N/A*	m^2	
Eff. shear force, $V_{eff,5}$ = (1.15 int., 1.40 edge column) . V_5							**2450**	kN	*BS8110*
Column fourth perimeter, $u_5 \leq \{2l_x+2l_y, l_x+l_y, l_x+l_y, l_x/2+l_y/2\}$							11984	mm	*cl.22.6.4.2*
				Rectangular		*Circular*			
Internal column:			$2.(l_{h,b}+l_{h,h})+25.46d$		*11984*	$_{D}+25.46d$	*N/A*	mm	
Edge column:	$2l_{h,b}+l_{h,h}+12.73d$ or $2l_{h,h}+l_{h,b}+12.73d$				*6792*	$_{D}+12.73d$	*N/A*	mm	
Corner column:			$(l_{h,b}+l_{h,h})+6.36d$		*3794*	$_{h,D}+6.36d$	*N/A*	mm	
Shear stress at column fourth perimeter, $v_5 = V_{eff,5} / u_5 d$							**0.59**	N/mm^2	
	Case $v_5 < \phi v_{c,5}$					**VALID**		**1.04**	*cl.22.6.5.2*
	No links required.								
	Case $\phi v_{c,5} < v_5 < 0.17\lambda\sqrt{f_c'}+NL$				**1.04**		**N/A**	**0.94**	*cl.22.6.6.1*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'}, 0.35\}$, $f_c' \leq 70N/mm^2$					0.26	N/mm^2	
		$A_{sv,prov,5} \geq A_{sv,nom,5}$			N/A	>=	N/A	mm^2	
		where $A_{sv,nom,5} > 4.MAX\{0.062\sqrt{f_c'}, 0.35\}.u_5 s_{v,5}/f_{yv}$							
		where λ={1.00 NWC, 0.75 LWC}, $f_{yv} \leq 420N/mm^2$, $f_c' \leq 70N/mm^2$							*cl.22.6.3.2*
cl.22.6.3.	**Case $\phi 0.17\lambda\sqrt{f_c'}+\phi NL < v_5 < \phi 0.5\sqrt{f_c'}$**				**0.94**		**N/A**	**1.98**	*cl.22.6.6.2*
		$\phi NL = \phi MAX\{0.062\sqrt{f_c'}, 0.35\}$, $f_c' \leq 70N/mm^2$					0.26	N/mm^2	
		$A_{sv,prov,5} \geq A_{sv,5}$			N/A	>=	N/A	mm^2	*cl.22.6.1.3*
6.7.2		where $A_{sv,5} > 4.(v_5-\phi 0.17\lambda\sqrt{f_c'}).u_5 s_{v,5}/(\phi f_{yv})$ assuming 4 peripherals							*cl.22.6.6.1, cl.22*
3.2		where ϕ=0.75, λ={1.00 NWC, 0.75 LWC}, $f_{yv} \leq 420N/mm^2$							*cl.21.2, cl.22.6*
6.3.1	**Case $v_5 > \phi 0.5\sqrt{f_c'}$**				**1.98**		**N/A**		*cl.22.6.6.2, cl.22*
Fourth shear perimeter shear utilisation							**57%**		**OK**

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Detailing Requirements

All detailing requirements met ? **OK**

Max column strip sagging steel reinforcement pitch in x (<3$d_{x,s}$, <750mm)	150	mm	OK
Max column strip sagging steel reinforcement pitch in y (<3$d_{y,s,c}$, <750mm)	150	mm	OK
Max column strip hogging steel reinforcement pitch in x (<3$d_{x,h,c}$, <750mm)	150	mm	OK
Max column strip hogging steel reinforcement pitch in y (<3$d_{y,h,c}$, <750mm)	150	mm	OK
Max middle strip sagging steel reinforcement pitch in x (<3$d_{x,s}$, <750mm)	150	mm	OK
Max middle strip sagging steel reinforcement pitch in y (<3$d_{y,s,m}$, <750mm)	150	mm	OK
Max middle strip hogging steel reinforcement pitch in x (<3$d_{x,h,m}$, <750mm)	150	mm	OK
Max middle strip hogging steel reinforcement pitch in y (<3$d_{y,h,m}$, <750mm)	150	mm	OK

Maximum pitch of bars: (Notation as for BS 8110)

	%A_s/bh	Maximum Pitch (mm)
Main bars :	0.5 or less	300
	1.0 or more	150

Max column strip sagging steel reinforcement pitch in x	150	mm	OK
Max column strip sagging steel reinforcement pitch in y	150	mm	OK
Max column strip hogging steel reinforcement pitch in x	150	mm	OK
Max column strip hogging steel reinforcement pitch in y	150	mm	OK
Max middle strip sagging steel reinforcement pitch in x	150	mm	OK
Max middle strip sagging steel reinforcement pitch in y	150	mm	OK
Max middle strip hogging steel reinforcement pitch in x	150	mm	OK
Max middle strip hogging steel reinforcement pitch in y	150	mm	OK

Note that d_{dp} only incorporated in column strip hogging above if w_{dp} >= l_x/3.
Note that d_{dp} only incorporated in column strip sagging above if w_{dp} >= l_x/3 and banded.

