

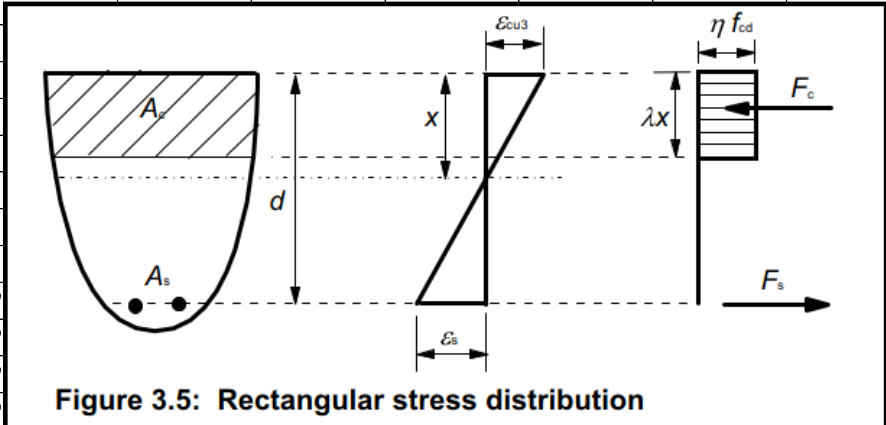
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.			Rev.	
						jXXX	1				
						Member/Location					
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.					
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.	
						BS8110					
Effects From Structural Analysis						Do not adopt code equivalence			▼		
Design axial force, F (tension -ve and comp. +ve) (ensure < 0.1f _{cu} b _w h)						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,r} φ _{link,t,r} cover _{add,t}) - [						917		mm			
Effective depth to compression steel, d' = cover + MAX(φ _{link,r} φ _{link,t,r} 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 ·π·φ _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 ·π·φ _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,r} 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,r} 25mm)						25		mm			

CONSULTING ENGINEERS Engineering Calculation Sheet Consulting Engineers						Job No.		Sheet No.		Rev.	
						jXXX		2			
						Member/Location					
Job Title		Member Design - Reinforced Concrete Beam BS8110,							Org. Ref.		
Member Design - RC Beam						Made by XX		Date 16/9/2026		Chd.	
										BS8110	
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} = π.ϕ _{link} ² /4.n _{leg}						314		mm ²			
Pitch of shear links, S						100		mm			
Torsion Reinforcement Details											
Torsion link diameter, ϕ _{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} = π.ϕ _{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} must						0		0		0	
Utilisation Summary											
Item						UT		Remark			
RB tens steel						86%		OK			
RB comp steel						N/A		N/A			
RB % min tens reinf.						10%		20%		OK	
RB % min comp reinf.						N/A		N/A			
RB % max tens and c						32%		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 c						N/A		N/A			
RB shear ultimate str						54%		57%		77%	
RB shear design capa						72%		42%		94%	
RB torsion ultimate st						0%		0%		0%	
RB torsion design cap						0%		0%		0%	
RB shear and torsion						54%		57%		77%	
RB or FB deflection requirements						49%		OK			
Total utilisation rectangular beam						94%		OK			
Total utilisation rectangular beam (excluding defl						94%		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/m³)						158		kg/m³			
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/m³)						222		kg/m³		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/m³ can be obtained from 110.0 x % rebar];											
Material cost: concrete, c						315		units/m³		steel, s	
Reinforced concrete material cost = [c+(est. rebar quant).s].(b _w or b).						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 /d)											

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.	
						jXXX	4			
						Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.				
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.
										BS8110
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, ϕ _{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

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No. jXXX		Sheet No. 5		Rev.	
				Member/Location					
Job Title		Member Design - Reinforced Concrete Beam BS8110,		Drg. Ref.					
Member Design - RC Beam		Made by XX		Date		16/9/2026		Chd.	
