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Effects From Structural Analysis — Do not adopt code equivalence

Design axial force, F (tension -ve and comp. +ve) (ensure < $0.1f_{cu}b_wh$)	0	kN	OK
Design shear force, V_d	1170	kN	
Design bending moment, M	1849	kNm	
Design torsion moment, T	0	kNm	

Material Properties

Characteristic strength of concrete, f_{cu} / f_{ck} | f_c' (f_{cu} ≤ 75 or 105N/	35	28	N/mm^2	OK
Yield strength of longitudinal steel, f_y	Higher	500	N/mm^2	
Yield strength of shear and torsion link steel, f_{yv}	Higher	500	N/mm^2	

Section Dimensions

Span (effective width, deflection, long'l shear and deep beam calcs)	10.000	m	
Available beam spacing	1.000	m	
(effective width calcs; usual spacing for interior beams; half for edge beams)			
Beam section type and support condition	Rect - continuous		
(section type for bending, defl'n calcs; support condition for LTB restraint, eff width, defl'n, long'l shear calcs)			
Ratio β_b (1.0 for no moment redistribution, ≥0.70, ≤1.30)	1.00		OK

Section Type and Support Condition Option Selection

Downstand Beam					*Upstand Beam*				
Support	*Effect*	*Slab*	*Type*	*Defl'n*	*Support*	*Effect*	*Slab*	*Type*	*Defl'n*
S/S	*Sag*	*Precast*	*Rect-s/s*	*Yes*	*S/S*	*Sag*	*Precast*	*Rect-s/s*	*Yes*
S/S	*Sag*	*Insitu*	*T/L-s/s*	*Yes*	*S/S*	*Sag*	*Insitu*	*Rect-s/s*	*Yes*
Cont.	*Sag*	*Precast*	*Rect-cont.*	*Yes*	*Cont.*	*Sag*	*Precast*	*Rect-cont.*	*Yes*
Cont.	*Sag*	*Insitu*	*T/L-cont.*	*Yes*	*Cont.*	*Sag*	*Insitu*	*Rect-cont.*	*Yes*
Cont.	*Hog*	*Precast*	*Rect-cont.*	*N/A*	*Cont.*	*Hog*	*Precast*	*Rect-cont.*	*N/A*
Cont.	*Hog*	*Insitu*	*Rect-cont.*	*N/A*	*Cont.*	*Hog*	*Insitu*	*T/L-cont.*	*N/A*
Cant.	*Hog*	*Precast*	*Rect-cant.*	*Yes*	*Cant.*	*Hog*	*Precast*	*Rect-cant.*	*Yes*
Cant.	*Hog*	*Insitu*	*Rect-cant.*	*Yes*	*Cant.*	*Hog*	*Insitu*	*T/L-cant.*	*Yes*

Overall depth, h (includes insitu slab thks; excludes precast slab thks; span/	1000	mm	OK
Depth of flange, h_f *(bending flanged beam, longitudinal shear calcs)*	200	mm	
Width (rectangular) or web width (flanged), b_w	500	mm	
Cover to all reinforcement, cover (usually 35 (C35) or 30 (C40) internal; 40	25	mm	
Add cover to tension steel (due to transverse steel layer(s)), $cover_{add,t}$	0	mm	
Add cover to compression steel (due to transverse steel layer(s)), $cover_{add,c}$	0	mm	
Effective depth to tension steel, d = h - cover - MAX(ϕ_{link}, $\phi_{link,t}$, $cover_{add,t}$) - [	917	mm	
Effective depth to compression steel, d' = cover + MAX(ϕ_{link}, $\phi_{link,t}$, $cover_{add,c}$)	35	mm	

Longitudinal Reinforcement Details

Tension steel reinforcement diameter, ϕ_t	32	mm	
Tension steel reinforcement number, n_t	8		
Tension steel area provided, $A_{s,prov} = n_t.\pi.\phi_t^2/4$	6434	mm^2	
Compression steel reinforcement diameter, ϕ_c (where applicable)	None	mm	
Compression steel reinforcement number, n_c	0		
Compression steel area provided, $A_{s,prov}' = n_c.\pi.\phi_c^2/4$	0	mm^2	
Number of layers of tension steel, $n_{layers,tens}$	2	layer(s)	
Spacer for tension steel, $s_{r,tens}$ (≥ MAX (ϕ_t, 25mm))	32	mm	OK
Number of layers of compression steel, $n_{layers,comp}$	1	layer(s)	
Spacer for compression steel, $s_{r,comp}$ = MAX (ϕ_c, 25mm)	25	mm	

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Shear Reinforcement Details

Shear link diameter, ϕ_{link}	10	mm
Number of shear links in a cross section, i.e. number of legs, n_{leg}	4	
Area provided by all shear links in a cross-section, $A_{sv,prov} = \pi.\phi_{link}^2/4.n_{leg}$	314	mm^2
Pitch of shear links, S	100	mm

Torsion Reinforcement Details

Torsion link diameter, $\phi_{link,t}$				None	mm
Number of torsion links in a cross section, i.e. number of legs, $n_{leg,t}$ = 2				2	
Area provided by all torsion links in a cross-section, $A_{sv,prov,t} = \pi.\phi_{link,t}^2/4.n_{leg,t}$				0	mm^2
Pitch of torsion links, S_t				200	mm
Note that further longitudinal steel $A_{s,t}$ mus	*0*	*0*	*0*	*0*	mm^2

Utilisation Summary

Item				UT	Remark
RB tens steel		*86%*	*79%*	86%	OK
RB comp steel		*N/A*	*N/A*	N/A	N/A
RB % min tens reinf.		*10%*	*20%*	20%	OK
RB % min comp reinf.				N/A	N/A
RB % max tens and comp reinf.				32%	OK
FB tens steel				N/A	N/A
FB comp steel				N/A	N/A
FB % min tens reinf.				N/A	N/A
FB % min comp reinf.				N/A	N/A
FB % max tens and comp reinf.				N/A	N/A
RB shear ultimate str	*54%*	*57%*	*77%*	77%	OK
RB shear design capa	*72%*	*42%*	*96%*	96%	OK
RB torsion ultimate st	*0%*	*0%*	*0%*	0%	OK
RB torsion design cap	*0%*	*0%*	*0%*	0%	OK
RB shear and torsion	*54%*	*57%*	*77%*	77%	OK
RB or FB deflection requirements				49%	OK
Total utilisation rectangular beam				**96%**	**OK**
Total utilisation rectangular beam (excluding def				**96%**	**OK**
Total utilisation flanged beam				**N/A**	**N/A**
Detailing requirements				**OK**	

Note RB = rectangular beam, FB = flanged beam;

% Tension reinforcement (rectangular)		**1.29**	%	
% Compression reinforcement (rectangular)		**N/A**	%	
% Tension reinforcement (flanged)		**N/A**	%	
% Compression reinforcement (flanged)		**N/A**	%	
Estimated steel reinforcement quantity (125 – 160kg/m^3)		**158**	kg/m^3	
7850 . [($A_{s,prov}$ +$A_{s,prov}$') / (b_w or b).(h or h_f) + ($A_{sv,prov}$.(h+anc.)/S+$A_{sv,prov,t}$.(h+b_w+anc.)/S_t)				
Estimated steel reinforcement quantity (125 – 160kg/m^3)		**222**	kg/m^3	*IStructE*
11000 . [($A_{s,prov}$ +$A_{s,prov}$') / (b_w or b).(h or h_f) + ($A_{sv,prov}$.(h+anc.)/S+$A_{sv,prov,t}$.(h+b_w+anc.)/S_t)				
[Note that steel quantity in kg/m^3 can be obtained from 110.0 x % rebar];				
Material cost: concrete, c	315 units/m^3 steel, s	4600	units/tonne	
Reinforced concrete material cost = [c+(est. rebar quant).s].(b_w		**667**	units/m	

Ductility of failure mechanism	Consider	**Section Under Reinforced**	**OK**
Rectangular beam crack width		**101%**	**NOT OK**
Max LTB stability (compression flange) restraints spacing, L_{LTB}		**30.0**	m

Note s/s / cont L_{LTB} = MIN (60(b_w or b), 250(b_w or b)^2/d) and cant L_{LTB} = MIN (25b_w, 100b_w^2/

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Material Stresses

Axial stress, $F/b_w h$	0.00	N/mm^2
Shear stress, $V_d/b_w d$	2.55	N/mm^2
Bending stress, $6M/(b_w \text{ or } b).d^2$	26.39	N/mm^2
Torsion stress, $2T/((h_{min}{}^2)(h_{max}-h_{min}/3))$	0.00	N/mm^2

Material Stress-Strain Curves

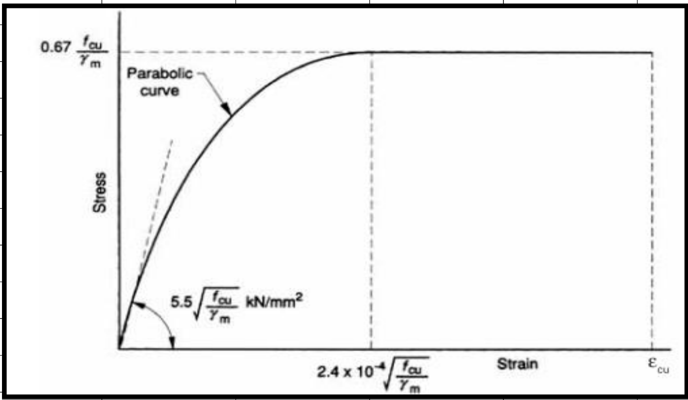


Figure 2.1 — Short term design stress-strain curve for normal-weight concrete

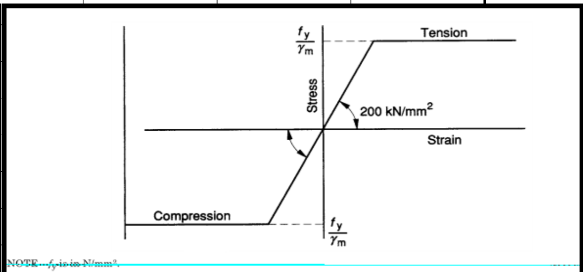


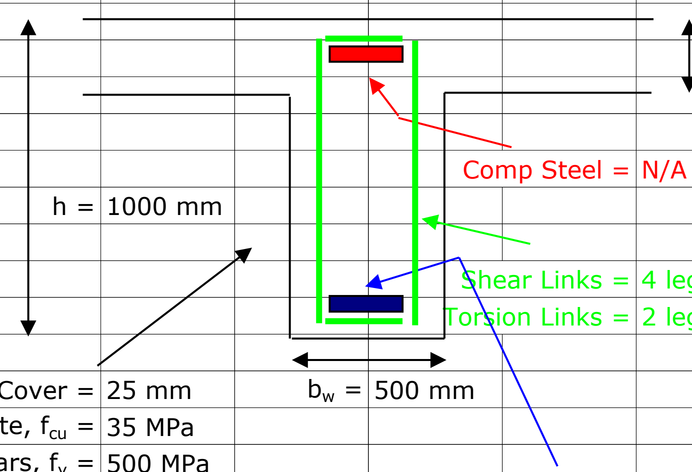
Figure 2.2 — Short term design stress-strain curve for reinforcement

For $f_{cu} \le 60$ N/mm², $\varepsilon_{cu} = 0.0035$

For $f_{cu} > 60$ N/mm², $\varepsilon_{cu} = 0.0035 - (f_{cu} - 60)/50000$

Detailing Instructions

Not to Scale



h_f = N/A

Comp Steel = N/A

h = 1000 mm

d = 917 mm

d' = 35 mm

Shear Links = 4 legs of T10@100mm pitch

Torsion Links = 2 legs of T0@200mm pitch

Cover =	25 mm
Concrete, f_{cu} =	35 MPa
Rebars, f_y =	500 MPa
Links, f_{yv} =	500 MPa

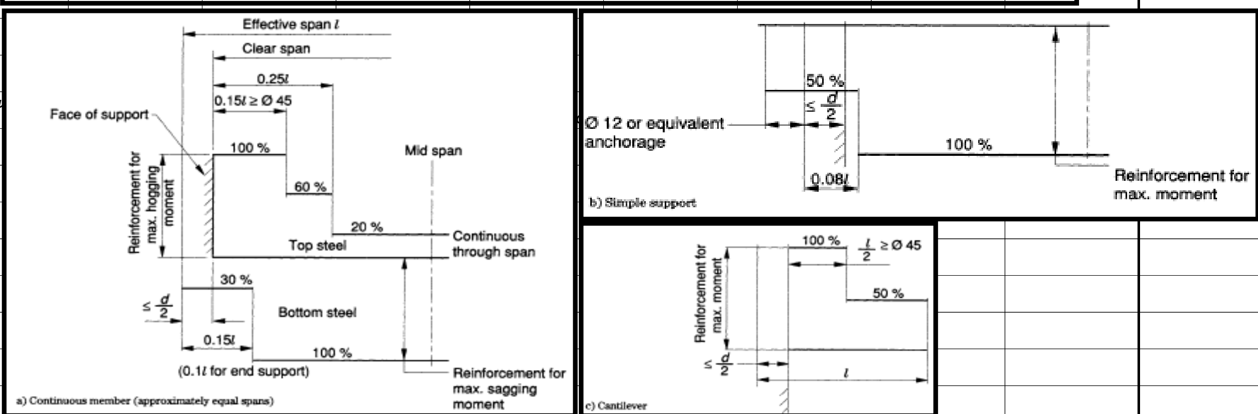
b_w = 500 mm

Tension Steel = 8 T32@133mm centres

Diagram Produced for Sagging Design; Flip if Hogging Design;

/ b w . h

/ b w



(d);

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Additional Longitudinal Shear Rectangular or Flanged Beam Utilisation Summary

Longitudinal shear between web and flange	Consider only if applicable		
Longitudinal shear within web	Consider only if applicable		
Length under consideration, Δx (span/2 s/s, span/4 cont, span cant)	2500	mm	
Applicability of longitudinal shear design	Applicable		

Longitudinal Shear Between Web and Flange (EC2)		
Longitudinal shear stress limit to prevent crushing	32%	OK
Longitudinal shear stress limit for no transverse reinforcement	312%	NOT OK
Required design transverse reinforcement per unit length	94%	OK
Longitudinal Shear Between Web and Flange (BS5400-4)		
Longitudinal shear force limit per unit length	64%	OK
Required nominal transverse reinforcement per unit length	38%	OK
Longitudinal Shear Between Web and Flange Mandatory Cr	94%	OK
Longitudinal Shear Within Web (EC2)		
Longitudinal shear stress limit	49%	OK
Longitudinal Shear Within Web (BS8110)		
Longitudinal shear stress limit for no nominal / design vertical rei	110%	NOT OK
Required nominal vertical reinforcement per unit length	24%	OK
Required design vertical reinforcement per unit length	94%	OK
Longitudinal Shear Within Web (BS5400-4)		
Longitudinal shear force limit per unit length	79%	OK
Required nominal vertical reinforcement per unit length	24%	OK
Longitudinal Shear Within Web Mandatory Criteria	94%	OK

Additional Deep Beam Rectangular Beam Utilisation Summary

Deep beam design	Consider only if applicable			
Span to depth ratio, span / h		10.00		OK
Applicability of deep beam design	Not Applicable			
Concrete type	Normal weight			
Horizontal shear link diameter, $\phi_{link,h}$	None		mm	
Number of horizontal shear links in a horizontal section, i.e. number of legs,		0		
Pitch of horizontal shear links, S_h		0	mm	
Clear distance from edge of load to face of support, a_1 o	DL @ mid 0.625h	N/A	mm	
Detailing requirements		N/A		

Reynolds				
Tension steel (deep beam)		N/A		N/A
Tension steel zone depth, T_{zone} (sag s/s, sag cont, hog cant)		N/A	mm	
Tension steel zone depth, T_{zone} (hog cont)		N/A	mm	
Minimum breadth for deep beam, b_w		N/A	mm	N/A
Shear ultimate force (deep beam)		N/A		N/A
Shear design capacity (deep beam)		N/A		N/A
CIRIA Guide 2				
Bending ultimate moment (deep beam)		N/A		N/A
Tension steel (deep beam)		N/A		N/A
Tension steel zone depth, T_{zone} (sag s/s, sag cont, hog cant)		N/A	mm	
Tension steel zone depth, T_{zone} (hog cont) upper band	N/A	N/A	mm	
Tension steel zone depth, T_{zone} (hog cont) lower band	N/A	N/A	mm	
Shear ultimate force (deep beam)		N/A		N/A
Shear design capacity (deep beam)		N/A		N/A

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BS8110

Bending Rectangular Beam (Singly or Doubly Reinforced) (BS8110)

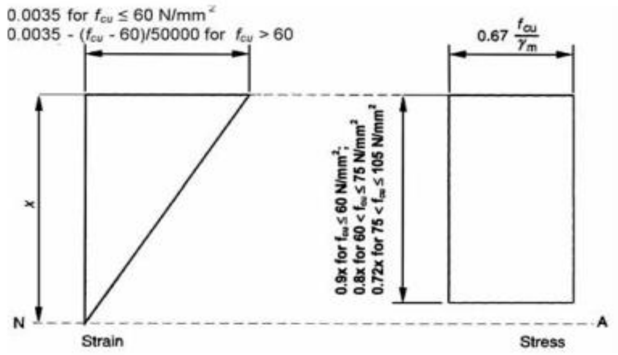

Figure 3.3 (amended) – Simplified stress block for concrete at ultimate limit state

$M_u = K'f_{cu}bd^2$	*Note b here is b_w;*		2296	kNm	
Ratio, K'			0.156		
$0.90 \le \beta_b \le 1.10$	K' =	0.156 for $f_{cu} \le 60$ N/mm²	0.156	mm	3.4.4.4 BC
		0.120 for 60< $f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
		0.094 for 75< $f_{cu} \le 105$ N/mm² *[β_b =1.0]*	N/A	mm	3.4.4.4 BC
$0.90 > \beta_b > 1.10$	K' =	$0.402(\beta_b-0.4) - 0.18(\beta_b-0.4)^2$, for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
		$0.357(\beta_b-0.5) - 0.143(\beta_b-0.5)^2$ for 60< $f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC

Note the expression for K' ensures that the section remains ductile i.e. ensuring that failure occurs with the gradual yielding of the tension steel and not by a sudden catastrophic compression failure of the concrete;

If: $M < M_u$ → no compression steel — **VALID**

$K = M/bd^2f_{cu}$ *Note b here is b_w;* — **0.126**

To scheme beam, choose d such that $K = M/(b_w d^2 f_{cu}) < K'$ (0.156 for β_b =1.0 and NSC) for no compression steel, i.e. d > 823 and h > 906 mm

$$z = d\left\{0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)}\right\}$$ z <=0.95d — **763** mm

Depth of neutral axis, $x \le x_{limit}$		342 ≤	459	mm	OK
Neutral axis, x	x =	$(d - z)/0.45$, for $f_{cu} \le 60$ N/mm²	342	mm	3.4.4.4 BC
		$(d - z)/0.40$, for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
		$(d - z)/0.36$, for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC
Neutral axis limit, x_{limit}	$0.90 \le \beta_b \le 1.10$	$x \le 0.5d$ for $f_{cu} \le 60$ N/mm²	459	mm	3.4.4.4 BC
		$x \le 0.4d$ for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
		$x \le 0.33d$, for $75 < f_{cu} \le 105$ N/mm² *[β_b =1.0]*	N/A	mm	3.4.4.4 BC
	$0.90 > \beta_b > 1.10$	$x \le (\beta_b - 0.4)d$ for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
		$x \le (\beta_b - 0.5)d$ for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC

Tension steel $A_s = \frac{M}{(0.95f_y)z}$, $f_y \le 460$N/mm² — **5544** mm² — *Foreword*

Back-analysis of x in $(0.67f_{cu}/1.5).(s.b_w)=(f_y/1.05).(A_{s,prov})$, $f_y \le 46$	401	mm	OK
s =	**0.9x for $f_{cu} \le 60$ N/mm²;**	0.90x	mm	3.4.4.4 BC
	0.8x for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
	0.72x for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC

Note for an under reinforced section, require $x \le x_{limit}$ for yielding of tension steel;