Min column strip sagging steel reinforcement pitch in x (>75mm+$\phi_{sx,col}$, >100	150	mm	OK
Min column strip sagging steel reinforcement pitch in y (>75mm+$\phi_{sy,col}$, >100	150	mm	OK
Min column strip hogging steel reinforcement pitch in x (>75mm+$\phi_{hx,col}$, >100	150	mm	OK
Min column strip hogging steel reinforcement pitch in y (>75mm+$\phi_{hy,col}$, >100	150	mm	OK
Min middle strip sagging steel reinforcement pitch in x (>75mm+$\phi_{sx,mid}$, >100	150	mm	OK
Min middle strip sagging steel reinforcement pitch in y (>75mm+$\phi_{sy,mid}$, >100	150	mm	OK
Min middle strip hogging steel reinforcement pitch in x (>75mm+$\phi_{hx,mid}$, >100	150	mm	OK
Min middle strip hogging steel reinforcement pitch in y (>75mm+$\phi_{hy,mid}$, >100	150	mm	OK

Note an allowance has been made for laps in the min pitch by increasing the criteria by the bar diameter.

% Max column strip sag reinforcement in x (<= 0.04bh)	0.34	%	OK
% Max column strip sag reinforcement in y (<= 0.04b(h+d_{dp}))	0.34	%	OK
% Max column strip hog reinforcement x (<= 0.04b(h+d_{dp}))	0.52	%	OK
% Max column strip hog reinforcement y (<= 0.04b(h+d_{dp}))	0.52	%	OK
% Max middle strip sag reinforcement in x (<= 0.04bh)	0.19	%	OK
% Max middle strip sag reinforcement in y (<= 0.04bh)	0.19	%	OK
% Max middle strip hog reinforcement x (<= 0.04bh)	0.19	%	OK
% Max middle strip hog reinforcement y (<= 0.04bh)	0.19	%	OK

Note that d_{dp} only incorporated in column strip hogging above if w_{dp} >= l_x/3.
Note that d_{dp} only incorporated in column strip sagging above if w_{dp} >= l_x/3 and banded.

Sagging steel reinforcement diameter in x, $\phi_{sx,col}$ (>=6mm)	16	mm	OK
Sagging steel reinforcement diameter in y, $\phi_{sy,col}$ (>=6mm)	16	mm	OK
Hogging steel reinforcement diameter in x, $\phi_{hx,col}$ (>=6mm)	20	mm	OK
Hogging steel reinforcement diameter in y, $\phi_{hy,col}$ (>=6mm)	20	mm	OK
Sagging steel reinforcement diameter in x, $\phi_{sx,mid}$ (>=6mm)	12	mm	OK
Sagging steel reinforcement diameter in y, $\phi_{sy,mid}$ (>=6mm)	12	mm	OK
Hogging steel reinforcement diameter in x, $\phi_{hx,mid}$ (>=6mm)	12	mm	OK
Hogging steel reinforcement diameter in y, $\phi_{hy,mid}$ (>=6mm)	12	mm	OK

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Deflection Criteria

Span in x

Span, x		10.000	m
Span, x / effective depth, $d_{x,s}$ ratio		**28.5**	
Basic span / effective depth ratio criteria (20 single span; 23 edge; 26 cont)		26.0	
Multiplier $C_{1,\text{span more or less than 10m}}$	Include	1.00	
Multiplier $C_{1,\text{flat slab}}$		0.90	*cl.3.7.8 BS81*
Modification factor for tension C_2			
m_x/b = MAX ($M_{sag,lx,m}/w_{2,lx}$, $M_{sag,lx,c}/w_{1,lx}$)	Col Strip Governs	128.7	kNm/m
$m_x/bd_{x,s}^2$		1.04	N/mm²
$f_s = \frac{2 f_y A_{s\ req}}{3 A_{s\ prov}} \times \frac{1}{\beta_b}$	(β_b=1.2 unless single span or continuous ela	220	N/mm²
Modification $0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \leq 2.0$		1.65	
Modified span / effective depth ratio criteria		**38.7**	
Deflection utilisation		**74%**	**OK**