								BS8110	
Bending Rectangular Beam (Singly or Doubly Reinforced) (BS8110)									
Figure 3.3 (amended) – Simplified stress block for concrete at ultimate limit state									
$M_u = K' f_{cu} b d^2$		Note b here is b_w ;		2296		kNm			
Ratio, K'				0.156					
$\frac{v_i}{\beta_b} \leq 1.10$		$K' =$		0.156 for $f_{cu} \leq 60 \text{ N/mm}^2$		0.156		cl.3.4.4.4 BC	
$0.90 \leq \frac{v_i}{\beta_b} \leq 1.10$		$K' =$		0.120 for $60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A		cl.3.4.4.4 BC	
$0.90 > \frac{v_i}{\beta_b}$		$K' =$		0.094 for $75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A		cl.3.4.4.4 BC	
$0.90 > \frac{v_i}{\beta_b}$		$K' =$		$0.402(\beta_b - 0.4) - 0.18(\beta_b - 0.4)^2$, for $f_{cu} \leq 60 \text{ N/mm}^2$		N/A		cl.3.4.4.4 BC	
$0.90 > \frac{v_i}{\beta_b}$		$K' =$		$0.357(\beta_b - 0.5) - 0.143(\beta_b - 0.5)^2$ for $60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A		cl.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: $M < M_u \rightarrow$ no compression steel				VALID					
$K = M / b d^2 f_{cu}$		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 $\beta_b = 1.0$ and NSC)									
for no compression steel,		i.e. $d >$		823		and $h >$		906 mm	
$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\}$		$z \leq 0.95d$		763		mm			
Depth of neutral axis, $x \leq x_{limit}$		342		≤		459		mm OK	
Neutral axis, x		$x =$		$(d - z) / 0.45$, for $f_{cu} \leq 60 \text{ N/mm}^2$		342		mm 3.4.4.4 BC	
				$(d - z) / 0.40$, for $60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A		mm 3.4.4.4 BC	
				$(d - z) / 0.36$, for $75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A		mm 3.4.4.4 BC	
Neutral axis limit, x_{limit}		$0.90 \leq \frac{v_i}{\beta_b} \leq 1.10$		$x \leq 0.5d$ for $f_{cu} \leq 60 \text{ N/mm}^2$		459		mm 3.4.4.4 BC	
				$x \leq 0.4d$ for $60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A		mm 3.4.4.4 BC	
				$x \leq 0.33d$, for $75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A		mm 3.4.4.4 BC	
		$0.90 > \frac{v_i}{\beta_b}$		$x \leq (\beta_b - 0.4)d$ for $f_{cu} \leq 60 \text{ N/mm}^2$		N/A		mm 3.4.4.4 BC	
				$x \leq (\beta_b - 0.5)d$ for $60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A		mm 3.4.4.4 BC	
Tension steel		$A_s = \frac{M}{(0.95 f_y) z}$, $f_y \leq 460 \text{ N/mm}^2$		5544		mm ²		Foreword	
Back-analysis of x in $(0.67 f_{cu} / 1.5) \cdot (s \cdot b_w) = (f_y / 1.05) \cdot (A_{s,prov})$, $f_y \leq 460$		401		mm		OK			
$s =$		$0.9x$ for $f_{cu} \leq 60 \text{ N/mm}^2$;		0.90x		mm		3.4.4.4 BC	
		$0.8x$ for $60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A		mm		3.4.4.4 BC	
		$0.72x$ for $75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A		mm		3.4.4.4 BC	
Note for an under reinforced section, require $x \leq x_{limit}$ for yielding of tension steel;									

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.	
						jXXX	6			
						Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Org. Ref.				
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.