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If: $M > M_u$ → compression steel required		N/A		
To scheme beam, with compression steel, choose d such that $K = M/(b_w d^2 f_{cu}) < 10/f_{cu}$				
for non-excessive comp steel, i.e. d > 608 and h >		691	mm	
$z = d\left\{0.5 + \sqrt{\left(0.25 - \frac{K'}{0.9}\right)}\right\}$		N/A	mm	
Depth of neutral axis, $x \le x_{limit}$	N/A ≤	N/A	mm	N/A
Neutral axis, x: x =	$(d-z)/0.45$, for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
	$(d-z)/0.40$, for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
	$(d-z)/0.36$, for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC
Neutral axis limit, x_{limit}: $0.90 \le \beta_b \le 1.10$	$x \le 0.5d$ for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
	$x \le 0.4d$ for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
	$x \le 0.33d$, for $75 < f_{cu} \le 105$ N/mm² [β_b=1.0]	N/A	mm	3.4.4.4 BC
$0.90 > \beta_b > 1.10$	$x \le (\beta_b - 0.4)d$ for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
2	$x \le (\beta_b - 0.5)d$ for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
2				
2				
2 Compression steel	$A_s' = \frac{M - K' f_{cu} b d^2}{0.95 f_y (d-d')}$, $f_y \le 460$N/mm²	N/A	mm²	Foreword
2				
Tension steel	$A_s = \frac{M_u}{0.95 f_y z} + A_s'$, $f_y \le 460$N/mm²	N/A	mm²	Foreword
If $d' > \left(1 - \frac{f_y}{735}\right)x$, use $700\left(1 - \frac{d'}{x}\right)$ in lieu of $0.95f_y$	, $f_y \le 460$N/mm²	N/A		Foreword
Back-analysis of x in $(0.67f_{cu}/1.5).(s.b_w)+(f_y/1.05).(A_{s,prov}')=(f_y/1$		N/A	mm	N/A
s =	0.9x for $f_{cu} \le 60$ N/mm²;	N/A	mm	3.4.4.4 BC
	0.8x for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
	0.72x for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC

Note for an under reinforced section, require $\varepsilon_{st} = \varepsilon_{cc}(d-x)/x \ge \varepsilon_y$ for yielding of tension steel
and $\varepsilon_{sc} = \varepsilon_{cc}(x-d')/x \ge \varepsilon_y$ for yielding of compression steel, where ε_{cc}=0.0035 for $f_{cu} \le 60$N/mm
and $\varepsilon_{cc}=0.0035-(f_{cu}-60)/50000$ for $f_{cu} > 60$N/mm² and $\varepsilon_y=(f_y/1.05)/E_s$;

Tension steel area provided	6434	mm²	
Tension steel area provided utilisation	86%		OK
Compression steel area provided	N/A	mm²	
Compression steel area provided utilisation	N/A		N/A
% Min tension reinforcement	1.29	%	TR49
% Min tension reinforcement (>= 0.0024$b_w h$ G250; >= MAX (0.0013, 0.0013$(f_{cu}/40)^{2/3}$).$b_w h$ G			cl.3.1.7
% Min tension reinforcement utilisation	10%		OK
% Min compression reinforcement (>= 0.002b_wh)	N/A	%	
% Min compression reinforcement utilisation	N/A		N/A
% Max tension reinforcement (<= 0.04b_wh)	1.29	%	
% Max tension reinforcement utilisation	32%		OK
% Max compression reinforcement (<= 0.04b_wh)	N/A	%	
% Max compression reinforcement utilisation	N/A		N/A
% Max tension or compression reinforcement utilisation	32%		OK
2			
2			
2			

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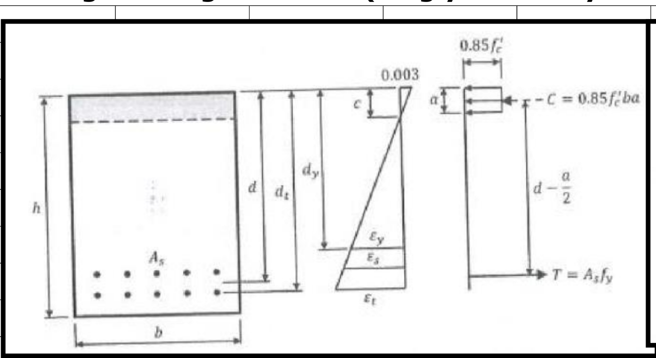
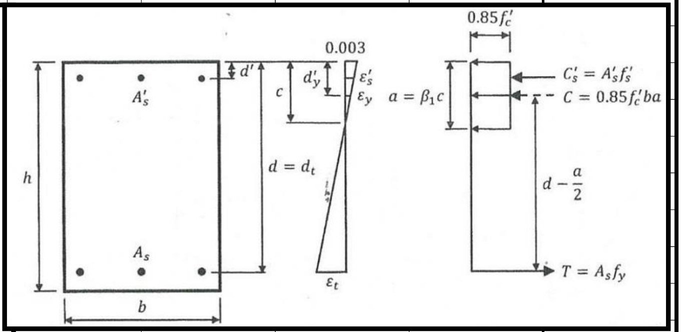
ACI318

Bending Rectangular Beam (Singly or Doubly Reinforced) (ACI318)





Singly Reinforced Rectangular Section | **Doubly Reinforced Rectangular Section**

Required nominal flexural strength coefficient of resistance, $R_n = M^* / \phi b_w d^2$	**4.89**	N/mm^2
Strength reduction factor for tension-controlled sections, ϕ = 0.90	0.90	
Ratio of A_s to $b_w d$, $\rho_w = 0.85f_c'/f_y.[1-\sqrt{(1-2R_n/(0.85f_c'))}]$, $f_y \le 550N/mm^2$	1.11	%
Tension steel, $A_s = MAX(\rho_w b_w d, A_{s,min})$	5070	mm^2
$A_{s,min} = MAX(0.25b_w d\sqrt{f_c'}/f_y, 1.4b_w d/f_y)$, $f_y \le 550N/mm^2$	1284	mm^2
Factor, $\beta_1 = MAX(MIN(0.85-(0.05/7).(f_c'-28),0.85),0.65)$	0.85	

Note β_1 is the factor relating depth of equivalent rectangular compressive stress block to neutral axis depth;

Depth of equivalent rectangular stress block, $a = A_s.f_y/(0.85f_c'b_w)$, $f_y \le 550N/$	213	mm
Distance, $c = a/\beta_1$	251	mm

Note c is the distance from extreme compression fiber to neutral axis (for singly reinforced section);

Distance, $d_t = h - cover - MAX(\phi_{link}, \phi_{link,t}, cover_{add,t}) - \phi_t/2$	949	mm

Note d_t is the distance from extreme compression fiber to centroid of extreme layer of longitudinal tension st

$\varepsilon_t = 0.003(d_t/c-1)$	**0.008**	

Note ε_t is the net tensile strain in extreme layer of longitudinal tension steel at nominal strength;

ε_t > 0.005	→	***no compression steel***	**VALID**
	Tension steel, A_s		**5070** mm^2
0.004 < ε_t ≤ 0.005	→	***compression steel required***	**N/A**

Ratio of A_s to $b_w d$, $\rho_w = 0.319\beta_1 f_c'/f_y \cdot d_t/d$, $f_y \le 550N/mm^2$	N/A	%
$R_{nt} = \rho_w f_y(1-0.59\rho_w f_y/f_c')$, $f_y \le 550N/mm^2$	N/A	N/mm^2

Note R_{nt} is the required nominal flexural strength coefficient of resistance for rectangular beam corresponding to ε_t=0.005;

$M_{nt} = R_{nt} b_w d^2$	N/A	kNm

Note M_{nt} is the required nominal flexural strength corresponding to ε_t=0.005;

$M_n' = M^*/\phi - M_{nt}$	N/A	kNm

Note M_n' is the required nominal flexural strength that need to be resisted by the compression reinforcement;

Distance, $c_t = 0.375d_t$	N/A	mm

Note c_t is the distance from extreme compression fiber to neutral axis (for doubly reinforced section);

Stress in compression reinforcement, $f_s' = 0.003E_s(1-d'/c_t) \le f_y$, f	N/A	N/mm^2
Modulus of elasticity of reinforcement, E_s = 200,000	N/A	N/mm^2
Compression steel, $A_s' = M_n' / [f_s'(d-d')]$	**N/A**	mm^2
Tension steel, $A_s = \rho_w b_w d + M_n' / [f_y(d-d')]$, $f_y \le 550N/mm^2$	**N/A**	mm^2

ε_t ≤ 0.004	→	***no valid solution***	**N/A**

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			ACI318
Tension steel area provided	6434	mm^2	
Tension steel area provided utilisation	**79%**		**OK**
Compression steel area provided	N/A	mm^2	
Compression steel area provided utilisation	**N/A**		**N/A**
% Min tension reinforcement	1.29	%	*cl.9.6.1.2*
% Min tension reinforcement (>= MAX ($0.25b_w d \sqrt{f_c'}/f_y$, $1.4b_w d/f_y$))			
% Min tension reinforcement utilisation	**20%**		**OK**

eel;

BS8110

Bending Flanged Beam (Singly or Doubly Reinforced)

	Simply supported	Continuous	Cantilever
T-Beam	$b_w + L / 5$	$b_w + L / 7.14$	b_w
L-Beam	$b_w + L / 10$	$b_w + L / 14.29$	b_w
and ≤ (i) actual flange width, (ii) beam spacing			

For cantilevers, width shown is applicable for downstand beams as rect- sections.
For upstand beams, T- or L-sections will apply with insitu

Span	N/A	m	
Depth of flange, h_f	N/A	mm	N/A
Effective width, b = MIN(b_w + function (span, section, structure), beam spac	N/A	mm	
(bending flanged beam, deflection calcs flanged beam)			
$K = M/bd^2f_{cu}$	N/A		
$z = d \cdot [0.5 + (0.25-K/0.9)^{0.5}] <= 0.95d$	N/A	mm	
Depth of compression stress block, s = 2.(d-z) (applicable for all f_{cu})	N/A	mm	

Compression Stress Block in Flange ($s <= h_f$) N/A

$M_u = K'f_{cu}bd^2$	N/A	kNm	
Ratio, K'	N/A		
$0.90 \le \beta_b \le 1.10$: K' = 0.156 for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
K' = 0.120 for 60< $f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
K' = 0.094 for 75< $f_{cu} \le 105$ N/mm² [β_b=1.0]	N/A	mm	3.4.4.4 BC
$0.90 > \beta_b > 1.10$: K' = $0.402(\beta_b-0.4) - 0.18(\beta_b-0.4)^2$, for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
K' = $0.357(\beta_b-0.5) - 0.143(\beta_b-0.5)^2$ for 60< $f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC

Note the expression for K' ensures that the section remains ductile i.e. ensuring that failure occurs with the gradual yielding of the tension steel and not by a sudden catastrophic compression failure of the concrete;

If: $M < M_u$ → no compression steel — N/A

$K = M/bd^2f_{cu}$ — N/A

To scheme beam, choose d such that $K = M/(bd^2 f_{cu}) < K'$ (0.156 for β_b=1.0 and NSC)
for no compression steel, i.e. d > N/A and h > N/A mm

$$z = d\left\{0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)}\right\}$$ z <=0.95d — N/A mm

Depth of neutral axis, $x \le x_{limit}$	N/A ≤	N/A	mm	N/A
Neutral axis, x: x = $(d-z)/0.45$, for $f_{cu} \le 60$ N/mm²		N/A	mm	3.4.4.4 BC
x = $(d-z)/0.40$, for $60 < f_{cu} \le 75$ N/mm²		N/A	mm	3.4.4.4 BC
x = $(d-z)/0.36$, for $75 < f_{cu} \le 105$ N/mm²		N/A	mm	3.4.4.4 BC
Neutral axis limit, x_{limit}, $0.90 \le \beta_b \le 1.10$: $x \le 0.5d$ for $f_{cu} \le 60$ N/mm²		N/A	mm	3.4.4.4 BC
$x \le 0.4d$ for $60 < f_{cu} \le 75$ N/mm²		N/A	mm	3.4.4.4 BC
$x \le 0.33d$, for $75 < f_{cu} \le 105$ N/mm² [β_b=1.0]		N/A	mm	3.4.4.4 BC
$0.90 > \beta_b > 1.10$: $x \le (\beta_b - 0.4)d$ for $f_{cu} \le 60$ N/mm²		N/A	mm	3.4.4.4 BC
$x \le (\beta_b - 0.5)d$ for $60 < f_{cu} \le 75$ N/mm²		N/A	mm	3.4.4.4 BC

Tension steel $A_s = \frac{M}{(0.95f_y)z}$, $f_y \le 460$N/mm²	N/A	mm²	*Foreword*
Back-analysis of x in $(0.67f_{cu}/1.5).(s.b)=(f_y/1.05).(A_{s,prov})$, $f_y \le 46$	N/A	mm	N/A
s = **0.9x for $f_{cu} \le 60$ N/mm²;**	N/A	mm	3.4.4.4 BC
0.8x for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
0.72x for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC

Note for an under reinforced section, require $x \le x_{limit}$ for yielding of tension steel;

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If: $M > M_u$ → compression steel required	N/A		
To scheme beam, with compression steel, choose d such that $K = M/(b\,d^2 f_{cu}) < 10/f_{cu}$			
for non-excessive comp steel, i.e. d > N/A and h >	N/A	mm	
$z = d\left\{0.5 + \sqrt{\left(0.25 - \frac{K'}{0.9}\right)}\right\}$	N/A	mm	
Depth of neutral axis, $x \le x_{limit}$ — N/A ≤	N/A	mm	N/A
Neutral axis, x: x = $(d - z)/0.45$, for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
$(d - z)/0.40$, for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
$(d - z)/0.36$, for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC
Neutral axis limit, x_{limit} ($0.90 \le \beta_b \le 1.10$): $x \le 0.5d$ for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
$x \le 0.4d$ for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
$x \le 0.33d$, for $75 < f_{cu} \le 105$ N/mm² [β_b =1.0]	N/A	mm	3.4.4.4 BC
($0.90 > \beta_b > 1.10$): $x \le (\beta_b - 0.4)d$ for $f_{cu} \le 60$ N/mm²	N/A	mm	3.4.4.4 BC
$x \le (\beta_b - 0.5)d$ for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
2 Compression steel $A_s' = \frac{M - K' f_{cu} b d^2}{0.95 f_y (d - d')}$, $f_y \le 460$N/mm²	N/A	mm²	Foreword
2			
2 Tension steel $A_s = \frac{M_u}{0.95 f_y z} + A_s'$, $f_y \le 460$N/mm²	N/A	mm²	Foreword
2			
2			
If $d' > \left(1 - \frac{f_y}{735}\right)x$, use $700\left(1 - \frac{d'}{x}\right)$ in lieu of $0.95 f_y$, $f_y \le 460$N/mm²	N/A		Foreword
Back-analysis of x in $(0.67f_{cu}/1.5).(s.b)+(f_y/1.05).(A_{s,prov}')=(f_y/1.$	N/A	mm	N/A
s = 0.9x for $f_{cu} \le 60$ N/mm²;	N/A	mm	3.4.4.4 BC
0.8x for $60 < f_{cu} \le 75$ N/mm²	N/A	mm	3.4.4.4 BC
0.72x for $75 < f_{cu} \le 105$ N/mm²	N/A	mm	3.4.4.4 BC

Note for an under reinforced section, require $\varepsilon_{st} = \varepsilon_{cc}(d\text{-}x)/x \ge \varepsilon_y$ for yielding of tension steel
and $\varepsilon_{sc} = \varepsilon_{cc}(x\text{-}d')/x \ge \varepsilon_y$ for yielding of compression steel, where ε_{cc} =0.0035 for $f_{cu} \le$ 60N/mm
and ε_{cc} =0.0035 – (f_{cu} – 60)/50000 for f_{cu} > 60N/mm² and $\varepsilon_y = (f_y/1.05)/E_s$;

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Compression Stress Block in Web (s > h_f AND h_f <={0.45,0.36,0.30} **N/A**

Note simplified method as equations within this section only valid for β_b ≥ 0.90 and ≤ 1.10 as x = 0.5d;

h_f < 0.45d for f_{cu} ≤ 60 N/mm²; or	cl.3.4.4.4 BC
h_f < 0.36d for 60 < f_{cu} ≤ 75 N/mm²; or	cl.3.4.4.4 BC
h_f < 0.30d for 75 < f_{cu} ≤ 105 N/mm² and no moment redistribution.	cl.3.4.4.4 BC
	cl.3.4.4.4 BC

$M_{uf} = 0.45 f_{cu} (b - b_w) h_f (d - 0.5 h_f)$ — **N/A** kNm

$K_f = \dfrac{M - M_{uf}}{f_{cu} b_w d^2}$ — **N/A**

Ratio, K' — **N/A**

0.90 ≤ β_b ≤ 1.10	K' =	0.156 for f_{cu} ≤60 N/mm²	N/A mm	.3.4.4.4 BC
		0.120 for 60< f_{cu} ≤75 N/mm²	N/A mm	.3.4.4.4 BC
		0.094 for 75< f_{cu} ≤105 N/mm² [β_b =1.0]	N/A mm	.3.4.4.4 BC

If K_f < K' → no compression steel **N/A**

$A_s = \dfrac{M + k_1 f_{cu} b_w d (k_2 d - h_f)}{0.95 f_y (d - 0.5 h_f)}$, f_y≤460N/mm² — **N/A** mm² — Foreword

k_1 = 0.1 for f_{cu} ≤ 60 N/mm²,	k_2 = 0.45 for f_{cu} ≤60 N/mm²,	cl.3.4.4.5 BC
0.072 for 60 < f_{cu} ≤ 75 N/mm² and	0.32 for 60 < f_{cu} ≤75 N/mm² and	cl.3.4.4.5 BC
0.054 for 75< f_{cu} ≤ 105 N/mm²; and	0.24 for 75< f_{cu} ≤105N/mm²	cl.3.4.4.5 BC
		cl.3.4.4.5 BC

Back-analysis of x in (0.67f_{cu}/1.5).[b.h_f+(s-h_f).b_w]=(f_y/1.05).($A_{s,p}$ — **N/A** mm — **N/A**

s =	0.9x for f_{cu} ≤ 60 N/mm²;	x_{limit}=0.5d	N/A mm	.3.4.4.4 BC
	0.8x for 60 < f_{cu} ≤ 75 N/mm²		N/A mm	.3.4.4.4 BC
	0.72x for 75 < f_{cu} ≤ 105 N/mm²		N/A mm	.3.4.4.4 BC

Note for an under reinforced section, require x ≤ x_{limit} for yielding of tension steel;

If K_f > K' → compression steel required **N/A**

Compression steel, A_s' = [M-(K'$f_{cu}b_w d^2$+M_{uf})]/[0.95f_y(d-d')], f_y≤46	**N/A**	mm²	Foreword
Tension steel, A_s = [{0.20,0.18,0.16}$f_{cu}b_w$d+0.45$f_{cu}h_f$(b-b_w)]/(0.9	**N/A**	mm²	Foreword

Note the coefficient 0.20, 0.18 or 0.16 is used for f_{cu} ≤ 60, 75 or 105N/mm² respectively;

If $d' > \left(1 - \dfrac{f_y}{735}\right) x$, use $700\left(1 - \dfrac{d'}{x}\right)$ in lieu of $0.95 f_y$ — x=0.5d , f_y≤46 — N/A — Foreword

Back-analysis of x in (0.67f_{cu}/1.5).[b.h_f+(s-h_f).b_w]+(f_y/1.05).($A_{s,p}$ — **N/A** mm — **N/A**

s =	0.9x for f_{cu} ≤ 60 N/mm²;	N/A mm	.3.4.4.4 BC
	0.8x for 60 < f_{cu} ≤ 75 N/mm²	N/A mm	.3.4.4.4 BC
	0.72x for 75 < f_{cu} ≤ 105 N/mm²	N/A mm	.3.4.4.4 BC

Note for an under reinforced section, require $\varepsilon_{st} = \varepsilon_{cc}$(d-x)/x ≥ ε_y for yielding of tension steel

and $\varepsilon_{sc} = \varepsilon_{cc}$(x-d')/x ≥ ε_y for yielding of compression steel, where ε_{cc}=0.0035 for f_{cu} ≤ 60N/mm

and ε_{cc}=0.0035−(f_{cu}−60)/50000 for f_{cu} > 60N/mm² and ε_y=(f_y/1.05)/E_s;

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Member Design - RC Beam		Made by **XX**	Date **16/1/2024**	Chd.