Span in y

Span, y		10.000	m
Span, y / effective depth, $d_{y,s,c/m}$ ratio		**27.2**	
Basic span / effective depth ratio criteria (20 single span; 23 edge; 26 cont)		26.0	
Multiplier $C_{1,\text{span more or less than 10m}}$	Include	1.00	
Multiplier $C_{1,\text{flat slab}}$		0.90	*cl.3.7.8 BS81*
Modification factor for tension C_2			
m_y/b = MAX ($M_{sag,ly,m}/w_{2,ly}$, $M_{sag,ly,c}/w_{1,ly}$)	Col Strip Governs	128.7	kNm/m
$m_y/bd_{y,s,c/m}^2$		0.96	N/mm²
$f_s = \frac{2 f_y A_{s\ req}}{3 A_{s\ prov}} \times \frac{1}{\beta_b}$	(β_b=1.2 unless single span or continuous ela	210	N/mm²
Modification $0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \leq 2.0$		1.75	
Modified span / effective depth ratio criteria		**40.9**	
Deflection utilisation		**67%**	**OK**

Utilisation in x and y

Deflection utilisation		**74%**	**OK**

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Scheme Design: Flat Slab

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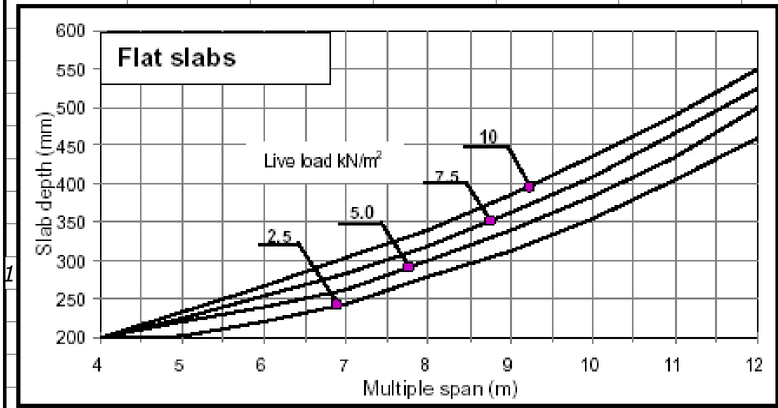

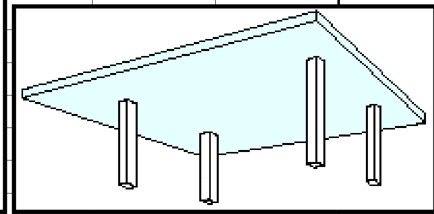

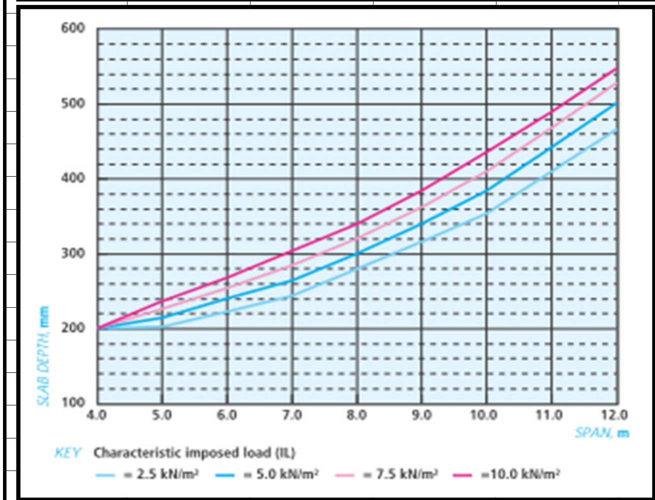

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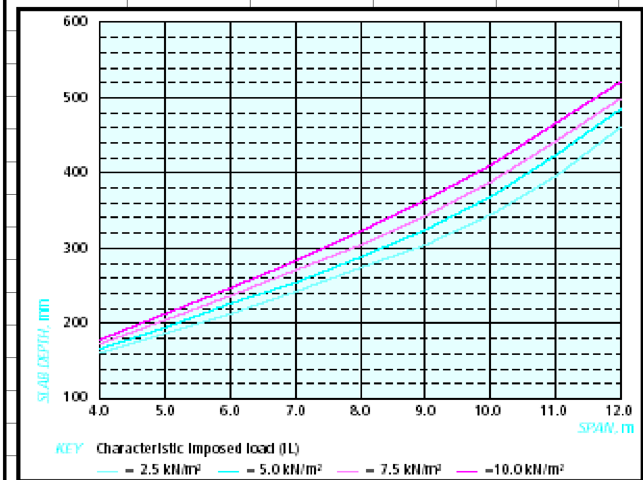
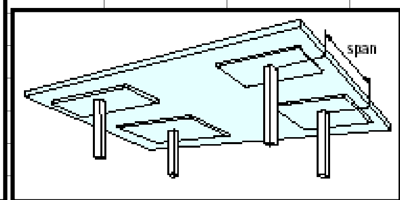
Scheme Design: Flat Slab With Drops