										BS8110
If: $M > M_u \rightarrow$ 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			
$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K'}{0.9}} \right\}$							N/A	mm		
Depth of neutral axis, $x \leq x_{limit}$						N/A	$\leq$	N/A	mm	N/A
Neutral axis, x	$x =$	$(d - z)/0.45, \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
		$(d - z)/0.40, \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
		$(d - z)/0.36, \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
Neutral axis limit, x_{limit}	$0.90 \leq \beta_b \leq 1.10$	$x \leq 0.5d \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
		$x \leq 0.4d \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
		$x \leq 0.33d, \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$				$\beta_b = 1.0$	N/A	mm		3.4.4.4 BC
		$x \leq (\beta_b - 0.4)d \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
		$x \leq (\beta_b - 0.5)d \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
Compression steel		$A_s' = \frac{M - K' f_{cu} b d^2}{0.95 f_y (d - d')}$				$f_y \leq 460 \text{ N/mm}^2$	N/A	mm ²		Foreword
Tension steel		$A_s = \frac{M_u}{0.95 f_y z} + A_s'$				$f_y \leq 460 \text{ N/mm}^2$	N/A	mm ²		Foreword
If $d' > \left(1 - \frac{f_y}{735}\right)x$, use $700 \left(1 - \frac{d'}{x}\right)$ in lieu of $0.95 f_y$						$f_y \leq 460 \text{ N/mm}^2$	N/A			Foreword
Back-analysis of x in $(0.67 f_{cu}/1.5) \cdot (s \cdot b_w) + (f_y/1.05) \cdot (A_{s,prov'}) = (f_y/1$							N/A	mm		N/A
$s =$		$0.9x \text{ for } f_{cu} \leq 60 \text{ N/mm}^2;$ $0.8x \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$ $0.72x \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$					N/A	mm		3.4.4.4 BC
Note for an under reinforced section, require $\epsilon_{st} = \epsilon_{cc}(d-x)/x \geq \epsilon_y$ for yielding of tension steel										
and $\epsilon_{sc} = \epsilon_{cc}(x-d')/x \geq \epsilon_y$ for yielding of compression steel, where $\epsilon_{cc} = 0.0035$ for $f_{cu} \leq 60 \text{ N/mm}^2$										
and $\epsilon_{cc} = 0.0035 - (f_{cu} - 60)/50000$ for $f_{cu} > 60 \text{ N/mm}^2$ and $\epsilon_y = (f_y/1.05)/E_s$;										
Tension steel area provided							6434	mm ²		
Tension steel area provided utilisation							86%			OK
Compression steel area provided							N/A	mm ²		
Compression steel area provided utilisation							N/A			N/A
% Min tension reinforcement							1.29	%		TR49
% Min tension reinforcement ($\geq 0.0024 b_w h$ G250; $\geq \text{MAX}(0.0013, 0.0013(f_{cu}/40)^{2/3}) \cdot b_w h$ G4										
% Min tension reinforcement utilisation							10%			OK
% Min compression reinforcement ($\geq 0.002 b_w h$)							N/A	%		
% Min compression reinforcement utilisation							N/A			N/A
% Max tension reinforcement ($\leq 0.04 b_w h$)							1.29	%		
% Max tension reinforcement utilisation							32%			OK
% Max compression reinforcement ($\leq 0.04 b_w h$)							N/A	%		
% Max compression reinforcement utilisation							N/A			N/A
% Max tension or compression reinforcement utilisation							32%			OK

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers			Job No.	Sheet No.	Rev.
					jXXX	7	
					Member/Location		
Job Title Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.		
Member Design - RC Beam					Made by XX	Date 16/9/2026	Chd.
Bending Rectangular Beam (Singly or Doubly Reinforced) (EC2)					EC2		
					for $f_{ck} \leq 50$ MPa $\lambda = 0.8 - (f_{ck} - 50)/400$ for $50 < f_{ck} \leq 90$ MPa and $\eta = 1.0$ for $f_{ck} \leq 50$ MPa $\eta = 1.0 - (f_{ck} - 50)/200$ for $50 < f_{ck} \leq 90$ MPa		
$M_u = K' f_{ck} b d^2$					1962 kNm		
Ratio, K'					0.167		
$K' = \frac{\eta \alpha_{cc}}{\gamma_c} \cdot \lambda \cdot \left(\frac{x}{d}\right)_{limit} \cdot \left(1 - \frac{\lambda}{2} \left(\frac{x}{d}\right)_{limit}\right)$					0.167		
Partial factor for concrete, $\gamma_c = 1.50$					1.5		
Strength coefficient, $\alpha_{cc} = 1.00$ 0.85					EC2 UK 0.85		
Stress block factor, $\eta = \text{MIN}[1.0, 1.0 - (f_{ck} - 50)/200]$					1.00		
Stress block factor, $\lambda = \text{MIN}[0.8, 0.8 - (f_{ck} - 50)/400]$					0.80		
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: $M < M_u \rightarrow$ no compression steel					VALID		
$K = \frac{M}{b d^2 f_{ck}}$					0.157		
$z = d \left[0.5 + \sqrt{0.25 - \frac{K}{C}} \right], \quad C = \frac{2 \eta \alpha_{cc}}{\gamma_c}$					765 mm		
Depth of neutral axis, $x \leq x_{limit}$					381 ≤ 411 mm OK		
Neutral axis, x					381 mm		
$x_{limit} = [(\beta_b - k_1)/k_2] \cdot d$ for $f_{ck} \leq 50$ MPa					EC2 411 mm		
$k_1 = 0.44$ 0.40					0.44		
$k_2 = 1.25(0.6 + 0.0014/\epsilon_{cu2})$ $0.6 + 0.0014/\epsilon_{cu2}$					1.25		
Neutral axis limit, x_{limit}					N/A mm		
$x_{limit} = [(\beta_b - k_3)/k_4] \cdot d$ for $f_{ck} > 50$ MPa					EC2		
$k_3 = 0.54$ 0.40					N/A		
$k_4 = 1.25(0.6 + 0.0014/\epsilon_{cu2})$ $0.6 + 0.0014/\epsilon_{cu2}$					N/A		
$\epsilon_{cu2} = 0.0035$ $f_{ck} \leq 50$ MPa					0.0035		
$\epsilon_{cu2} = [2.6 + 35[(90 - f_{ck})/100]^4]/1000$ $f_{ck} > 50$ MPa					N/A		
Tension steel					5559 mm ²		
$A_s = \frac{M}{(0.87 f_y) z}$					cl.3.2.7(2)		

[illegible]

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
				jXXX	9	
				Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.		