				BS8110
Compression Stress Block in Web (s > h_f AND h_f > {0.45,0.36,0.30}d	**N/A**			
Note complex method as equations within this section valid for β_b ≥ 0.70 and ≤ 1.30;				
h_f < 0.45d for f_{cu} ≤ 60 N/mm²; or				*cl.3.4.4.5 BC*
h_f < 0.36d for 60 < f_{cu} ≤ 75 N/mm²; or				*cl.3.4.4.5 BC*
h_f < 0.30d for 75 < f_{cu} ≤ 105 N/mm² and no moment redistribution.				*cl.3.4.4.5 BC*
				cl.3.4.4.5 BC
Tension steel area provided	N/A	mm^2		
Tension steel area provided utilisation	**N/A**			**N/A**
Compression steel area provided	N/A	mm^2		
Compression steel area provided utilisation	**N/A**			**N/A**
% Min tension reinforcement	N/A	%		*TR49*
% Min tension reinforcement (>= 0.0032b_wh G250; >= MAX (0.0018, 0.0018(f_{cu}/40)$^{2/3}$).b_wh G				*cl.3.1.7*
% Min tension reinforcement utilisation	**N/A**			**N/A**
% Min compression reinforcement (>= 0.004bh_f flange; >= 0.002b_wh web)	N/A	%		
% Min compression reinforcement utilisation	**N/A**			**N/A**
% Max tension reinforcement (<= 0.04b_wh)	N/A	%		
% Max tension reinforcement utilisation	**N/A**			**N/A**
% Max compression reinforcement (<= 0.04bh_f flange; <= 0.04b_wh web)	N/A	%		
% Max compression reinforcement utilisation	**N/A**			**N/A**
% Max tension or compression reinforcement utilisation	**N/A**			**N/A**

BS8110

Shear Rectangular Beam (BS8110)

Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;

Note that d is the effective depth to the tension reinforcement irrespective of whether the section is sagging at midspans or hogging at supports. It follows then, for midspans, tension steel is the bottom steel whilst for supports, tension steel is the top steel;

2

Ultimate shear stress, $v_{ult} = V_d/b_wd$ (< $0.8f_{cu}^{0.5}$ & {5.0,7.0}N/mm²)		2.55	N/mm²	3.4.5.2 BC
Ultimate shear strength, MIN{$0.8f_{cu}^{0.5}$ & {5.0,7.0}N/mm²}		4.73	N/mm²	3.4.5.2 BC
Ultimate shear stress utilisation		54%		OK

Design shear stress, $v_d = V_d/b_wd$		2.55	N/mm²	
Enhanced shear strength, $2d/a_v.v_c$ (< $0.8f_{cu}^{0.5}$ & {5.0,7.0}N/mm²	x 1.00	0.79	N/mm²	cl.3.4.5.8
Distance, a_v	2.00d	1834	mm	

(Shear capacity enhancement by calculating v_d within 2d of support and comparing against enhanced v_c within 2d of the support as clause 3.4.5.8 BS8110 employed instead of calculating v_d at d from support and comparing against unenhanced v_c as clause 3.4.5.10 BS8110;) **Note**

Area of tension steel reinforcement provided, $A_{s,prov}$	6434	mm²	
$\rho_w = 100A_{s,prov}/b_wd$	1.40	%	
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$	0.79	N/mm²	cl.3.4.5.4
$\rho_w = 100A_{s,prov}/b_wd \le 3$	1.40	%	cl.3.4.5.4
$f_{cu} = f_{cu} \le 80N/mm^2$	35	N/mm²	3.4.5.4 BC
$(400/d)^{1/4} \ge (0.67 \text{ or } 1.00)$	1.00		cl.3.4.5.4

Table 3.8 — Values of v_c design concrete shear stress

$\frac{100A_s}{b_vd}$	Effective depth mm							
	125	150	175	200	225	250	300	400
	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²
≤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

NOTE 1 Allowance has been made in these figures for a γ_m of 1.25.

NOTE 2 The values in the table are derived from the expression:

$0.79\{100A_s/(b_vd)\}^{1/3}(400/d)^{1/4}/\gamma_m$

where

$\frac{100A_s}{b_vd}$ should not be taken as greater than 3;

$\left(\frac{400}{d}\right)^{\frac{1}{4}}$ should not be taken as less than 0.67 for members without shear reinforcement;

$\left(\frac{400}{d}\right)^{\frac{1}{4}}$ should not be taken as less than 1 for members with shear reinforcement providing a design shear resistance of ≥0.4 N/mm².

For characteristic concrete strengths greater than 25 N/mm², the values in this table may be multiplied by $(f_{cu}/25)^{1/3}$. The value of f_{cu} should not be taken as greater than 40.

Minimum shear strength, v_r = MAX (0.4, $0.4(f_{cu}/40)^{2/3}$), $f_{cu}\le80N/mm^2$		0.40	N/mm²	3.4.5.3 BC
Check $v_d < 0.5.2d/a_v.v_c$ for no links (minor elements)		INVALID	0.40	cl.3.4.5.3
Concrete shear capacity $2d/a_v.v_c.(b_wd)$		363	kN	4.5.4, cl.3.4
Check 0.0 < v_d < v_r + $2d/a_v.v_c$ for nominal links	0.00	N/A	1.19	cl.3.4.5.3
$(A_{sv}/S)_{nom} > v_r.b_w/(0.95f_{yv})$, $f_{yv}\le460N/mm^2$ i.e. $(A_{sv}/S)_{nom}$ >		0.46	mm²/mm	4.5.3, cl.3.4
$V_{cap,nom} = (v_r+2d/a_v.v_c).(b_wd)$		546	kN	4.5.3, cl.3.4
Check $v_d > v_r + 2d/a_v.v_c$ for design links	1.19	VALID	4.73	cl.3.4.5.3
$A_{sv}/S > b_w(v_d-2d/a_v.v_c)/(0.95f_{yv})$, $f_{yv}\le460N/mm^2$ i.e. A_{sv}/S >		2.01	mm²/mm	cl.3.4.5.8,
$V_{cap} = (A_{sv,prov}/S+A_{sv,prov,t}/S_t).(0.95f_{yv}).d + 2d/a_v.v_c.(b_wd)$, $f_{yv}\le46($		1622	kN	cl.3.4.5.8,

Area prov. by all shear and torsion links in section, $A_{sv,prov}+A_{sv,prov,t}$	314	mm²	
Tried $A_{sv,prov}/S + A_{sv,prov,t}/S_t$ value	3.14	mm²/mm	
Design shear resistance utilisation	72%		OK

Note while minimum links should be provided in all beams of structural importance, it may be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than half $2d/a_v.v_c$;

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EC2

Shear Rectangular Beam (EC2)

Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;

Note that d is the effective depth to the tension reinforcement irrespective of whether the section is sagging at midspans or hogging at supports. It follows then, for midspans, tension steel is the bottom steel whilst for supports, tension steel is the top steel;

Ultimate shear stress, $v_{ult} = V_d/b_wd$ (< $v_{Rd,max(\theta=45°)}$=0.18(1-f_{ck}/250).f_{ck}, f_{ck}≤5		2.55	N/mm^2	3(3), cl.3.
Ultimate shear strength, $v_{Rd,max(\theta=45°)}$=0.18(1-f_{ck}/250).f_{ck}, f_{ck}≤50N/mm^2		4.48	N/mm^2	3(3), cl.3.
Ultimate shear stress utilisation		57%		OK
Design shear stress, $v_d = V_d/b_wd$		2.55	N/mm^2	
Enhanced shear strength, 2d/a_v.$v_{Rd,c}$ (< $v_{Rd,max(\theta=45°)}$=0.18(1-f_{ck}/2	x 1.00	0.60	N/mm^2	cl.6.2.3(8)
Distance, a_v	2.00d	1834	mm	

(Shear capacity enhancement by calculating v_d within 2d of support and comparing against enhanced $v_{Rd,c}$ within 2d of the support as clause 6.2.2(6) EC2 employed instead of calculating v_d at d from support and comparing against unenhanced $v_{Rd,c}$;) **Note**

	Area of tension steel reinforcement provided, $A_{s,prov}$	6434	mm^2	
	ρ_w = 100$A_{s,prov}$/b_wd	1.40	%	
	$v_{Rd,c}$ = $C_{Rd,c}$.k($\rho_w f_{ck}$)$^{1/3}$+$k_1.\sigma_{cp}$ ≥ v_{min}+$k_1.\sigma_{cp}$; v_{min}=0.035$k^{3/2}f_{ck}^{0.5}$, k	0.60	N/mm^2	2(1), cl.3.
	$C_{Rd,c}$ = 0.18/γ_c = 0.18/1.5 = 0.12	0.12		cl.6.2.2(1)
2	k = 1 + √(200/d) ≤ 2.0	1.47		cl.6.2.2(1)
	ρ_w = 100$A_{s,prov}$/b_wd ≤ 2	1.40	%	cl.6.2.2(1)
	σ_{cp} = F/b_wh ≤ 0.2f_{cd}=0.2α_{cc}.f_{ck}/γ_c=0.2(1.0.f_{ck}/1.5), f_{ck}≤	0.00	N/mm^2	2(1), cl.3.

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

$\rho_l = A_s/bd$	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

Minimum shear strength, v_r = 0.08/γ_s.$f_{ck}^{0.5}$, γ_s=1.15, f_{ck}≤50N/mm^2		0.37	N/mm^2	2(5), cl.3.
Check v_d < 2d/a_v.$v_{Rd,c}$ for no links (minor elements)		INVALID	0.60	cl.6.2.1(3)
.5.8 Concrete shear capacity 2d/a_v.$v_{Rd,c}$.(b_wd)		274	kN	2(1), cl.6.
Check 0.0 < v_d < v_r for nominal links	0.00	N/A	0.37	cl.6.2.1(5)
.5.1 (A_{sv}/S)$_{nom}$ > v_r.b_w/(0.87f_{yv}), f_{yv}≤600N/mm^2 i.e. (A_{sv}/S)$_{nom}$ >		0.42	mm^2/mm	3(3), cl.3.
.5.8 $V_{cap,nom}$ = 2d/a_v.(A_{sv}/S)$_{nom}$.(0.87f_{yv}).0.9d.cotθ, f_{yv}≤600N/mm^2		376	kN	cl.6.2.3(8)
Check v_d > v_r for design links	0.37	VALID	4.48	cl.6.2.1(5)
cl.3.4.5.1 A_{sv}/S > b_w.v_d/(0.9(0.87f_{yv}).cotθ)/(2d/a_v), f_{yv}≤600N/mm^2 i.e. A_{sv}/S		1.32	mm^2/mm	cl.6.2.3(8)
cl.3.4.5.1 V_{cap} = 2d/a_v.($A_{sv,prov}$/S+$A_{sv,prov,t}$/S_t).(0.87f_{yv}).0.9d.cotθ, f_{yv}≤600N/		2792	kN	cl.6.2.3(8)
Comp. strut angle, θ = 22° ≤ 0.5$\sin^{-1}${v_d/$v_{Rd,max(\theta=45°)}$} ≤ 45°		22.0	°	cl.6.2.3(2)
Area prov. by all shear and torsion links in section, $A_{sv,prov}$+$A_{sv,prov,t}$		314	mm^2	
Tried $A_{sv,prov}$/S + $A_{sv,prov,t}$/S_t value		3.14	mm^2/mm	
Design shear resistance utilisation		42%		OK

Note while minimum links should be provided in all beams of structural importance, it may be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than 2d/a_v.$v_{Rd,c}$;

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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

ACI318

Shear Rectangular Beam (ACI318)

Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;

Note that d is the effective depth to the tension reinforcement irrespective of whether the section is sagging at midspans or hogging at supports. It follows then, for midspans, tension steel is the bottom steel whilst for supports, tension steel is the top steel;

	Ultimate shear stress, $v_{ult} = V_d^*/b_w d$ (< $\phi(0.66+0.17\lambda)\sqrt{f_c'}$, $f_c' \leq 70N/mm^2$)		2.55	N/mm^2	cl.22.5.5.1
	Ultimate shear strength, $\phi(0.66+0.17\lambda)\sqrt{f_c'}$, $f_c' \leq 70N/mm^2$		3.29	N/mm^2	cl.22.5.5.1
	Ultimate shear stress utilisation		77%		OK
	Design shear stress, $v_d = V_d^*/b_w d$		2.55	N/mm^2	
	Enhanced shear strength, $d/a_v.v_c$	*x 1.00*	0.67	N/mm^2	cl.9.4.3.2
	Distance, a_v	2.00d	1834	mm	
	(Shear capacity enhancement by calculating v_d within d of support and comparing against enhanced v_c within d of the support employed instead of calculating v_d at d from support and comparing against unenhanced v_c as clause 9.4.3.2 ACI318);				Note
	Area of tension steel reinforcement provided, $A_{s,prov}$		6434	mm^2	
	$\rho_w = 100A_{s,prov}/b_w d$		1.40	%	
.2(2)	$v_c = \phi V_c/b_w d$	*Note $f_c' \leq 70N/mm^2$*; Simplified	0.67	N/mm^2	
	$V_c = 0.17\lambda\sqrt{f_c'}b_w d$	*[Simplified] ϕV_c =*	309	kN	5.5.1, cl.22
.2(2)	where λ = {1.00 NWC, 0.75 LWC}	Normal weight			
		tailed MIN(a-c)] ϕV_c =	339	kN	5.5.1, cl.22
		Note $f_c' \leq 70N/mm^2$			
		[Detailed (a)] ϕV_c =	339	kN	5.5.1, cl.22
		Note $f_c' \leq 70N/mm^2$			
		[Detailed (b)] ϕV_c =	373	kN	5.5.1, cl.22
		Note $f_c' \leq 70N/mm^2$			
		[Detailed (c)] ϕV_c =	528	kN	5.5.1, cl.22
		Note $f_c' \leq 70N/mm^2$			
	where λ = {1.00 NWC, 0.75 LWC}	Normal weight			

Table 22.5.5.1—Detailed method for calculating V_c

	V_c	
Least of (a), (b), and (c):	$\left(0.16\lambda\sqrt{f_c'}+17\rho_w\frac{V_u d}{M_u}\right)b_w d$ [1]	(a)
	$(0.16\lambda\sqrt{f_c'}+17\rho_w)b_w d$	(b)
	$0.29\lambda\sqrt{f_c'}b_w d$	(c)

[1] M_u occurs simultaneously with V_u at the section considered.

	Strength reduction factor for shear, ϕ = 0.75		0.75		cl.21.2
	Minimum shear strength, v_r = MAX (0.35, $0.062\sqrt{f_c'}$), $f_c' \leq 70N/mm^2$		0.35	N/mm^2	5.2.2, cl.22
	Check v_d < 0.5.d/a_v.v_c for no links (minor elements)		INVALID	0.34	cl.9.6.3.1
.2(6)	Concrete shear capacity $d/a_v.v_c.(b_w d)$		309	kN	5.5.1, cl.9.
	Check 0.0 < v_d < ϕv_r + d/a_v.v_c for nominal links	0.00	N/A	0.94	cl.9.6.3.1
.2(3)	$(A_{sv}/S)_{nom} > v_r.b_w/f_{yv}$, $f_{yv} \leq 420N/mm^2$ i.e. $(A_{sv}/S)_{nom}$ >		0.42	mm^2/mm	5.10.5, cl.2
cl.3.2.2(3	$V_{cap,nom} = (\phi d/a_v.v_r + d/a_v.v_c).(b_w d)$		430	kN	.10.5.3, cl.
	Check v_d > ϕv_r + d/a_v.v_c for design links	0.94	VALID	3.29	l.22.5.10.1
cl.3.2.2(3	$A_{sv}/S > b_w(v_d - d/a_v.v_c)/(\phi f_{yv})/(d/a_v)$, $f_{yv} \leq 420N/mm^2$ i.e. A_{sv}/S >		2.98	mm^2/mm	, cl.9.4.3.2
cl.3.2.2(3	$V_{cap} = \phi d/a_v.(A_{sv,prov}/S + A_{sv,prov,t}/S_t).f_{yv}.d + d/a_v.v_c.(b_w d)$, $f_{yv} \leq 420N$		1217	kN	.10.5.3, cl.
	Area prov. by all shear and torsion links in section, $A_{sv,prov}+A_{sv,prov,t}$		314	mm^2	
	Tried $A_{sv,prov}/S + A_{sv,prov,t}/S_t$ value		3.14	mm^2/mm	
	Design shear resistance utilisation		96%		OK

Note while minimum links should be provided in all beams of structural importance, it may be satisfactory to omit them in members of minor structural importance such as lintels or where the maximum design shear stress is less than half $d/a_v.v_c$;

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		Member/Location		
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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

			<u>*BS8110-2*</u>
Torsion Rectangular Beam (BS8110)			
Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;			
Larger dimension of rectangular section, h_{max} = MAX (h, b_w)	1000	mm	
Smaller dimension of rectangular section, h_{min} = MIN (h, b_w)	500	mm	
Design torsion moment, T	**0**	kNm	
Design torsion stress, $v_t = 2T/((h_{min}{}^2)(h_{max}-h_{min}/3))$ (< $0.8f_{cu}{}^{0.5}$ & {5.0,7.0}N	**0.00**	N/mm^2	*cl.2.4.4.1*
Ultimate torsion strength, MIN{$0.8f_{cu}{}^{0.5}$ & {5.0,7.0}N/mm^2}	**4.73**	N/mm^2	*.3.4.5.2 BC*
Design ultimate torsion stress utilisation	**0%**		**OK**
Check v_t < $v_{t.min}$ for no torsion links	**VALID**		*cl.2.4.6*
Concrete torsion resistance, $v_{t.min}$ = MIN (0.6, $0.067\sqrt{f_{cu}}$), f_{cu} ≤ 80	**0.40**	N/mm^2	*cl.2.3 BC2*
Check v_t > $v_{t.min}$ for design torsion links	**N/A**		*cl.2.4.6*
Horizontal dimension of closed link, $x_1 = b_w$ - 2 x cover - $\phi_{link,t}$	450	mm	
Vertical dimension of closed link, y_1 = h - 2 x cover - $\phi_{link,t}$	950	mm	
Provide torsion links $A_{sv,t}/S_t > T/(0.8x_1y_1(0.95f_{yv}))$, f_{yv}≤460N/mm	**0.00**	mm^2/mm	*.4.7, Forew*
Provide torsion longitudinal bars, $A_{s,t}$	**0**	mm^2	*.4.7, Forew*
Note require $A_{s,t} > (A_{sv,t}/S_t)(f_{yv}/f_y)(x_1+y_1)$, f_y ≤460N/mm^2, f_{yv} ≤460N/mm^2; cl.2.4.7, Forew			
Area provided by all torsion links in a cross-section, $A_{sv,prov,t}$	0	mm^2	
Tried $A_{sv,prov,t}/S_t$ value	0.00	mm^2/mm	
Design torsion resistance utilisation	**0%**		**OK**
Combined Shear and Torsion Rectangular Beam (BS8110)			
Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;			
.5.3.1			
Ultimate shear and torsion stresses $v_{ult} + v_t$ (< $0.8f_{cu}{}^{0.5}$ & {5.0,7.0}N/mm^2)	**2.55**	N/mm^2	*cl.2.4.5*
Ultimate shear and torsion strength, MIN{$0.8f_{cu}{}^{0.5}$ & {5.0,7.0}N/mm^2}	**4.73**	N/mm^2	*.3.4.5.2 BC*
Ultimate shear and torsion stresses utilisation	**54%**		**OK**
.5.3.1			
Check design shear and torsion links			
Provide single torsion link leg > $A_{sv}/S/(n_{leg}+n_{leg,t}) + A_{sv,t}/S_t/n_{leg,t}$	**0.34**	mm^2/mm	
Area provided by single torsion link leg, $A_{sv,prov,t}/n_{leg,t}$	0	mm^2	
Tried $A_{sv,prov,t}/n_{leg,t}/S_t$ value	0.00	mm^2/mm	
Design shear and torsion resistance utilisation	**0%**		**OK**
4.3.2			
2.5.3.3			
9.4.3.2, cl.22.5.3.3			
, cl.22.5.3.3			
9.4.3.2, cl.22.5.3.3			

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		Member/Location	
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.	
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

			EC2
Torsion Rectangular Beam (EC2)			
Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;			
Larger dimension of rectangular section, h_{max} = MAX (h, b_w)	1000	mm	
Smaller dimension of rectangular section, h_{min} = MIN (h, b_w)	500	mm	
Design torsion moment, T_d	**0**	kNm	
Design torsion stress, $v_t = T_d/(t_{ef}.2A_k)$	**0.00**	N/mm^2	*MOSLEY*
Ultimate torsion moment capacity, $T_{Rd,max(\theta=45°)}=1.33\nu_1.f_{ck}.t_{ef}.A_k/(\cot\theta+\tan\theta)$,	**459**	kNm	*2(4), cl.3.*
Design ultimate torsion moment capacity utilisation	**0%**		**OK**
Check T_d < $T_{Rd,c}$ for no torsion links	**VALID**		
Concrete torsion resistance, $T_{Rd,c} = f_{ctd}.t_{ef}.2A_k$, f_{ck}≤50N/mm^2	**120**	kNm	
where $f_{ctd} = \alpha_{ct}.f_{ctk}/\gamma_c$, α_{ct}=1.0, γ_c=1.5, f_{ctk}=0.7f_{ctm}, f_{ctm}=0.30$f_{ck}^{2/3}$			*cl.3.1.6*
Check T_d > $T_{Rd,c}$ for design torsion links	**N/A**		
Effective wall thickness, $t_{ef} = (b_w.h)/[2.(b_w+h)]$	167	mm	*cl.6.3.2(1)*
Area enclosed, $A_k = (b_w-t_{ef}).(h-t_{ef})$	277778	mm^2	*cl.6.3.2(1)*
ord Provide torsion links $A_{sv,t}/S_t > T_d/(A_k.(0.87f_{yv}).\cot\theta)$, f_{yv}≤600N/mr	**0.00**	mm^2/mm	*MOSLEY*
ord Provide torsion longitudinal bars, $A_{s,t}$	**0**	mm^2	*cl.6.3.2(3)*
ord *Note require $A_{s,t} > T_d.u_k/(2.A_k).\cot\theta/(0.87f_y)$, $u_k=2.(b_w+h-2t_{ef})$, f_y≤600N/mm^2;*			*cl.6.3.2(3)*
Area provided by all torsion links in a cross-section, $A_{sv,prov,t}$	0	mm^2	
Tried $A_{sv,prov,t}/S_t$ value	0.00	mm^2/mm	
Design torsion resistance utilisation	**0%**		**OK**
Combined Shear and Torsion Rectangular Beam (EC2)			
Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;			
Ultimate shear and torsion $V_d/V_{Rd,max(\theta=45°)} + T_d/T_{Rd,max(\theta=45°)}$ (≤ 1.00)	**0.57**		*cl.6.3.2(4)*
Ultimate shear and torsion utilisation, ≤ 1.00	**1.00**		*cl.6.3.2(4)*
Ultimate shear and torsion utilisation	**57%**		**OK**
Check design shear and torsion links			
Provide single torsion link leg > $A_{sv}/S/(n_{leg}+n_{leg,t}) + A_{sv,t}/S_t/n_{leg,t}$	**0.22**	mm^2/mm	
Area provided by single torsion link leg, $A_{sv,prov,t}/n_{leg,t}$	0	mm^2	
Tried $A_{sv,prov,t}/n_{leg,t}/S_t$ value	0.00	mm^2/mm	
Design shear and torsion resistance utilisation	**0%**		**OK**