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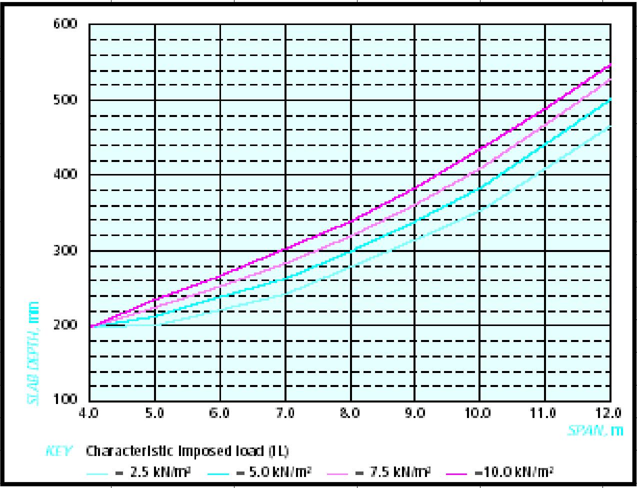
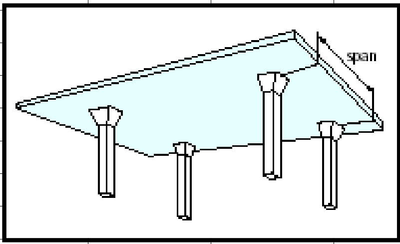
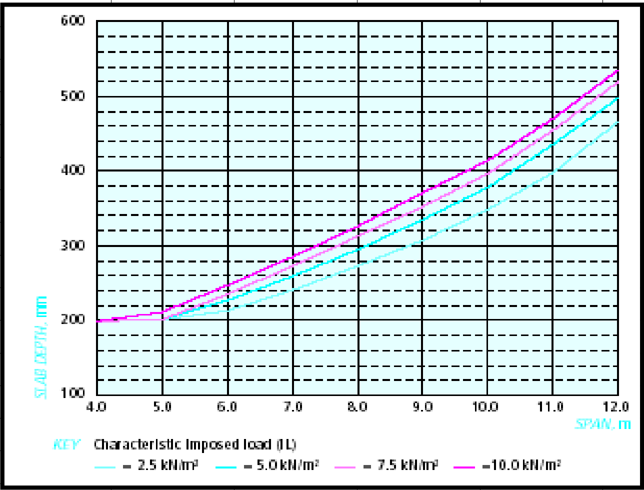
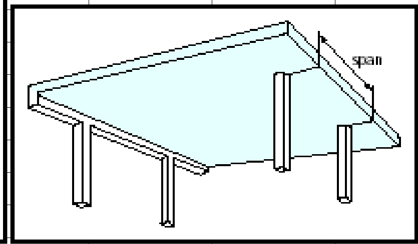
Scheme Design: Flat Slab With Column Heads





Scheme Design: Flat Slab With Edge Beams





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Beam Section Input Description

Edge Beam

	Depth	Width	Sag Section
No downstand	*slab*	b_{eff}	*Rect - continuous*
With downstand	*slab + downstand*	b_w	*L - continuous*

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Typical Initial Span / Effective Depth Ratios

Table 3 Span/effective depth ratios for initial design of slabs

	One-way spanning			Two-way spanning		Flat slab
Imposed load kN/m²	simply supported	continuous	cantilever	simply supported	continuous	
5.0	23	30	11	30	39	28
10.0	21	27	10	27	35	25

Flat slab design should be based on the longer span dimension. For exterior panels, 85% of the ratios quoted in Table 3 should be used.

For ribbed slabs, 85% of the ratios quoted in Table 3 should be used.

Span-to-depth ratios for flat slabs

spans are in the range 4 to 10 m.

Imposed load, Q_k (kN/m²)	Multiple span
2.5	28
5.0	26
7.5	25
10.0	23

Note
This table assumes a 3 x 3 bay layout. Where there are only 2 bays in one direction the ratio will need to be decreased.