Member Design - RC Beam				Made by	XX	Date 16/9/2026
				Chd. 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, $\phi = 0.90$				0.90		
Ratio of A_s to $b_w d$, $\rho_w = 0.85f'_c / f_y \cdot [1 - \sqrt{1 - 2R_n / (0.85f'_c)}]$, $f_y \leq 550 \text{ N/mm}^2$				1.11	%	
Tension steel, $A_s = \text{MAX}(\rho_w b_w d, A_{s,\text{min}})$				5070	mm ²	
$A_{s,\text{min}} = \text{MAX}(0.25b_w d \sqrt{f'_c} / f_y, 1.4b_w d / f_y)$, $f_y \leq 550 \text{ N/mm}^2$				1284	mm ²	
Factor, $\beta_1 = \text{MAX}(\text{MIN}(0.85 - (0.05/7) \cdot (f'_c - 28), 0.85), 0.65)$				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, $a = A_s \cdot f_y / (0.85f'_c \cdot b_w)$, $f_y \leq 550 \text{ N/mm}^2$				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, $d_t = h - \text{cover} - \text{MAX}(\phi_{\text{link}}, \phi_{\text{link},tr}, \text{cover}_{\text{add},t}) - \phi_t / 2$				949	mm	
Note d_t is the distance from extreme compression fiber to centroid of extreme layer of longitudinal tension steel						
$\epsilon_t = 0.003(d_t / c - 1)$				0.008		
Note ϵ_t is the net tensile strain in extreme layer of longitudinal tension steel at nominal strength;						
$\epsilon_t > 0.005$ → no compression steel				VALID		
Tension steel, A_s				5070	mm ²	
$0.004 < \epsilon_t \leq 0.005$ → compression steel required				N/A		
Ratio of A_s to $b_w d$, $\rho_w = 0.319\beta_1 f'_c / f_y \cdot d_t / d$, $f_y \leq 550 \text{ N/mm}^2$				N/A	%	
$R_{nt} = \rho_w f_y (1 - 0.59\rho_w f_y / f'_c)$, $f_y \leq 550 \text{ N/mm}^2$				N/A	N/mm ²	
Note R_{nt} is the required nominal flexural strength coefficient of resistance for rectangular beam corresponding to $\epsilon_t = 0.005$;						
$M_{nt} = R_{nt} b_w d^2$				N/A	kNm	
Note M_{nt} is the required nominal flexural strength corresponding to $\epsilon_t = 0.005$;						
$M_n' = M^* / \phi - M_{nt}$				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, $f_s' = 0.003E_s(1 - d_t / c_t) \leq f_y$, $f_y \leq 550 \text{ N/mm}^2$				N/A	N/mm ²	
Modulus of elasticity of reinforcement, $E_s = 200,000$				N/A	N/mm ²	
Compression steel, $A_s' = M_n' / [f_s'(d - d_t)]$				N/A	mm ²	
Tension steel, $A_s = \rho_w b_w d + M_n' / [f_y(d - d_t)]$, $f_y \leq 550 \text{ N/mm}^2$				N/A	mm ²	
$\epsilon_t \leq 0.004$ → no valid solution				N/A		

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CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.																
				jXXX	11																	
				Member/Location																		
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.																		