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		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

			ACI318
Torsion Rectangular Beam (ACI318)			
Note that this design check is performed for both rectangular and flanged sections, adopting			
rectangular section equations in the case of flanged sections;			
Larger dimension of rectangular section, h_{max} = MAX (h, b_w)	1000	mm	
Smaller dimension of rectangular section, h_{min} = MIN (h, b_w)	500	mm	
Design torsion moment, T*	0	kNm	
Design torsion stress, v_t = T*.2.(x_1+y_1)/1.7/(x_1y_1)^2 (< ϕ(0.66+0.17λ)√f_c', ϕ=	0.00	N/mm^2	cl.22.7.7.1
Ultimate torsion strength, ϕ(0.66+0.17λ)√f_c', ϕ=0.75, f_c'≤70N/mm^2	3.29	N/mm^2	cl.22.5.5.1
Design ultimate torsion stress utilisation	0%		OK
Check T < ϕT_{th} for no torsion links	VALID		
Concrete torsion resistance, ϕT_{th} = ϕ0.083λ√f_c'.(h_{min}.h_{max})^2/[2.(h_{mi}	27	kNm	7.4.1, cl.22
where λ = {1.00 NWC, 0.75 LWC} — Normal weight			
Check T > ϕT_{th} for design torsion links	N/A		
Horizontal dimension of closed link, x_1 = b_w - 2 x cover - $\phi_{link,t}$	450	mm	
Vertical dimension of closed link, y_1 = h - 2 x cover - $\phi_{link,t}$	950	mm	
Provide torsion links $A_{sv,t}/S_t$ > T*/(ϕ0.85x_1y_1.f_{yv}), ϕ=0.75, θ=40°,	0.00	mm^2/mm	cl.22.7.6.1
Provide torsion longitudinal bars, $A_{s,t}$ — 0	0	mm^2	7.6.1, cl.9.
Note require $A_{s,t}$ > ($A_{sv,t}/S_t$)(f_{yv}/f_y)(x_1+y_1) > $A_{l,min}$, f_y ≤420N/mm^2, f_{yv} ≤420N/mm			*cl.22.7.6.1, cl.9.*
Area provided by all torsion links in a cross-section, $A_{sv,prov,t}$	0	mm^2	
Tried $A_{sv,prov,t}/S_t$ value	0.00	mm^2/mm	
Design torsion resistance utilisation	0%		OK
Combined Shear and Torsion Rectangular Beam (ACI318)			
Note that this design check is performed for both rectangular and flanged sections, adopting			
rectangular section equations in the case of flanged sections;			
Ultimate shear and torsion stresses [v_{ult}^2 + v_t^2]$^{1/2}$ (< ϕ(0.66+0.17λ)√f_c', ϕ=0	2.55	N/mm^2	cl.22.7.7.1
Ultimate shear and torsion strength, ϕ(0.66+0.17λ)√f_c', ϕ=0.75, f_c'≤70N/mm	3.29	N/mm^2	5.5.1, cl.22
Ultimate shear and torsion stresses utilisation	77%		OK
Check design shear and torsion links			
Provide single torsion link leg > A_{sv}/S/(n_{leg}+$n_{leg,t}$) + $A_{sv,t}/S_t/n_{leg,t}$	0.50	mm^2/mm	
Provide single torsion link leg > MAX(0.175,0.031√f_c').b_w/f_{yv}, f_{yv}≤	0.21	mm^2/mm	cl.9.6.4.2
Area provided by single torsion link leg, $A_{sv,prov,t}/n_{leg,t}$	0	mm^2	
Tried $A_{sv,prov,t}/n_{leg,t}/S_t$ value	0.00	mm^2/mm	
Design shear and torsion resistance utilisation	0%		OK

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		jXXX	19	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

BS8110

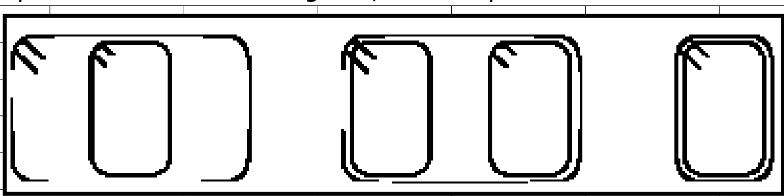
Detailing Requirements Rectangular or Flanged Beam

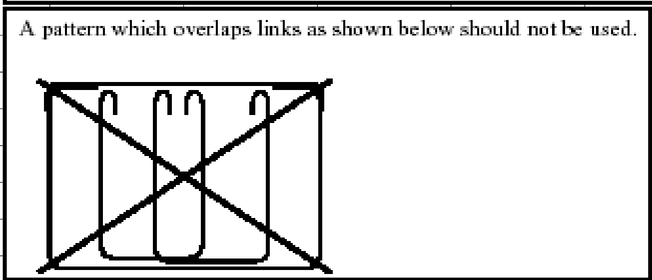
All detailing requirements met ?	OK		
Min tension steel reinforcement diameter, ϕ_t (>=12mm)	32	mm	OK
Min tension steel reinforcement pitch (b_w–2.cover–2.MAX(ϕ_{link},$\phi_{link,t}$)–ϕ_t)/(n_t/n	133	mm	OK
Max tension steel reinforcement pitch (b_w–2.cover–2.MAX(ϕ_{link},$\phi_{link,t}$)–ϕ_t)/(n_t/r	133	mm	OK

Note that max pitch assumes no moment redistribution in beam, if less than 20% moment redistribution then pitch to be less than 175mm for G250 and less than 150mm for G460;

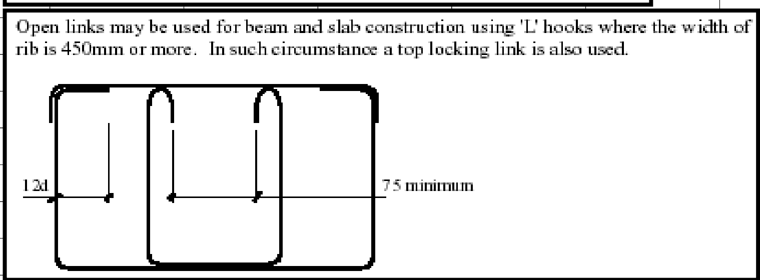
Min compression steel reinforcement diameter, ϕ_c (>=12mm)	N/A	mm	N/A
Min compression steel reinforcement pitch (b_w or b–2.cover–2.MAX(ϕ_{link},$\phi_{link,t}$	N/A	mm	N/A
Max compression steel reinforcement pitch (b_w or b–2.cover–2.MAX(ϕ_{link},$\phi_{link,t}$	N/A	mm	N/A
Min shear link diameter, ϕ_{link} (>=6mm, >=ϕ_c/4)	10	mm	OK
Min torsion link diameter, $\phi_{link,t}$ (>=6mm, >=ϕ_c/4)	N/A	mm	N/A
Shear link pitch, S (<=0.75d, <=12ϕ_c, <=300mm, >=MAX(100mm, 50+12.	100	mm	OK
Torsion link pitch, S_t (<=0.75d, <=12ϕ_c, <=300mm, >=MAX(100mm, 50+1	N/A	mm	N/A
$A_{sv,prov}$ / (b_w.S) + $A_{sv,prov,t}$ / (b_w.S_t) (>0.10% G460; >0.17% G250)	0.63	%	OK
$A_{sv,prov}$ / (b_w.S) + $A_{sv,prov,t}$ / (b_w.S_t) (<4.00%)	0.63	%	OK

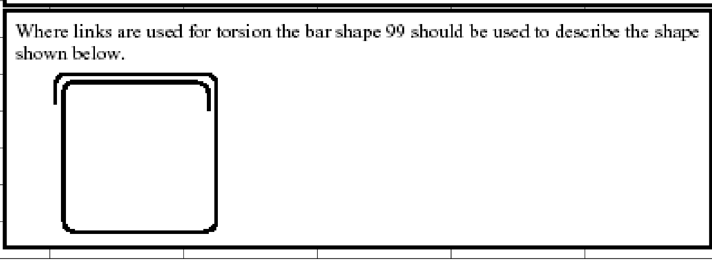
Note require an overall enclosing link; Note require additional restraining links for each alternate longitudinal





.5.3.1,





Note lacer bars of 16mm are required at the sides of beams more than 750mm deep at 250mm pitch;

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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

BS8110

Deflection Criteria Rectangular or Flanged Beam

Typically require :	Total deflection	< span/250
	Live Load + creep	< span/350
	and	< 20mm

Criterion satisfied if span / effective depth < (Basic x C_1 x C_2 x C_3)

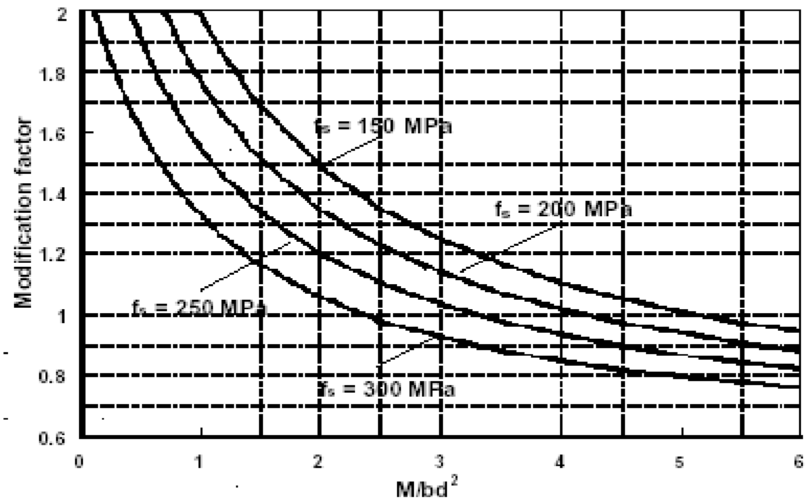
Typical multiplers (C1):
C1 = 0.8 for flanged beams with $b_w/b < 0.3$
C1 = 10/span(m) for spans beyond 10m
C1 = 0.9 for flat slabs (use longer span)

NOTE: For two-way slabs on continuous support, use shorter span.

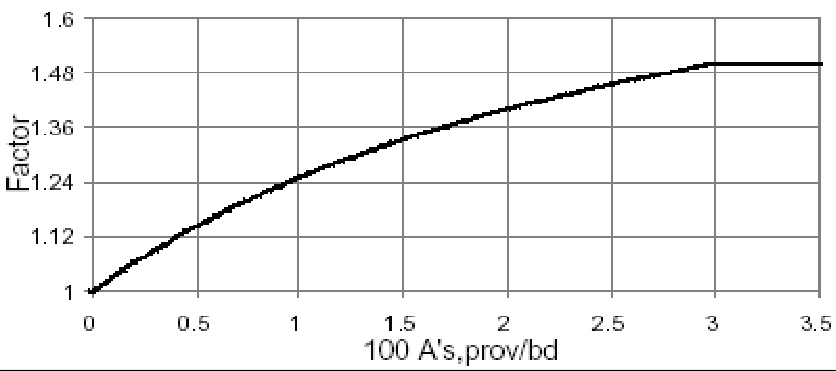
Basic span/effective depth ratios for rectangular beams	
Support conditions	Rectangular sections
Cantilever	7
Simple supported	20
Continuous	26

Tension reinforcement modification factor (C_2)[4]

f_s = service stress in reinforcement



Compression reinforcement modification factor (C_3)



Shrinkage

For normal situations, assume long term shrinkage strain of 300 x 10^{-6}

Creep

For normal situations, assume creep coefficient of $\phi = 2$

Hence long term E value: $E = \frac{E_{28}}{1+\phi}$

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		jXXX	21	
		Member/Location		
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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

				BS8110
Span		10.000	m	
Span / effective depth ratio		**10.9**		
Basic span / effective depth ratio criteria		**26.0**		
Multiplier $C_{1,\text{rect or flanged}}$		1.00		*cl.3.4.6.3*
Multiplier $C_{1,\text{span more or less than 10m}}$	Include	1.00		*cl.3.4.6.4*
Modification factor for tension C_2				
$M/(b_w \text{ or } b).d^2$		4.40	N/mm^2	*cl.3.4.6.2*
$f_s = \frac{2 f_y A_{s\,req}}{3 A_{s\,prov}} \times \frac{1}{\beta_b}$		287	N/mm^2	
Modification	$0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \leq 2.0$	**0.85**		*T.3.10*
Modification factor for compression C_3				
$100A_{s,prov}'/(b_w \text{ or } b).d$		N/A		*cl.3.4.6.2*
Modification	$1 + \frac{100A'_{s\,prov}}{bd} / \left(3 + \frac{100A'_{s\,prov}}{bd}\right) \leq 1.5$	**1.00**		*T.3.11*
Modified span / effective depth ratio criteria		**22.1**		
Deflection utilisation		**49%**		**OK**

Table 3.10 — Modification factor for tension reinforcement

Service stress		M/bd^2								
		0.50	0.75	1.00	1.50	2.00	3.00	4.00	5.00	6.00
	100	2.00	2.00	2.00	1.86	1.63	1.36	1.19	1.08	1.01
	150	2.00	2.00	1.98	1.69	1.49	1.25	1.11	1.01	0.94
(f_y = 250)	167	2.00	2.00	1.91	1.63	1.44	1.21	1.08	0.99	0.92
	200	2.00	1.95	1.76	1.51	1.35	1.14	1.02	0.94	0.88
	250	1.90	1.70	1.55	1.34	1.20	1.04	0.94	0.87	0.82
	300	1.60	1.44	1.33	1.16	1.06	0.93	0.85	0.80	0.76
(f_y = 460)	307	1.56	1.41	1.30	1.14	1.04	0.91	0.84	0.79	0.76

NOTE 1 The values in the table derive from the equation:

$$\text{Modification factor} = 0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \leq 2.0 \qquad \text{equation 7}$$

where

M is the design ultimate moment at the centre of the span or, for a cantilever, at the support.

NOTE 2 The design service stress in the tension reinforcement in a member may be estimated from the equation:

$$f_s = \frac{2 f_y A_{s\,req}}{3 A_{s\,prov}} \times \frac{1}{\beta_b} \qquad \text{equation 8}$$

NOTE 3 For a continuous beam, if the percentage of redistribution is not known but the design ultimate moment at mid-span is obviously the same as or greater than the elastic ultimate moment, the stress f_s in this table may be taken as $2/3 f_y$.

Table 3.11 — Modification factor for compression reinforcement

$100 \frac{A'_{s\,prov}}{bd}$	Factor
0.00	1.00
0.15	1.05
0.25	1.08
0.35	1.10
0.50	1.14
0.75	1.20
1.0	1.25
1.5	1.33
2.0	1.40
2.5	1.45
≥ 3.0	1.50

NOTE 1 The values in this table are derived from the following equation:

$$\text{Modification factor for compression reinforcement} = 1 + \frac{100A'_{s\,prov}}{bd} / \left(3 + \frac{100A'_{s\,prov}}{bd}\right) \leq 1.5 \qquad \text{equation 9}$$

NOTE 2 The area of compression reinforcement A used in this table may include all bars in the compression zone, even those not effectively tied with links.

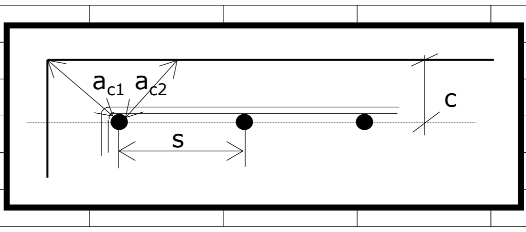
BS8110

Crack Width Estimation Rectangular Beam

Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections;

Maximum crack width criteria, Standard concrete BS8110 (0.30mm)	**0.30**	mm	
Load factor, LF = 1.4 conservatively	1.4		
SLS bending moment, M_{sls} = M / LF	1321	kNm	
SLS axial force (tension +ve and compression -ve), F_{sls} = -F / LF	0	kN	
Uncracked elastic modulus, $E_{uncracked} = 20+0.2f_{cu}$	27	GPa	
Cracked elastic modulus, $E_{cracked} = E_{uncracked} / 2$	14	GPa	
Steel elastic modulus, E_s	200	GPa	
Modulus ratio, $\alpha_e = E_s / E_{cracked}$	14.8		
Reinforcement ratio, $\rho_w = A_{s,prov}/b_wd$	0.01		
Factor, $\alpha_e.\rho_w$	0.21		
Depth of neutral axis, $x = \alpha_e.\rho_w.[(1+2/(\alpha_e\rho_w))^{1/2}-1].d$	431	mm	
Lever arm, z = d-x/3	773	mm	
Steel tensile service stress (flexural), $f_{s1} = M_{sls}/z / A_{s,prov}$	265	N/mm^2	
Steel tensile (tensile +ve and compressive -ve) service stress (axial), f_{s2}	0	N/mm^2	
Note if $F_{sls} > 0$, $f_{s2} = F_{sls}/(A_{s,prov}+A_{s,prov}')$,			
else $f_{s2} = \alpha_e.(A_{s,prov}+A_{s,prov}')/[\alpha_e.(A_{s,prov}+A_{s,prov}')+(b_w.h-A_{s,prov}-A_{s,prov}')].F_{sls}/(A_{s,prov}+A_{s,prov}')$;			
Steel tensile service stress utilisation, $(f_{s1}+f_{s2}) / (0.8f_y) < 1.00$	**66%**		**OK**
Concrete compressive service stress (flexural and axial), f_c	16	N/mm^2	
Note if $F_{sls} > 0$, $f_c = 2(M_{sls}/z)/(b_wx)$, else $f_c = 2(M_{sls}/z)/(b_wx)$ +			
$(b_w.h-A_{s,prov}-A_{s,prov}')/[\alpha_e.(A_{s,prov}+A_{s,prov}')+(b_w.h-A_{s,prov}-A_{s,prov}')].ABS(F_{sls})/(b_w.h-A_{s,prov}+A_{s,prov}')$;			
Concrete compressive service stress utilisation, $f_c / (0.45f_{cu}) < 1.00$	**101%**		**NOT OK**
Strain at tension face, $\varepsilon_1 = (h-x)/(d-x).(f_{s1}+f_{s2})/E_s$	1.55	$x10^{-3}$	
Strain stiffening, $\varepsilon_2 = b_w(h-x)^2/[3E_sA_{s,prov}(d-x)]$	0.09	$x10^{-3}$	
Mean strain, $\varepsilon_m = \varepsilon_1 - \varepsilon_2$	1.47	$x10^{-3}$	
Distance to face of extreme rebar, c_f = cover + MAX(ϕ_{link},$\phi_{link,t}$)	35	mm	
Distance to centroid of extreme rebar, $c = c_f + \phi_t/2$	51	mm	
Distance, $s = (b_w-2.cover-2.MAX(\phi_{link},\phi_{link,t})-\phi_t)/(n_t/n_{layers,tens}-1)$	133	mm	
Distance, $a_{c1} = [c^2+c^2]^{1/2} - \phi_t/2$	56	mm	
Distance, $a_{c2} = [c^2+(s/2)^2]^{1/2} - \phi_t/2$	68	mm	
Distance, $a_{cr} = MAX(a_{c1}, a_{c2})$	68	mm	

Note a_{c1} is not applicable for continuous width sections, i.e. slabs or walls;



Maximum crack width, $w_{max} = 3a_{cr}\varepsilon_m/[1+2.(a_{cr}-c_f)/(h-x)]$	**0.27**	mm	
Maximum crack width utilisation, $w_{max} / w_{allow} < 1.00$	**89%**		**OK**

Note for a particular section and force / moment, crack widths can be reduced by increasing steel area, reducing spacing between rebars and reducing concrete cover (limited to durability requirements). The employment of smaller diameter bars at closer centres is thus preferable to larger diameter bars at further centres. There should be a provision for longitudinal steels at the side faces of beams of moderate depths;