Economic depths (mm) for multiple span flat slabs

Imposed load	Span (m) 4	5	6	7
2.5	200	202	222	244
5.0	200	214	240	264
7.5	200	226	254	284
10.0	200	236	268	304

Assumptions
- Class C28/35 concrete
- Super-imposed dead load of 1.5 kN/m²
- Perimeter load of 10 kN/m for cladding

8	9	10	11	12
280	316	354	410	466
300	340	384	442	502
320	362	410	468	528
340	384	436	490	548

- Fire resistance 1 hour (increase depth by 10 mm for 2 hours)
- Multiple spans (increase depth by 10 mm for 2 spans)
- No holes

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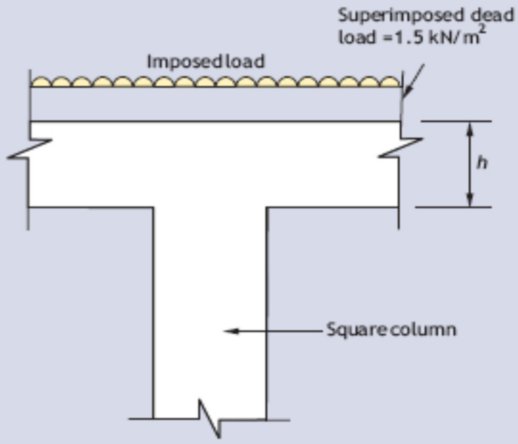
Punching shear is often a governing criterion for flat slabs and should be checked at the initial stages of design. Table 2.17 gives the maximum floor area for a selection of imposed loads and column sizes. It assumes a superimposed dead load of 1.5 kN/m², internal conditions, a value for v_c of 0.75 N/mm², with v limited to $1.6v_c$.

Table 2.17
Punching shear: maximum panel areas for flat slabs (m²)

Overall slab depth, *h* (mm)	Imposed load (kN/m²)			
	2.5	5.0	7.5	10
300 x 300 column				
200	38.6	29.4	23.8	19.9
225	46.2	35.7	29.1	24.6
250	54.0	42.3	34.8	29.5
275	59.7	47.3	39.2	33.5
300	67.7	54.3	45.3	38.9
325	75.9	61.4	51.6	44.5
350 x 350 column				
200	41.1	31.3	25.3	21.2
225	49.0	37.9	30.9	26.1
250	56.9	44.6	36.7	31.2
275	62.8	49.8	41.2	35.2
300	70.9	56.9	47.5	40.7
325	79.2	64.2	53.9	46.5
400 x 400 column				
200	43.7	33.3	26.9	22.5
225	51.7	40.0	32.6	27.5
250	59.9	46.9	38.6	32.8
275	65.8	52.2	43.3	36.9
300	74.2	59.5	49.6	42.6
325	82.6	66.9	56.2	48.5

Overall slab depth, *h* (mm)	Imposed load (kN/m²)			
	2.5	5.0	7.5	10
450 x 450 column				
200	46.2	35.2	28.4	23.9
225	54.5	42.1	34.3	29.0
250	62.8	49.3	40.5	34.4
275	68.9	54.6	45.3	38.6
300	77.4	62.1	51.8	44.4
325	86.0	69.6	58.5	50.4
500 x 500 column				
200	48.7	37.1	30.0	25.2
225	57.2	44.2	36.1	30.5
250	65.8	51.6	42.4	36.0
275	71.9	57.1	47.3	40.4
300	80.6	64.6	53.9	46.3
325	89.4	72.4	60.8	52.4



Notes

1. Superimposed load of 1.5 kN/m² included.
2. Cover of 25 mm has been assumed.
3. v_c for main reinforcement is 0.75 N/mm².
4. v for punching reinforcement is limited to $1.6\,v_c$.
5. Shear links should be provided in accordance with BS 8110.

How to use this table

For example:
300 x 300 column
250 thick slab
5 kN/m² imposed load
From table maximum area that can be supported = 42.3 m²
(e.g. 6.5 x 6.5 m grid)

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Table 1:
Span/depth ratios for *insitu* concrete slabs *(from Reynolds's Reinforced Concrete Designer's Handbook)*

Slab type	5 kN/m² Imposed load	10 kN/m² Imposed load
Simply supported one-way	27	24
Simply supported two-way	30	27
Continuous one-way	34	30
Continuous two-way	44	40
Cantilever	11	10
Flat slab	30	27

Table 4: Estimated depths of *insitu* concrete flat slabs with no column heads

Span	4m	5m	6m	7m	8m	9m	10m
Multi span thickness	200mm	200mm	225mm	250mm	250mm	300mm	350mm