Member Design - RC Beam				Made by	XX	Date 16/9/2026																
						Chd. BS8110																
Bending Flanged Beam (Singly or Doubly Reinforced)																						
<table><tr><td></td><td>Simply supported</td><td>Continuous</td><td>Cantilever</td><td rowspan="4">For cantilevers, width shown is applicable for downstand beams as rect- sections. For upstand beams, T- or L- sections will apply with insitu</td></tr><tr><td>T-Beam</td><td>$b_w + L / 5$</td><td>$b_w + L / 7.14$</td><td>b_w</td></tr><tr><td>L-Beam</td><td>$b_w + L / 10$</td><td>$b_w + L / 14.29$</td><td>b_w</td></tr><tr><td colspan="3">and $\leq$ (i) actual flange width, (ii) beam spacing</td></tr></table>								Simply supported	Continuous	Cantilever	For cantilevers, width shown is applicable for downstand beams as rect- sections. For upstand beams, T- or L- sections will apply with insitu	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 $\leq$ (i) actual flange width, (ii) beam spacing		
	Simply supported	Continuous	Cantilever	For cantilevers, width shown is applicable for downstand beams as rect- sections. For upstand beams, T- or L- sections will apply with insitu																		
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 $\leq$ (i) actual flange width, (ii) beam spacing																						
Span				N/A	m																	
Depth of flange, h_f				N/A	mm	N/A																
Effective width, $b = \text{MIN}(b_w + \text{function (span, section, structure), beam spacing})$ (bending flanged beam, deflection calcs flanged beam)				N/A	mm																	
$K = M/bd^2f_{cu}$				N/A																		
$z = d \cdot [0.5 + (0.25-K/0.9)^{0.5}] \leq 0.95d$				N/A	mm																	
Depth of compression stress block, $s = 2 \cdot (d-z)$ (applicable for all f_{cu})				N/A	mm																	
Compression Stress Block in Flange ($s \leq h_f$)				N/A																		
$M_u = K'f_{cu}bd^2$				N/A	kNm																	
Ratio, K'				N/A																		
$\begin{matrix} \leq 1.10 \\ \beta_b \\ 0.90 \end{matrix}$				$K' =$	$0.156 \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$	N/A mm 3.4.4.4 BC																
					$0.120 \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$	N/A mm 3.4.4.4 BC																
					$0.094 \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$ [$\beta_b = 1.0$]	N/A mm 3.4.4.4 BC																
$\begin{matrix} > 1.10 \\ \beta_b \\ 0.90 \end{matrix}$				$K' =$	$0.402(\beta_b - 0.4) - 0.18(\beta_b - 0.4)^2, \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$	N/A mm 3.4.4.4 BC																
					$0.357(\beta_b - 0.5) - 0.143(\beta_b - 0.5)^2 \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$	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: $M < M_u \rightarrow$ no compression steel				N/A																		
$K = M/bd^2f_{cu}$				N/A																		
To scheme beam, choose d such that $K = M/(bd^2f_{cu}) < K'$ (0.156 for $\beta_b = 1.0$ and NSC)																						
for no compression steel,				i.e. $d >$	N/A	and $h >$ N/A mm																
$z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\}$				$z \leq 0.95d$	N/A	mm																
Depth of neutral axis, $x \leq x_{limit}$				N/A	$\leq$ N/A mm	N/A																
Neutral axis, x		$x =$	$(d - z)/0.45, \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
			$(d - z)/0.40, \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
			$(d - z)/0.36, \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
Neutral axis limit, x_{limit}		$\begin{matrix} \leq 1.10 \\ \beta_b \\ 0.90 \end{matrix}$	$x \leq 0.5d \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
			$x \leq 0.4d \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
			$x \leq 0.33d, \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
			$\beta_b = 1.0$		N/A mm	3.4.4.4 BC																
			$x \leq (\beta_b - 0.4)d \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
			$x \leq (\beta_b - 0.5)d \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$		N/A mm	3.4.4.4 BC																
		$\begin{matrix} > 1.10 \\ \beta_b \\ 0.90 \end{matrix}$																				
Tension steel		$A_s = \frac{M}{(0.95f_y)z}, f_y \leq 460 \text{ N/mm}^2$		N/A	mm ²	Foreword																
Back-analysis of x in $(0.67f_{cu}/1.5) \cdot (s.b) = (f_y/1.05) \cdot (A_{s,prov}), f_y \leq 460$				N/A	mm	N/A																
$s =$		$0.9x \text{ for } f_{cu} \leq 60 \text{ N/mm}^2;$ $0.8x \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$ $0.72x \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$		N/A	mm	3.4.4.4 BC																
				N/A	mm	3.4.4.4 BC																
				N/A	mm	3.4.4.4 BC																
Note for an under reinforced section, require $x \leq x_{limit}$ for yielding of tension steel;																						

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CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
				jXXX	13	
				Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.		