Crack width utilisation	**101%**	**NOT OK**

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		jXXX	23	
		Member/Location		
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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

			EC2
Longitudinal Shear Between Web and Flange Rectangular or Flanged Beam (EC2)			
Note that this check is performed for both rectangular and flanged section designs, although theoretically only applicable in the latter case;			
Longitudinal shear stress, K_S . v_{Ed}	**1.61**	N/mm^2	
Longitudinal shear stress, $v_{Ed} = \Delta F_d/(h_f \cdot \Delta x)$	1.21	N/mm^2	cl.6.2.4
Change of normal force in flange half over Δx, $\Delta F_d = K_B.(M^*-0)/z$	606	kN	
Note conservatively factor, $K_B = 0.5(b_{eff}-b_w)/b_{eff}$ employed even if neutral axis within web;			
Lever arm, z	0.763	m	
Note if neutral axis within web, for simplicity, $z = d - h_f/2$;			
Thickness of the flange at the junctions, h_f	200	mm	
Length under consideration, Δx	2500	mm	
Note the maximum value that may be assumed for Δx is half the distance between the section where the moment is 0 and the section where the moment is maximum. However, since ΔF_d is also calculated over Δx based on a variation of moment of ~ M/2 − 0 say, it is deemed acceptable to use for Δx the full distance between the section where the moment is 0 and the section where the moment is maximum based on a variation of moment of M − 0 and factored by K_S.			cl.6.2.4
Shear stress distribution factor, K_S	1.33		
For UDLs, K_S may be taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;			
Effective width, b_{eff} = MIN(b_w + function (span, section, structure	1000	mm	
Note for rectangular sections, b_{eff} equivalent to that of T-sections assumed;			
Width (rectangular) or web width (flanged), b_w	500	mm	
Longitudinal shear stress limit to prevent crushing, $\nu f_{cd} \sin\theta_f \cos\theta_f$	**4.97**	N/mm^2	cl.6.2.4
Design compressive strength, f_{cd}	19	N/mm^2	
$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C$ with α_{cc}=1.0, γ_c=1.5			cl.3.1.6
Strength reduction factor for concrete cracked in shear, ν	0.533		
$\nu = 0{,}6\left[1-\frac{f_{ck}}{250}\right]$			cl.6.2.2
Longitudinal shear stress limit to prevent crushing utilisation, $(K_S.v_{Ed})/(\nu f_{cd}$si	**32%**		**OK**
Longitudinal shear stress limit for no transverse reinforcement, $0.4f_{ctd}$	**0.52**	N/mm^2	cl.6.2.4
Design tensile strength, f_{ctd}	**1.29**	N/mm^2	
$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C$ with α_{ct}=1.0, γ_c=1.5			cl.3.1.6
$f_{ctk;0,05} = 0{,}7 \times f_{ctm}$	1.94	N/mm^2	T.3.1
$f_{ctm}=0{,}30\times f_{ck}^{(2/3)} \le$C50/60 | $f_{ctm}=2{,}12\cdot\ln(1+(f_{cm}/10))$ | > C50/60	2.77	N/mm^2	T.3.1
$f_{cm} = f_{ck}+8(MPa)$	36	N/mm^2	T.3.1
Characteristic cylinder strength of concrete, f_{ck}	28	N/mm^2	T.3.1
Characteristic cube strength of concrete, f_{cu}	35	N/mm^2	T.3.1
Longitudinal shear stress limit for no transverse reinforcement utilisation, (K	**312%**		**NOT OK**
Required design transverse reinforcement per unit length, A_{sf}/s_f >	**741**	mm^2/m	
$(A_{sf}f_{yd}/s_f) \ge v_{Ed} \cdot h_f / \cot\theta_f$			
Note area of transverse steel to be provided should be the greater of $1.0A_{sf}/s_f$ and $0.5A_{sf}/s_f$ + area required for slab bending; Note K_S factored onto v_{Ed} herein;			cl.6.2.4
Design yield strength of reinforcement, $f_{yd} = f_y/\gamma_S$, γ_S=1.15, f_y≤60	435	N/mm^2	.2.4, cl.3.2
Thickness of the flange at the junctions, h_f	200	mm	
Angle, θ_f	45.0	degrees	
$1{,}0 \le \cot\theta_f \le 2{,}0$ for compression flanges ($45° \ge \theta_f \ge 26{,}5°$) $1{,}0 \le \cot\theta_f \le 1{,}25$ for tension flanges ($45° \ge \theta_f \ge 38{,}6°$)			cl.6.2.4
Provided transverse reinforcement per unit length, A_e	785	mm^2/m	
Required design transverse reinforcement per unit length utilisation, (A_{sf}/s_f)/	**94%**		**OK**

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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

BS5400-4

Longitudinal Shear Between Web and Flange Rectangular or Flanged Beam (BS5400-4)

Note that this check is performed for both rectangular and flanged section designs, although theoretically only applicable in the latter case;

Longitudinal shear force per unit length, $V_1 = K_S \cdot \Delta F_d / \Delta x$	**322**	kN/m	
Change of normal force in flange half over Δx, $\Delta F_d = K_B.(M^*-0)/z$	606	kN	
Note conservatively factor, $K_B = 0.5(b_{eff} - b_w)/b_{eff}$ employed even if neutral axis within web;			
Lever arm, z	0.763	m	
Note if neutral axis within web, for simplicity, $z = d - h_f/2$;			
Thickness of the flange at the junctions, h_f	200	mm	
Length under consideration, Δx	2500	mm	
Note Δx is the beam length between the point of maximum design moment and the point of zero moment;			
Shear stress distribution factor, K_S	1.33		
The longitudinal shear should be calculated per unit length. For UDLs, K_S may be taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;			*cl.7.4.2.3*
Effective width, b_{eff} = MIN(b_w + function (span, section, structure	1000	mm	
Note for rectangular sections, b_{eff} equivalent to that of T-sections assumed;			
Width (rectangular) or web width (flanged), b_w	500	mm	
Longitudinal shear force limit per unit length, $V_{1,limit}$	**503**	kN/m	

V_1 should not exceed the lesser of the following:
a) $k_1 f_{cu} L_s$
b) $v_1 L_s + 0.7 A_e f_y$

(a)	1050	kN/m	*cl.7.4.2.3*
(b)	503	kN/m	*cl.7.4.2.3*

Table 31 — Ultimate longitudinal shear stress, v_1, and values of k_1 for composite members

Type of shear plane	Longitudinal shear stress for concrete grade				k_1
	20	25	30	40 or more	
	N/mm^2	N/mm^2	N/mm^2	N/mm^2	
Monolithic construction	0.90	0.90	1.25	1.25	0.15
Surface type 1	0.50	0.63	0.75	0.80	0.15
Surface type 2	0.30	0.38	0.45	0.50	0.09

NOTE For construction with lightweight aggregate concrete, the values given in this table should be reduced by 25 %.

Concrete bond constant, k_1	0.15		*T.31*
Ultimate longitudinal shear stress limit, v_1	1.25	N/mm^2	*T.31*
Surface type: Monolithic construction			*T.31*
Length of shear plane, $L_s = h_f$	200	mm	
Provided transverse reinforcement per unit length, A_e	785	mm^2/m	
Note reinforcement provided for coexistent bending effects and shear reinforcement crossing the shear plane, provided to resist vertical shear, may be included provided they are fully anchored;			*cl.7.4.2.3*
Characteristic strength of reinforcement, $f_y \leq 460$N/mm^2	460	N/mm^2	
Longitudinal shear force limit per unit length utilisation, $V_1/V_{1,limit}$	**64%**		**OK**
Required nominal transverse reinforcement per unit length, 0.15%L_s	**300**	mm^2/m	*cl.7.4.2.3*
Required nominal transverse reinforcement per unit length utilisation, 0.15%	**38%**		**OK**

CONSULTING ENGINEERS Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 25	Rev.
	Member/Location		
Job Title: Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam	Made by XX	Date 16/1/2024	Chd.

EC2

Longitudinal Shear Within Web Rectangular or Flanged Beam (EC2)

Longitudinal shear stress, $v_{Edi} = \beta V_{Ed} / (z b_i)$	**1.53**	N/mm^2	*cl.6.2.5*
Ratio, β = 1.0	1.0		*cl.6.2.5*
Transverse shear force (averaged), $V_{Ed} = V_d*/2$	585	kN	*cl.6.2.5*
Lever arm, z	0.763	m	*cl.6.2.5*
Note if neutral axis within web, for simplicity, $z = d - h_f/2$;			
Width of the interface, $b_i = b_w$	500	mm	*cl.6.2.5*
Longitudinal shear stress limit, v_{Rdi}	**3.10**	N/mm^2	
$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) \le 0,5\, \nu f_{cd}$			*cl.6.2.5*
Note $c.f_{ctd}$ = 0.00 if σ_n is negative (tension);			*cl.6.2.5*
Roughness coefficient, c — Indented	0.500		*cl.6.2.5*
Roughness coefficient, μ — Indented	0.9		*cl.6.2.5*

Very smooth: a surface cast against steel, plastic or specially prepared wooden moulds: c = 0,025 to 0,10 and μ = 0,5
Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: c = 0,20 and μ = 0,6
Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: c = 0,40 and μ = 0,7 (AC1)
Indented: a surface with indentations complying with Figure 6.9: c = 0,50 and μ = 0,9

Design tensile strength, f_{ctd}	**1.29**	N/mm^2	
$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C$ with α_{ct}=1.0, γ_c=1.5			*cl.3.1.6*
$f_{ctk;0,05} = 0,7 \times f_{ctm}$	1.94	N/mm^2	*T.3.1*
$f_{ctm} = 0,30 \times f_{ck}^{(2/3)} \le$ C50/60; $f_{ctm} = 2,12 \cdot \ln(1+(f_{cm}/10))$ > C50/60	2.77	N/mm^2	*T.3.1*
$f_{cm} = f_{ck}$+8(MPa)	36	N/mm^2	*T.3.1*
Characteristic cylinder strength of concrete, f_{ck}	28	N/mm^2	*T.3.1*
Characteristic cube strength of concrete, f_{cu}	35	N/mm^2	*T.3.1*
Normal stress across longitudinal shear interface, σ_n = 0	**0.00**	N/mm^2	
Reinforcement ratio, $\rho = A_s / A_i$	**0.006**		*cl.6.2.5*
Area of reinforcement, $A_s = A_{sv,prov}/S + A_{sv,prov,t}/S_t$	3142	mm^2/m	
Note that the area of reinforcement crossing the shear interface may include ordinary shear reinforcement with adequate anchorage at both sides of the interface;			*cl.6.2.5*
Area of the joint, A_i = 1000.b_i	500000	mm^2/m	
Design yield strength of reinforcement, $f_{yd} = f_{yv}/\gamma_S$, γ_S=1.15, $f_{yv} \le 6$	435	N/mm^2	.2.4, cl.3.2
Angle of reinforcement, α = 90.0°	90.0	degrees	*cl.6.2.5*
Design compressive strength, f_{cd}	19	N/mm^2	
$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C$ with α_{cc}=1.0, γ_c=1.5			*cl.3.1.6*
Strength reduction factor for concrete cracked in shear, ν	0.533		
$\nu = 0,6\left[1 - \frac{f_{ck}}{250}\right]$			*cl.6.2.2*
Longitudinal shear stress limit utilisation, v_{Edi}/v_{Rdi}	**49%**		**OK**

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		jXXX	26	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

BS8110

Longitudinal Shear Within Web Rectangular or Flanged Beam (BS8110)

Longitudinal shear stress, $v_h = K_S \cdot \Delta F_c / (b_w.\Delta x)$	**2.58**	N/mm^2	cl.5.4.7.2
Change of total compression force over Δx, $\Delta F_c = (M-0)/z$	2423	kN	cl.5.4.7.1
Lever arm, z	0.763	m	
Note if neutral axis within web, for simplicity, $z = d - h_f/2$;			
Length under consideration, Δx	2500	mm	
Note Δx is the beam length between the point of maximum design moment and the point of zero moment;			cl.5.4.7.2
Shear stress distribution factor, K_S	1.33		
The average design shear stress should then be distributed in proportion to the vertical design shear force diagram to give the horizontal shear stress at any point along the length of the member. For UDLs, K_S maybe taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;			cl.5.4.7.2
Width (rectangular) or web width (flanged), b_w	500	mm	

Longitudinal shear stress limit for no nominal / design vertical reinforcement	**2.35**	N/mm^2	
Surface type	Washed to remove laitance etc		T.5.5

Table 5.5 — Design ultimate horizontal shear stresses at interface

Precast unit	Surface type	Grade of in-situ concrete 25 N/mm²	30 N/mm²	40 and over N/mm²
Without links	As-cast or as-extruded	0.4	0.55	0.65
	Brushed, screeded or rough-tamped	0.6	0.65	0.75
	Washed to remove laitance or treated with retarder and cleaned	0.7	0.75	0.80
With nominal links projecting into in-situ concrete	As-cast or as-extruded	1.2	1.8	2.0
	Brushed, screeded or rough-tamped	1.8	2.0	2.2
	Washed to remove laitance or treated with retarder and cleaned	2.1	2.2	2.5

NOTE 1 The description "as-cast" covers those cases where the concrete is placed and vibrated leaving a rough finish. The surface is rougher than would be required for finishes to be applied directly without a further finishing screed but not as rough as would be obtained if tamping, brushing or other artificial roughening had taken place.

NOTE 2 The description "as-extruded" covers those cases in which an open-textured surface is produced direct from an extruding machine.

NOTE 3 The description "brushed, screeded or rough-tamped" covers those cases where some form of deliberate surface roughening has taken place but not to the extent of exposing the aggregate.

NOTE 4 For structural assessment purposes, it may be assumed that the appropriate value of γ_m included in the table is 1.5.

Longitudinal shear stress limit for no nominal / design vertical reinforcement	**110%**		**NOT OK**
.2(3)			
Required nominal vertical reinforcement per unit length, $0.15\%b_w$	**750**	mm^2/m	cl.5.4.7.3
Provided vertical reinforcement per unit length, A_e	3142	mm^2/m	
Note $A_e = A_{sv,prov} / S + A_{sv,prov,t} / S_t$;			
Required nominal vertical reinforcement per unit length utilisation, $0.15\%b_w/$	**24%**		**OK**
Note UT set to 0% if longitudinal shear stress limit for no nominal vertical reinforcement UT <= 100%;			
Required design vertical reinforcement per unit length, A_h	**2949**	mm^2/m	
			cl.5.4.7.4, Fore
Required design vertical reinforcement per unit length utilisation, A_h/A_e	**94%**		**OK**
Note UT set to 0% if longitudinal shear stress limit for no design vertical reinforcement UT <= 100%;			

$$A_h = \frac{1\,000 b v_h}{0.95 f_y}$$

Note $f_{yv} \leq 460N/mm^2$

BS5400-4

Longitudinal Shear Within Web Rectangular or Flanged Beam (BS5400-4)

Longitudinal shear force per unit length, $V_1 = K_S \cdot \Delta F_c / \Delta x$	**1289**	kN/m	
Change of total compression force over Δx, $\Delta F_c = (M-0)/z$	2423	kN	
Lever arm, z	0.763	m	
Note if neutral axis within web, for simplicity, $z = d - h_f/2$;			
Length under consideration, Δx	2500	mm	
Note Δx is the beam length between the point of maximum design moment and the point of zero moment;			
Shear stress distribution factor, K_S	1.33		
The longitudinal shear should be calculated per unit length. For UDLs, K_S may be taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;			*cl.7.4.2.3*
Width (rectangular) or web width (flanged), b_w	500	mm	
Longitudinal shear force limit per unit length, $V_{1,limit}$	**1637**	kN/m	

V_1 should not exceed the lesser of the following:
a) $k_1 f_{cu} L_s$
b) $v_1 L_s + 0.7 A_e f_y$

(a)	2625	kN/m	*cl.7.4.2.3*
(b)	1637	kN/m	*cl.7.4.2.3*

Table 31 — Ultimate longitudinal shear stress, v_1, and values of k_1 for composite members

Type of shear plane	Longitudinal shear stress for concrete grade 20	25	30	40 or more	k_1
	N/mm^2	N/mm^2	N/mm^2	N/mm^2	
Monolithic construction	0.90	0.90	1.25	1.25	0.15
Surface type 1	0.50	0.63	0.75	0.80	0.15
Surface type 2	0.30	0.38	0.45	0.50	0.09

NOTE For construction with lightweight aggregate concrete, the values given in this table should be reduced by 25 %.

Concrete bond constant, k_1	0.15		*T.31*
Ultimate longitudinal shear stress limit, v_1	1.25	N/mm^2	*T.31*
Surface type: Monolithic construction			*T.31*
Length of shear plane, $L_s = b_w$	500	mm	
Provided vertical reinforcement per unit length, A_e	3142	mm^2/m	
Note $A_e = A_{sv,prov} / S + A_{sv,prov,t} / S_t$;			
Note reinforcement provided for coexistent bending effects and shear reinforcement crossing the shear plane, provided to resist vertical shear, may be included provided they are fully anchored;			*cl.7.4.2.3*
Characteristic strength of reinforcement, $f_{yv} \leq 460N/mm^2$	460	N/mm^2	
Longitudinal shear force limit per unit length utilisation, $V_1/V_{1,limit}$	**79%**		**OK**
Required nominal vertical reinforcement per unit length, $0.15\%L_s$	**750**	mm^2/m	*cl.7.4.2.3*
Required nominal vertical reinforcement per unit length utilisation, $0.15\%L_s/$	**24%**		**OK**

Note UT set to 0% if longitudinal shear force limit per unit length for no nominal vertical reinforcement UT <= 100%;

BS8110

Deep Beam Rectangular Beam

Note that this design check is performed for both rectangular and flanged sections, adopting rectangular section equations in the case of flanged sections; Note this section assumes a ***top loaded*** *beam with* ***no web openings****; Note this section assumes that the deep beam is* ***laterally and rotationally restrained*** *at the top and bottom of the beam overall depth, h (ref. CIRIA Guide 2 cl.2.2.3);*

Span to depth ratio, span / h	10.00		
Applicability of deep beam design	Not Applicable		OK

Note deep beam design is applicable for {1.0 ≤ span / h ≤ 2.0 s/s; 1.0 ≤ span / h ≤ 2.5 cont; 0.5 ≤ span / h ≤ 1.0 cant} (Reynolds cl.21.4.1 and CIRIA Guide 2 cl.1.3);

Concrete type	Normal weight		
Horizontal shear link diameter, $\phi_{link,h}$	None	mm	
Number of horizontal shear links in a horizontal section, i.e. number of legs,	N/A		
Area provided by all horizontal shear links in a horizontal section, $A_{sv,prov,h}$ =	N/A	mm^2	
Pitch of horizontal shear links, S_h	N/A	mm	
$2.\pi.(\phi_{link}$ or $\phi_{link,t})^2/4$ / (b_w.(S or S_t)) (>0.20% G460; >0.25% G250)	N/A	%	N/A
$2.\pi.\phi_{link,h}{}^2/4$ / ($b_w.S_h$) (>0.20% G460; >0.25% G250)	N/A	%	N/A
Tension steel area provided (deep beam)	N/A	mm^2	CIRIA
% Min tension reinforcement (deep beam)	N/A	%	Guide 2
% Min tension reinforcement (deep beam) (>= 0.52 $\sqrt{f_{cu}}/0.95f_y$, f_y ≤460N/mm^2)			cl.2.6.2
% Min tension reinforcement (deep beam) utilisation	N/A		N/A
All detailing requirements met ?	N/A		

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Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

			<u>BS8110</u>
Deep Beam Bending			Reynolds
Design bending moment, M	N/A	kNm	
Tension steel (deep beam), $A_{s,db}$ = 1.75M / [f_y.h], f_y≤460N/mm^2	N/A	mm^2	T.148
Note that the factor 1.75 is obtained from 1 / (lever arm factor 0.6 x material factor 0.95);			cl.21.4.1
Tension steel zone depth, T_{zone} (sag s/s, sag cont, hog cant)	N/A	mm	cl.21.4.1
Tension steel zone depth, T_{zone} (hog cont)	N/A	mm	
Note $A_{s,db}$ to be distributed over depth of T_{zone} from tension face;			cl.21.4.1
Note T_{zone} = (5h – span$_{lim}$) / 20 s/s sag and cont sag, (5h – 2 x span$_{lim}$) / 20 cant hog;			cl.21.4.1 / SE
Tension steel area provided (deep beam)	N/A	mm^2	
Tension steel area provided (deep beam) utilisation	N/A		N/A
Deep Beam Shear			Reynolds
Design shear force, V_d	N/A	kN	
Note the ultimate shear force limit is b_w.h.f_c'/10 γ_m where f_c' is the cylinder comp strength and			cl.21.4.1
Ultimate shear force utilisation	N/A		N/A
Area of tension steel reinforcement provided, $A_{s,prov}$	N/A	mm^2	
Clear distance from edge of load to face of support, a_1 — DL @ mid 0.625h	N/A	mm	T.148
Note for UDLs, concentrate total UDL at {span/4 s/s and cont, span/2 cant} from the support(s);			T.148
Ratio a_1/h	N/A		N/A
Note ensure a_1/h is not greatly outside range of 0.23 to 0.70;			T.148
Angle between horizontal bar and critical diagonal crack, $\theta = \tan^{-1}(h/a_1)$	N/A	degrees	T.148
Empirical coefficient, k_1 = {0.70 NWC, 0.50 LWC}	N/A		T.148
Empirical coefficient, k_2 = {100 plain round bars, 225 deformed bars}	N/A	N/mm^2	T.148
Cylinder splitting tensile strength, $f_t = 0.5(f_{cu})^{0.5}$	N/A	N/mm^2	T.148
Minimum breadth for deep beam, $b_w \approx$ MAX {0, 0.65V_d/[k_1.(h-0.35a_1).f_t]}	N/A	mm	N/A
Number of rows of horizontal shear links in a vertical cross-section, n	N/A		
Note the no. of rows of horizontal shear links reduced to account for T_{zone}, i.e. (h – T_{zone})/S_h;			
Design horizontal links shear capacity, $V_r = k_2 \Sigma A_{sv,prov,h}.a_2.\sin^2\theta/h$	N/A	kN	T.148
Note the summation of the depths at which the horizontal shear links intersect the diagonal crack, Σa_2 is calculated as S_h.n.(n+1)/2 where n is the number of rows of horizontal shear links;			
Check V_d < 1.0V_1 for no horizontal links (minor elements)	N/A		T.148
Concrete shear capacity, V_1	N/A	kN	T.148
Note V_1 = MAX[0,k_1.(h-0.35a_1).f_t.b_w]+k_2.$A_{s,prov}$.d.$\sin^2\theta$/h;			T.148
Check V_d > 0.0 for design horizontal links	N/A		T.148
Concrete and design horizontal links shear capacity $V_r + V_1$	N/A	kN	T.148
Design shear resistance (deep beam) utilisation	N/A		N/A

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				BS8110
Deep Beam Bending				*CIRIA*
				Guide 2
Design bending moment, M		N/A	kNm	
Note the ultimate bending moment limit is $0.12f_{cu}b_wh^2$;				*cl.2.4.1*
Ultimate bending moment utilisation		N/A		N/A
Tension steel (deep beam), $A_{s,db}$ = M / [0.95f_y.z], f_y≤460N/mm^2		N/A	mm^2	*cl.2.4.1*
Lever arm at which the tension steel (deep beam) acts, z		N/A	mm	*cl.2.4.1*
Simply supported, z = 0.2 x $span_{lim}$ + 0.4h		N/A	mm	*cl.2.4.1*
LF Continuous, z = 0.2 x $span_{lim}$ + 0.3h		N/A	mm	*cl.2.4.1*
Cantilever, z = 0.4 x $span_{lim}$ + 0.4h		N/A	mm	*SELF*
Tension steel zone depth, T_{zone} (sag s/s, sag cont, hog cant)		N/A	mm	*cl.2.4.1*
Tension steel zone depth, T_{zone} (hog cont) upper band	N/A	N/A	mm	*cl.2.4.1*
Tension steel zone depth, T_{zone} (hog cont) lower band	N/A	N/A	mm	*cl.2.4.1*
Note $A_{s,db}$ to be distributed over depth of T_{zone} from tension face;				*cl.2.4.1*
Note T_{zone} = 0.2h s/s sag, cont sag and cant hog;				*cl.2.4.1*
Note T_{zone} = 0.2h cont hog upper band 0.5($span_{lim}$/h-1)$A_{s,db}$, 0.2h-0.8h cont hog lower band remainder;				
Tension steel area provided (deep beam)		N/A	mm^2	
Tension steel area provided (deep beam) utilisation		N/A		N/A
Deep Beam Shear				*CIRIA*
				Guide 2
Design shear force, V_d		N/A	kN	
Note the ultimate shear force limit is min{b_w.h. v_u,2b_w.h^2 $v_c k_s/x_e$} where v_u is the ultimate				*cl.2.4.2*
concrete shear strength from CP 110 T.6 and T.26 replaced by min{0.8$f_{cu}^{0.5}$,{5.0,7.0}N/mm^2}				
and v_c is the design concrete shear strength from CP 110 T.5 and T.25 replaced by v_c;				
Factor, k_s = 1.0 for h/b_w < 4, else 0.6		N/A		*cl.2.4.2*
Ultimate shear force utilisation		N/A		N/A
Area of tension steel reinforcement provided, $A_{s,prov}$		N/A	mm^2	
Clear distance from edge of load to face of support, x_e	DL @ mid 0.625h	N/A	mm	*cl.2.4.2*
Note for UDLs, concentrate total UDL at {span/4 s/s and cont, span/2 cant} from the support(s);				*cl.2.4.2*
Ratio x_e/h		N/A		N/A
Note ensure x_e/h is not greatly outside range of 0.23 to 0.70;				*cl.3.4.2*
Angle between horizontal bar and critical diagonal crack, $\theta = \tan^{-1}(h/x_e)$		N/A	degrees	*cl.2.4.2*
Empirical coefficient, λ_1 = {0.44 NWC, 0.32 LWC}		N/A		*cl.3.4.2*
Empirical coefficient, λ_2 = {0.85 plain round bars, 1.95 deformed bars}		N/A	N/mm^2	*cl.3.4.2*
Number of rows of horizontal shear links in a vertical cross-section, n		N/A		
Note the no. of rows of horizontal shear links reduced to account for T_{zone}, i.e. (h – T_{zone})/S_h;				
Design horizontal links shear capacity, $V_r = 100\lambda_2\Sigma A_{sv,prov,h}.y_r.\sin^2\theta/h$		N/A	kN	*cl.3.4.2*
Note the summation of the depths at which the horizontal shear links intersect the diagonal				
crack, Σy_r is calculated as S_h.n.(n+1)/2 where n is the number of rows of horizontal shear links;				
Check V_d < 1.0V for no horizontal links (minor elements)		N/A		*cl.3.4.2*
Concrete shear capacity, V (≤ $1.3\lambda_1\sqrt{f_{cu}}.b_w.h$)		N/A	kN	*cl.3.4.2*
Note V = MAX[0, λ_1.(h-0.35x_e). $\sqrt{f_{cu}}$.b_w]+100 λ_2.$A_{s,prov}$.d.$\sin^2\theta$/h;				*cl.3.4.2*
Note require [100 λ_2.$A_{s,prov}$.d.$\sin^2\theta$/h] / V ≥ 0.20;		*N/A*		*N/A*
Check V_d > 0.0 for design horizontal links		N/A		*cl.3.4.2*
Concrete and design horizontal links shear capacity V_r + V (≤ λ_1.√		N/A	kN	*cl.3.4.2*
Note v_{max} = 1.3 $\sqrt{f_{cu}}$;				*T.5*
Note require [V_r + 100 λ_2.$A_{s,prov}$.d.$\sin^2\theta$/h] / [V_r + V] ≥ 0.20;		*N/A*		*N/A*
Design shear resistance (deep beam) utilisation		N/A		N/A

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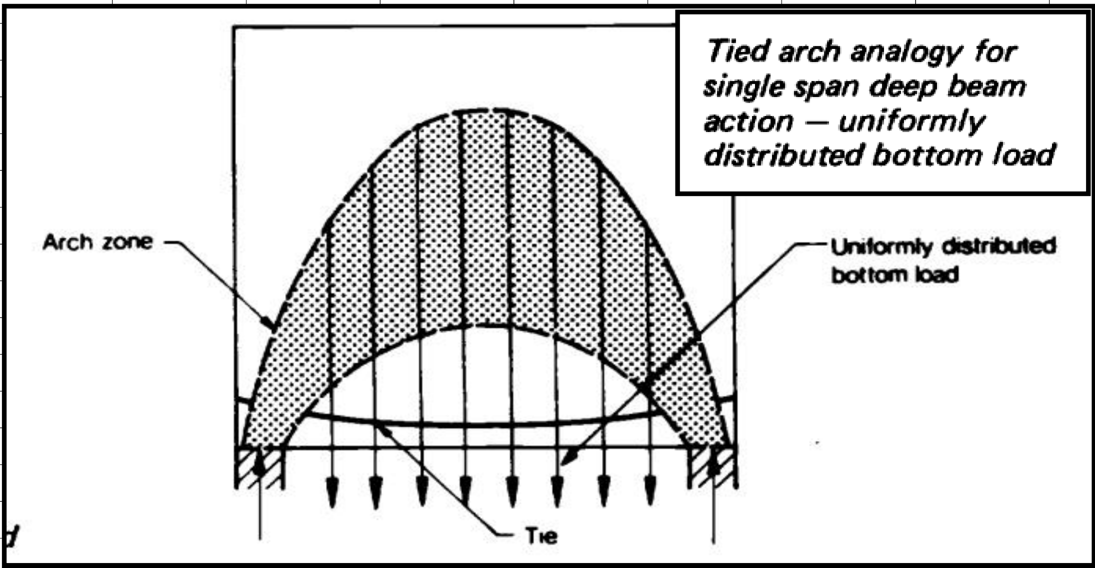
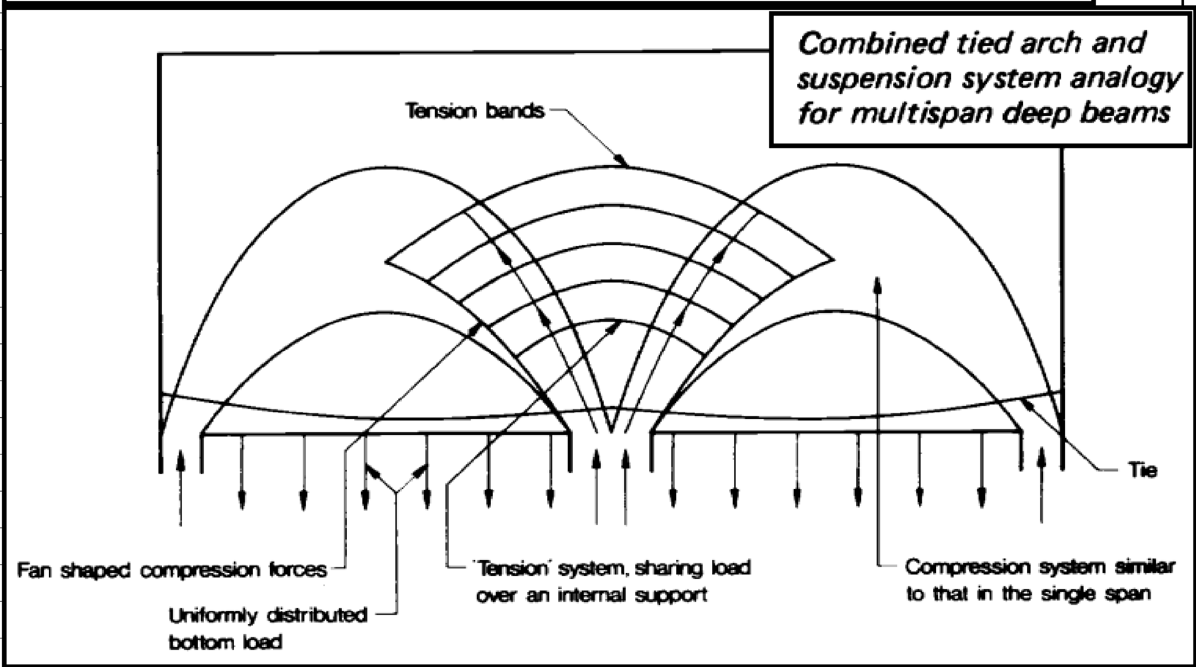
BS8110

Detailing Instructions (Deep Beam)





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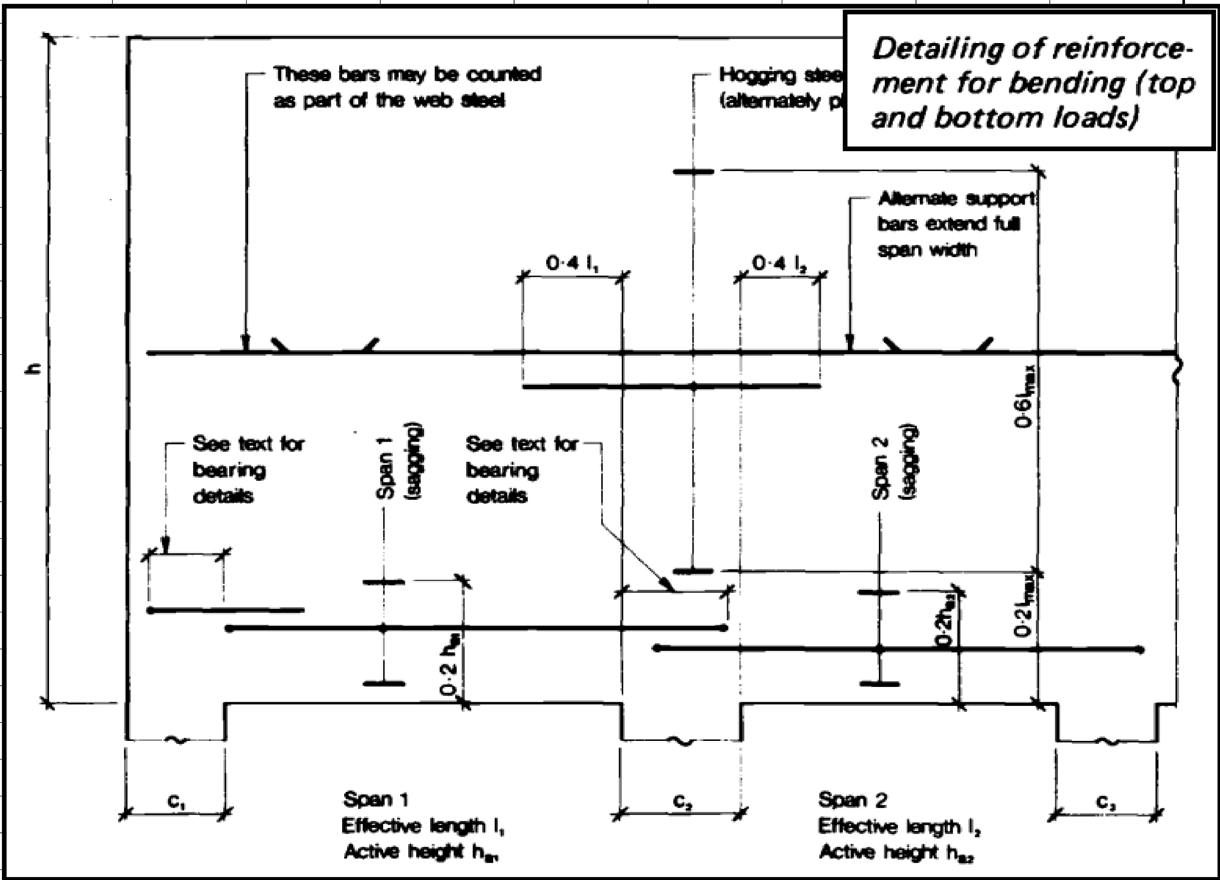

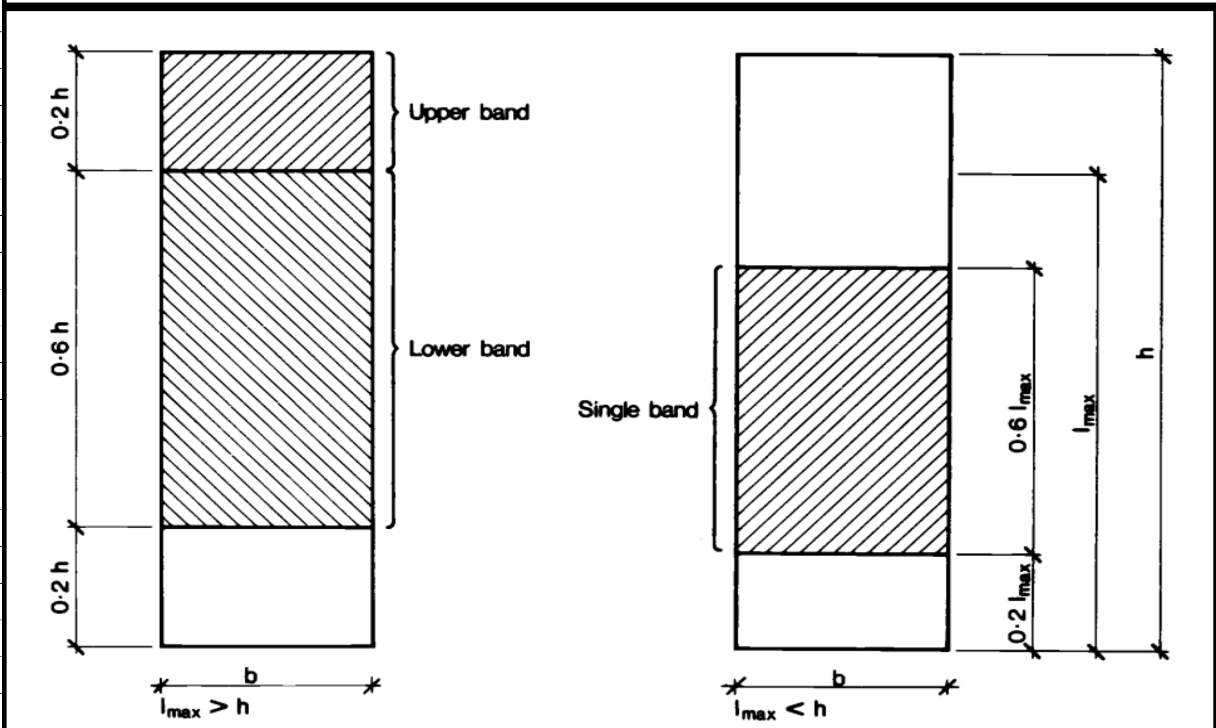

Figure 12 *Bands of reinforcement for hogging moments*

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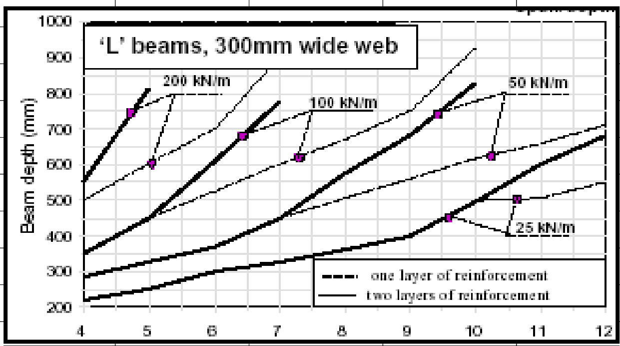
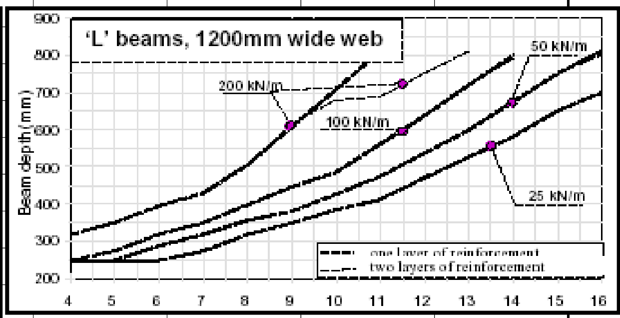
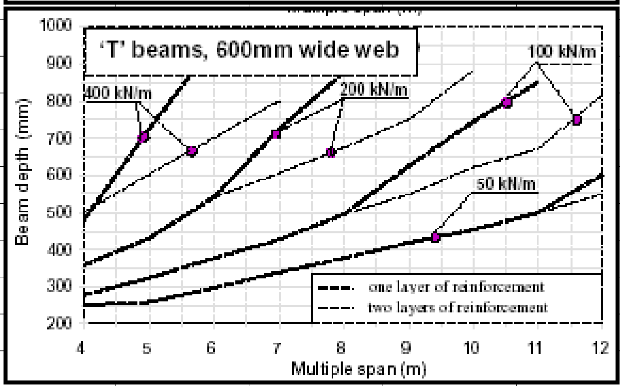
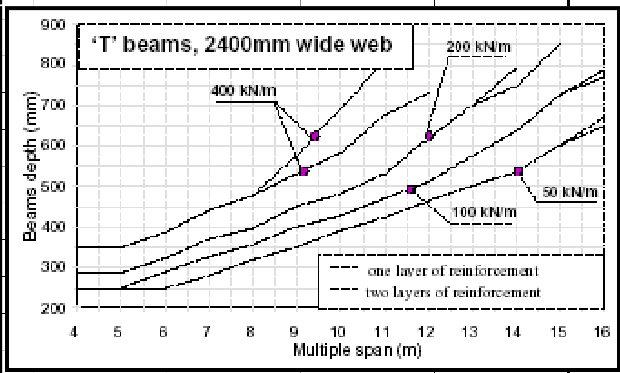
BS8110

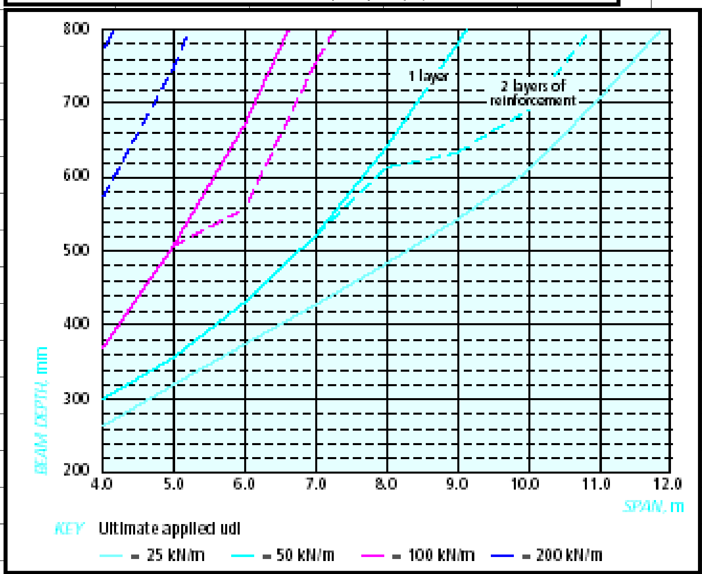
Scheme Design





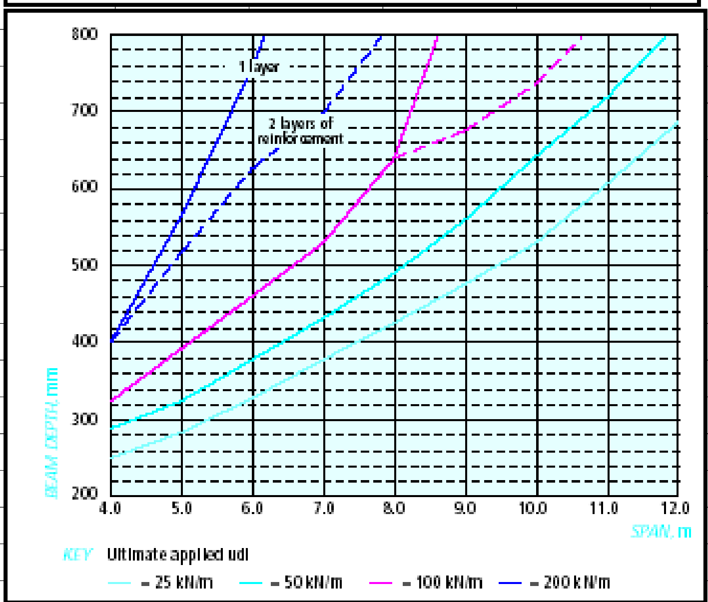






Rectangular beams
300 mm
wide

single span



Rectangular beams
600 mm
wide

single span

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		jXXX	34	
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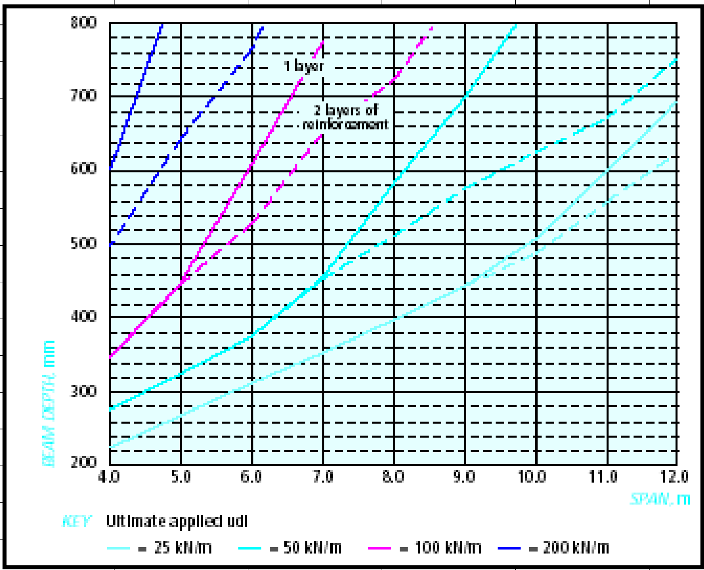
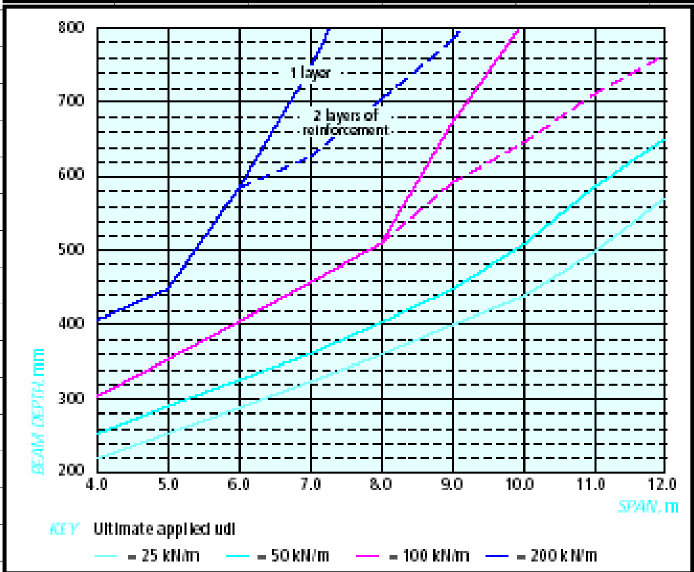


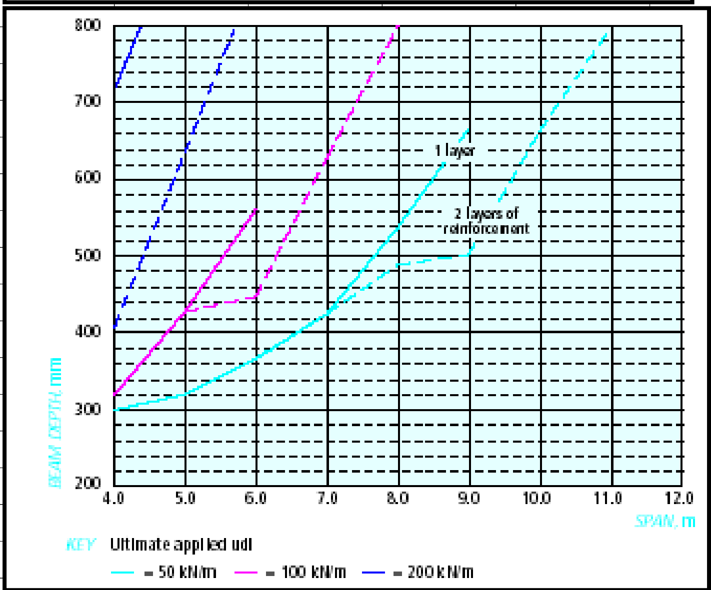
Rectangular beams
300 mm
wide

multiple span



Rectangular beams
600 mm
wide

multiple span



'T' beams
300 mm
wide web

single span

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		jXXX	35	
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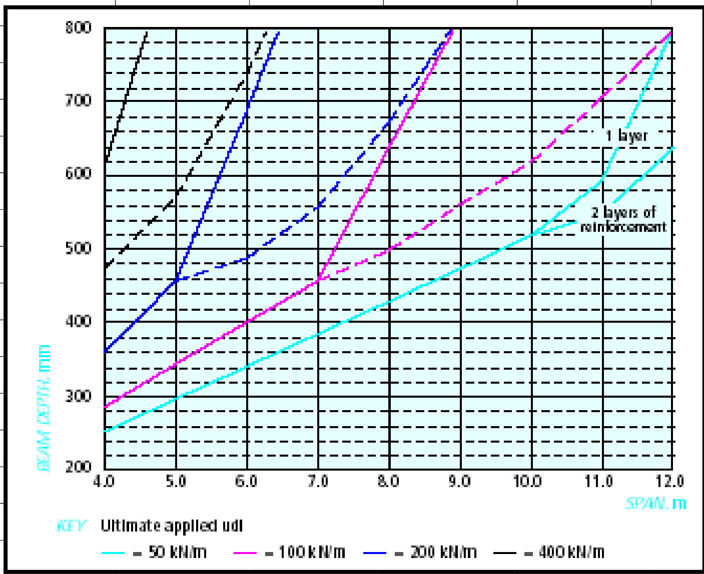
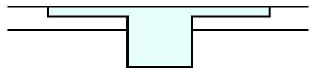
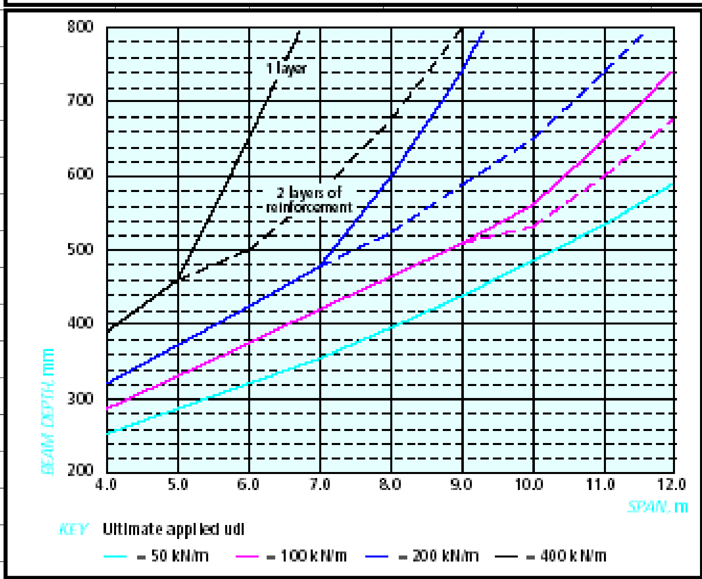


'T' beams
600 mm
wide web

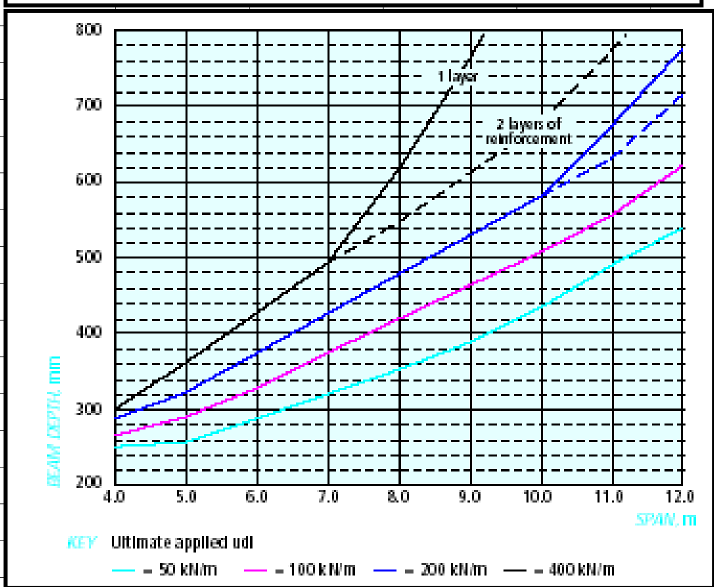
single span





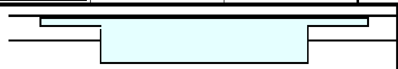
'T' beams
1200 mm
wide web

single span



'T' beams
2400 mm
wide web

single span



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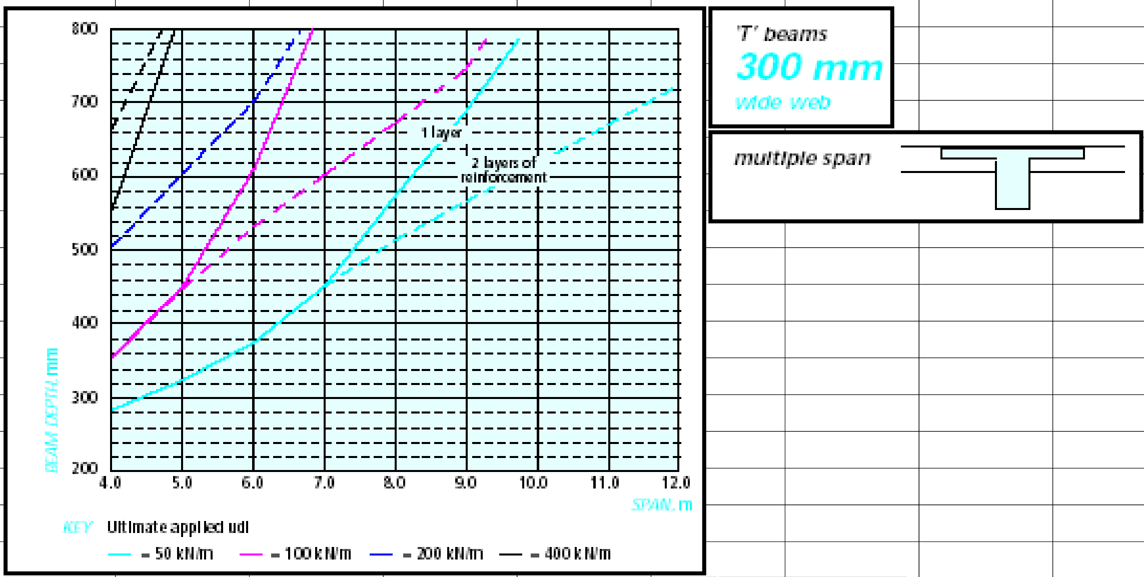
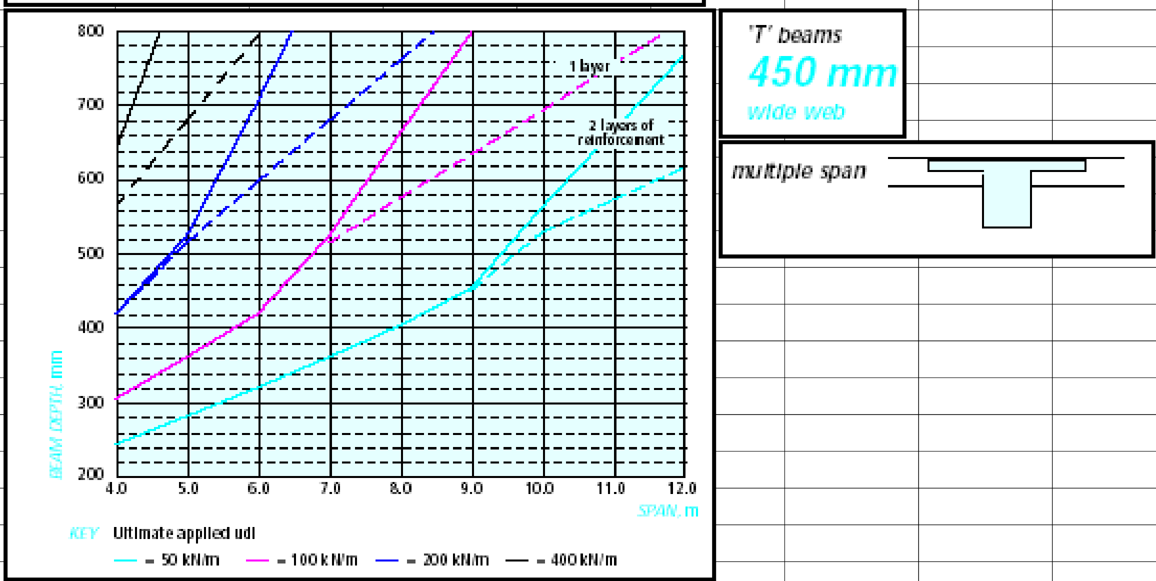
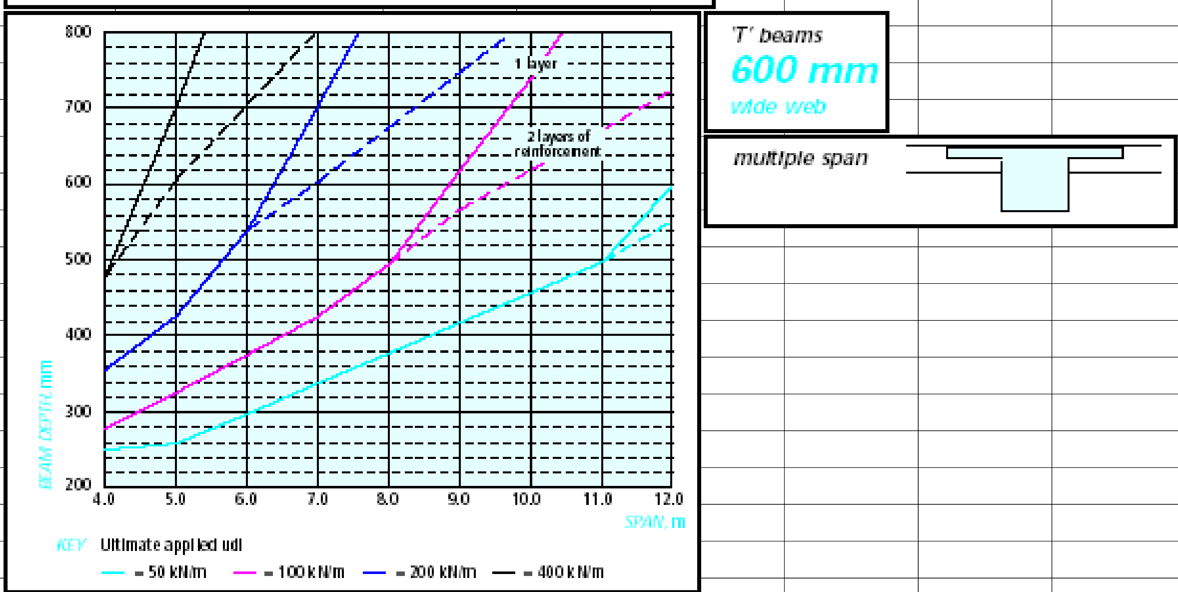
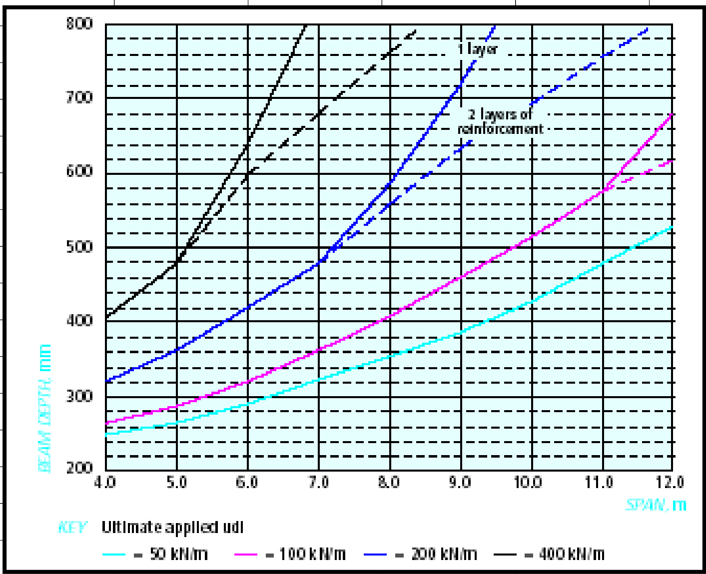




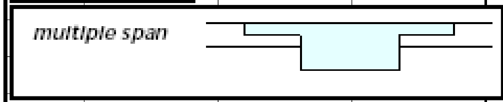


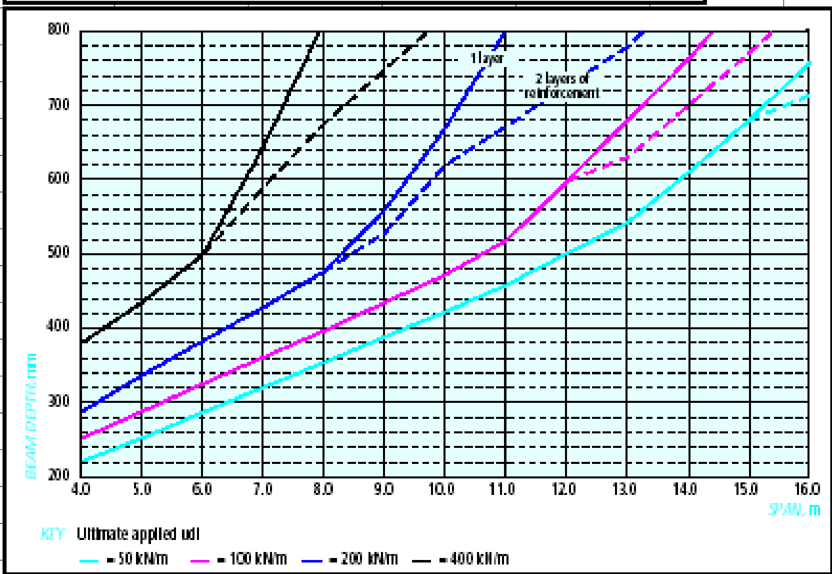
CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	37	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

BS8110

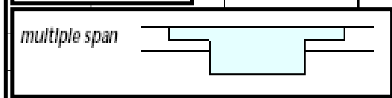


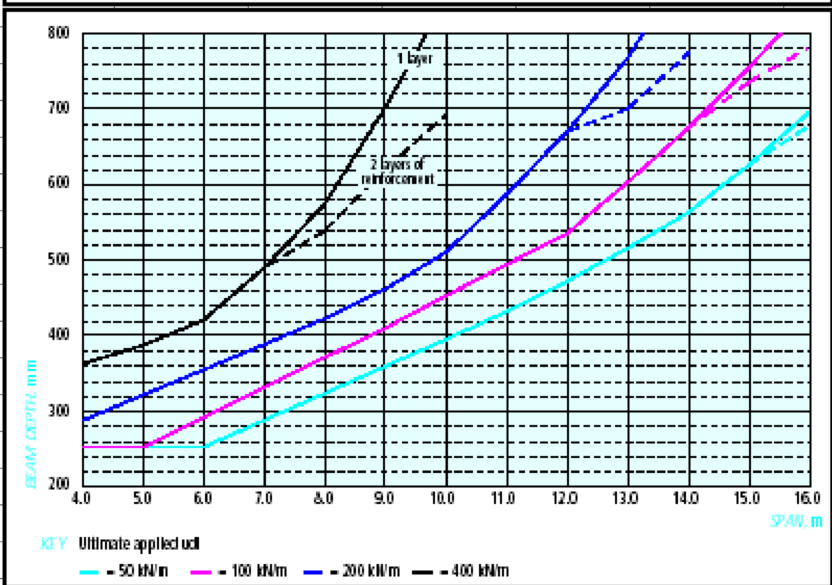
'T' beams
900 mm
wide web



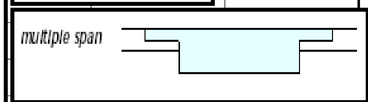


'T' beams
1200 mm
wide web

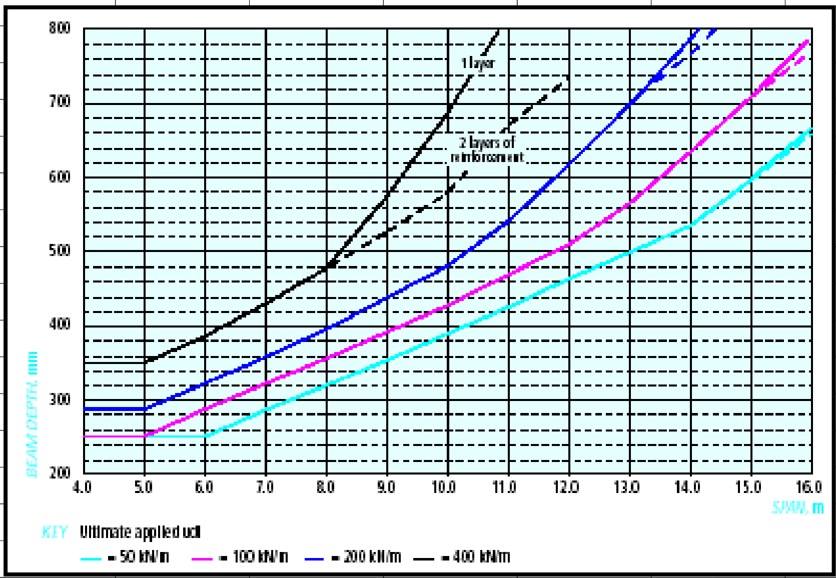




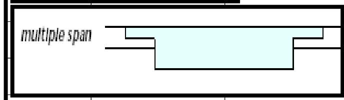
'T' beams
1800 mm
wide web

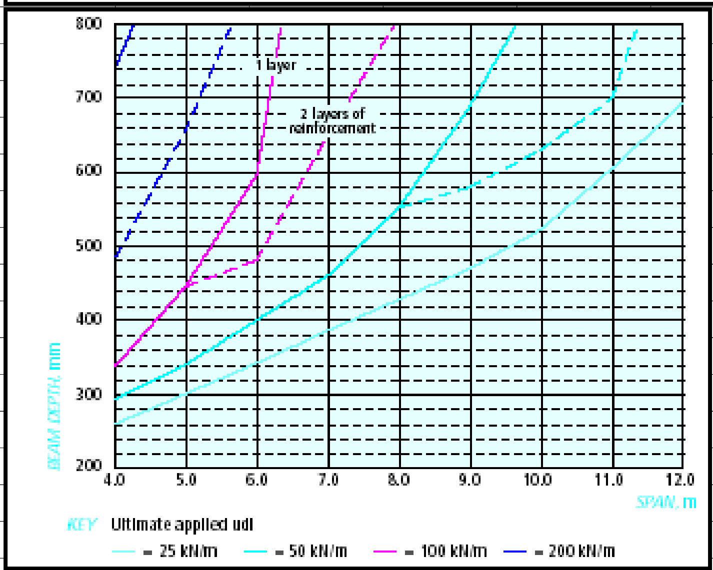


BS8110

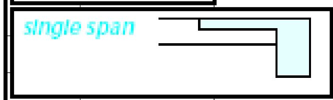


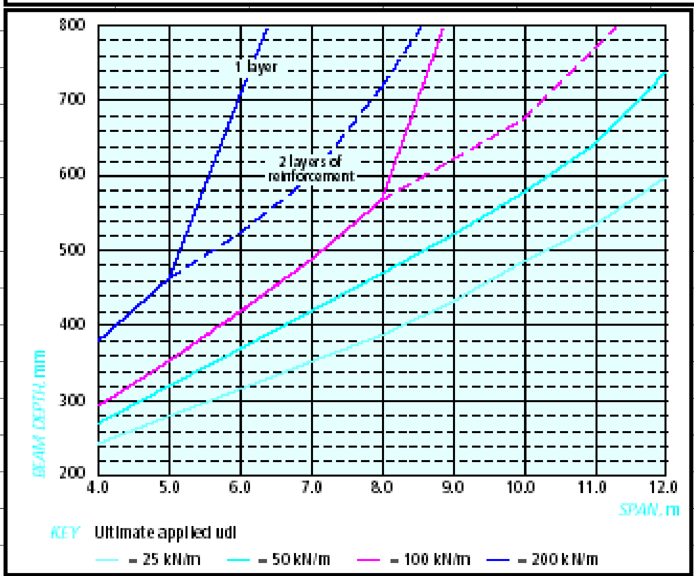
'T' beams
2400 mm
wide web





Inverted 'L' beams
300 mm
wide web

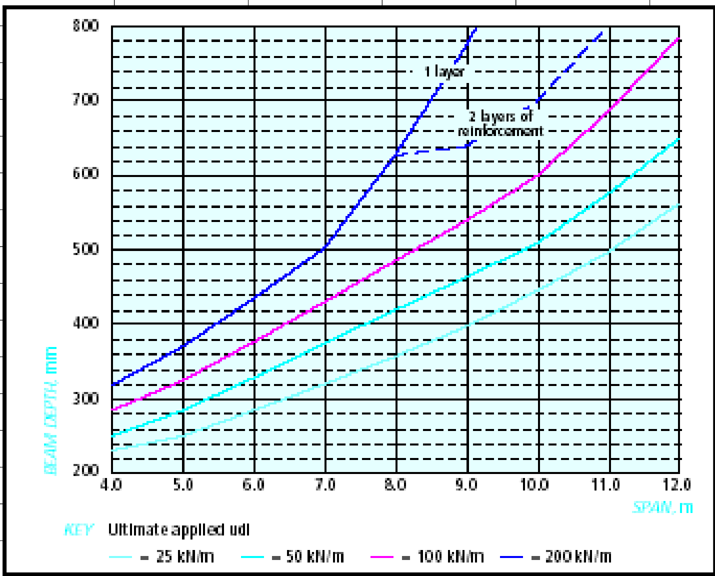




Inverted 'L' beams
600 mm
wide web

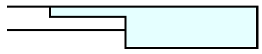
CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 39	Rev.
		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

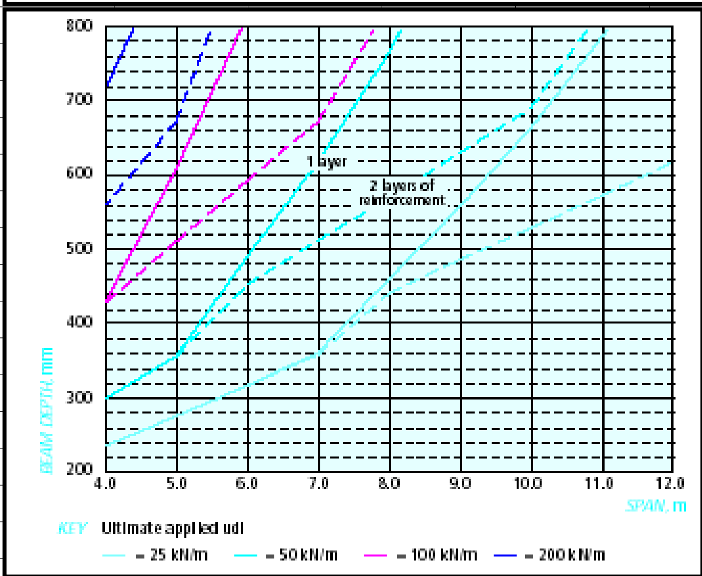
BS8110



Inverted 'L' beams
1200 mm
wide web

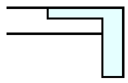
single span

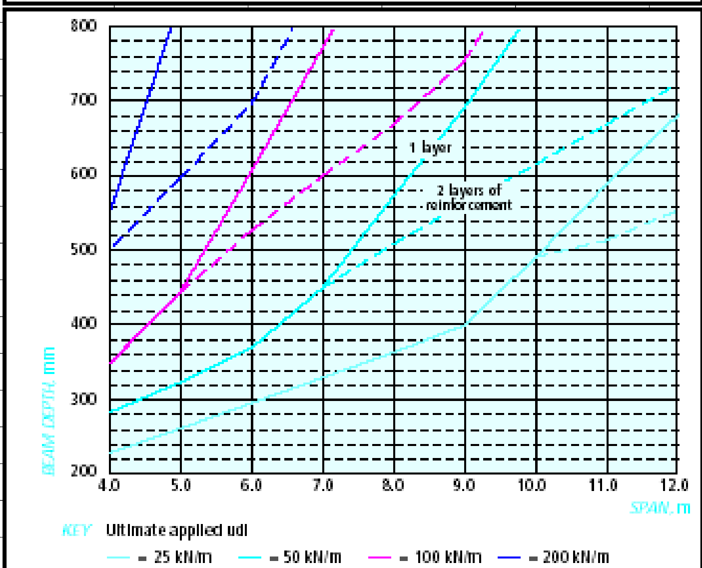




Inverted 'L' beams
225 mm
wide web

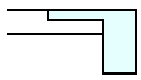
multiple span





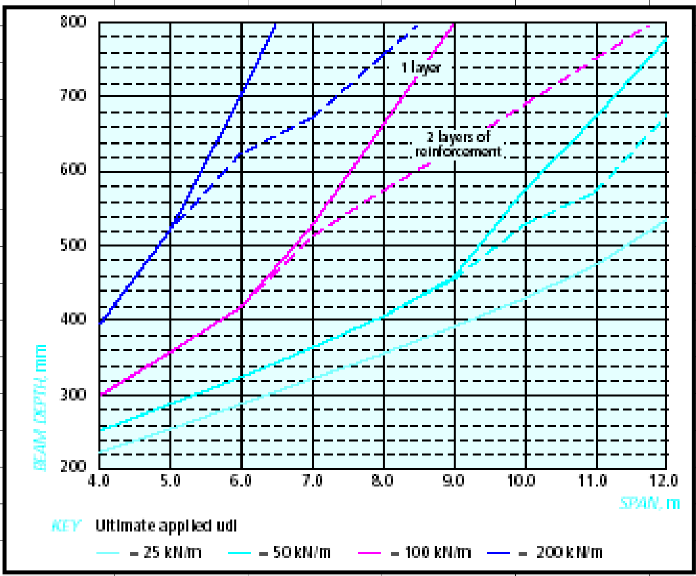
Inverted 'L' beams
300 mm
wide web

multiple span



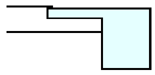
CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	40	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by XX	Date 16/1/2024	Chd.

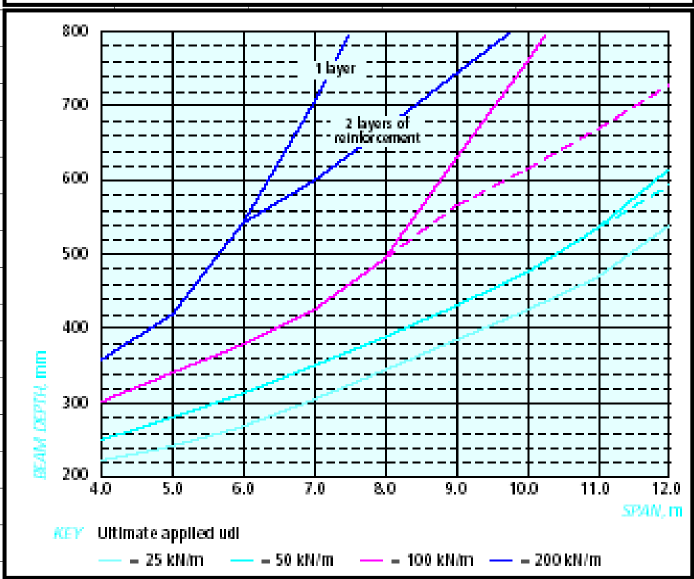
BS8110



Inverted 'L' beams
450 mm
wide web

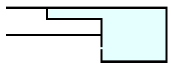
multiple span

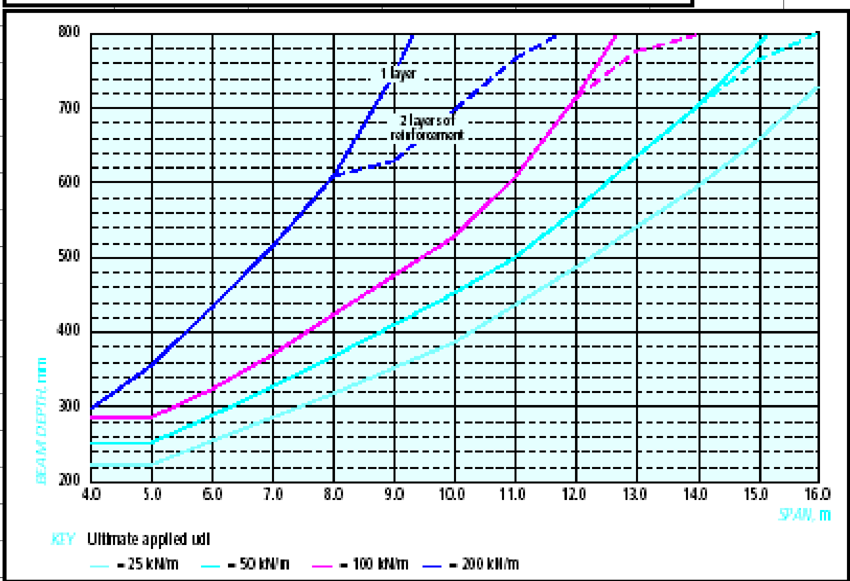




Inverted 'L' beams
600 mm
wide web

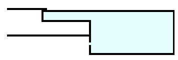
multiple span





Inverted 'L' beams
900 mm
wide web

multiple span



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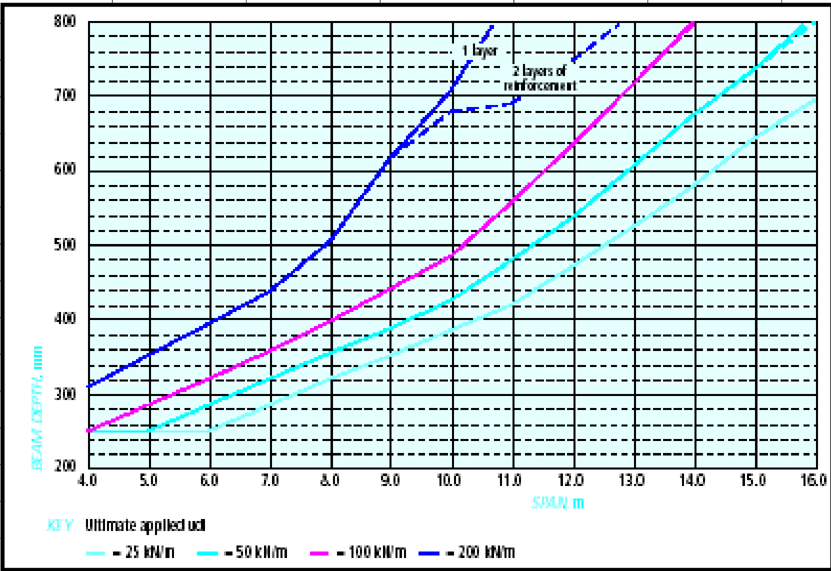

Inverted 'L' beams

1200 mm

wide web

multiple span

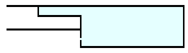

Table 3.1 — Allowable steel stresses in direct or flexural tension for serviceability limit states

Design crack width	Allowable stress	
	Plain bars[a]	Deformed bars[b]
mm	N/mm²	N/mm²
0.1	85	100
0.2	115	130

[a] Plain grade 250 bars complying with BS 4449.

[b] Deformed grade 460 bars complying with BS 4449 or BS 4461 and high-yield wire fabric complying with BS 4483 having a guaranteed yield or proof stress and guaranteed weld strength.

Note optional method of limiting allowable steel stress to full crack width calculation method;

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Typical Initial Span / Effective Depth Ratios

Span-to-depth ratios for beams

Condition	Span-to-depth ratio
Simply supported	15
End-bay	17
Cantilever	6

Transfer beams

The experienced engineer will understand that it is not advisable to determine the section size merely by using span-to-depth tables – careful consideration is needed. Shear strength is often the governing criteria for a reinforced concrete transfer beam. From BS 8110, $v = V/bd$. In no case should v exceed $0.8\sqrt{f_{cu}}$ or 5 N/mm^2 (whichever is smaller). If the section is not to become congested with shear reinforcement it is advisable to limit v to 2 N/mm^2. However, it may be necessary to increase this to 4 N/mm^2. If we assume a well proportioned beam has a width, b, which is half it's depth, then we rearrange the expression above so that:

$d = \sqrt{V}$, where V is in newtons (N) for shear stress of 2 N/mm^2

$d = \sqrt{(V/2)}$ for shear stress of 4 N/mm^2

The headroom under the beam should be checked and consideration given to the connection into the column. Deflection and flexural strength should also be considered because they may govern the design.

Table 5: Span/depth ratios for *insitu* concrete beams *(from Reynolds's Reinforced Concrete Designer's Handl*

Beam span condition	Ultimate line load 25 kN/m	Ultimate line load 50 kN/m	Ultim
Simply supported	18	14	
Continuous	22	17	
Cantilever	9	7	

Table 6: Estimated depths of *insitu* concrete single span T-beams (600mm wide)

Span	4m	5m	6m	7m	8m	
50 kN/m UDL	250mm	300mm	350mm	400mm	500mm	
100 kN/m UDL	275mm	325mm	400mm	450mm	575mm	
200 kN/m UDL	325mm	375mm	450mm	525mm	650mm	

Table 7: Estimated depths of *insitu* concrete single span band-beams (2400mm wide)

Span	6m	7m	8m	9m	10m	
50 kN/m UDL	250mm	300mm	350mm	400mm	475mm	
100 kN/m UDL	300mm	350mm	425mm	500mm	575mm	
200 kN/m UDL	350mm	400mm	475mm	575mm	675mm	

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Notes on Application to Upstand Beams

Rect - s/s	*N/A*	*N/A*
Rect - continuous	*Hog in continuous beam with precast slab*	*Deflections irrelevant*
Rect - cantilever	*Hog in cantilever beam with precast slab*	*Deflections relevant*
T - s/s	*N/A*	*N/A*
T - continuous	*Hog in continuous interior beam with insitu slab*	*Deflections irrelevant*
T - cantilever	*Hog in cantilever interior beam with insitu slab*	*Deflections relevant*
L - s/s	*N/A*	*N/A*
L - continuous	*Hog in continuous edge beam with insitu slab*	*Deflections irrelevant*
L - cantilever	*Hog in cantilever edge beam with insitu slab*	*Deflections relevant*
Rect - s/s	*Sag in s/s beam with precast or insitu slab*	*Deflections relevant*
Rect - continuous	*Sag in continuous beam with precast or insitu slab*	*Deflections relevant*

ook)
ate line load 100 kN/m
10
12
5

9m	10m
575mm	675mm
675mm	800mm
775mm	925mm

11m	12m
550mm	650mm
650mm	750mm
775mm	875mm