Member Design - RC Beam				Made by	XX	Date
					16/9/2026	Chd.
				BS8110		
Compression Stress Block in Web ($s > h_f$ AND $h_f \leq \{0.45, 0.36, 0.30\}$)				N/A		
Note simplified method as equations within this section only valid for $\beta_b \geq 0.90$ and ≤ 1.10 as $x = 0.5d$;						
$h_f < 0.45d$ for $f_{cu} \leq 60$ N/mm ² ; or $h_f < 0.36d$ for $60 < f_{cu} \leq 75$ N/mm ² ; or $h_f < 0.30d$ for $75 < f_{cu} \leq 105$ N/mm ² and no moment redistribution.				cl.3.4.4.4 BC		
				cl.3.4.4.4 BC		
				cl.3.4.4.4 BC		
				cl.3.4.4.4 BC		
2	$M_{uf} = 0.45f_{cu}(b - b_w)h_f(d - 0.5h_f)$			N/A	kNm	
2	$K_f = \frac{M - M_{uf}}{f_{cu}b_w d^2}$			N/A		
2	Ratio, K'			N/A		
2	β_b	$K' =$	0.156 for $f_{cu} \leq 60$ N/mm ²	N/A	mm	3.4.4.4 BC
2	0.90		0.120 for $60 < f_{cu} \leq 75$ N/mm ²	N/A	mm	3.4.4.4 BC
2	1.10		0.094 for $75 < f_{cu} \leq 105$ N/mm ²	N/A	mm	3.4.4.4 BC
				[$\beta_b = 1.0$]		
If $K_f < K' \rightarrow$ no compression steel				N/A		
$A_s = \frac{M + k_1 f_{cu} b_w d (k_2 d - h_f)}{0.95 f_y (d - 0.5h_f)}$, $f_y \leq 460$ N/mm ²				N/A	mm ²	Foreword
				cl.3.4.4.5 BC		
$k_1 =$ 0.1 for $f_{cu} \leq 60$ N/mm ² , 0.072 for $60 < f_{cu} \leq 75$ N/mm ² and 0.054 for $75 < f_{cu} \leq 105$ N/mm ² ; and				$k_2 =$ 0.45 for $f_{cu} \leq 60$ N/mm ² , 0.32 for $60 < f_{cu} \leq 75$ N/mm ² and 0.24 for $75 < f_{cu} \leq 105$ N/mm ²		
				cl.3.4.4.5 BC		
				cl.3.4.4.5 BC		
				cl.3.4.4.5 BC		
2	Back-analysis of x in $(0.67f_{cu}/1.5) \cdot [b \cdot h_f + (s - h_f) \cdot b_w] = (f_y/1.05) \cdot (A_{s,D} - A_s)$			N/A	mm	N/A
2	$s =$ 0.9x for $f_{cu} \leq 60$ N/mm ² ; 0.8x for $60 < f_{cu} \leq 75$ N/mm ² ; 0.72x for $75 < f_{cu} \leq 105$ N/mm ²			N/A	mm	3.4.4.4 BC
2	$x_{limit} = 0.5d$			N/A	mm	3.4.4.4 BC
2	$x_{limit} = 0.5d$			N/A	mm	3.4.4.4 BC
Note for an under reinforced section, require $x \leq x_{limit}$ for yielding of tension steel;						
If $K_f > K' \rightarrow$ compression steel required				N/A		
Compression steel, $A_s' = [M - (K'f_{cu}b_w d^2 + M_{uf})]/[0.95f_y(d - d')]$, $f_y \leq 460$				N/A	mm ²	Foreword
Tension steel, $A_s = [\{0.20, 0.18, 0.16\}f_{cu}b_w d + 0.45f_{cu}h_f(b - b_w)]/(0.95f_y)$				N/A	mm ²	Foreword
Note the coefficient 0.20, 0.18 or 0.16 is used for $f_{cu} \leq 60, 75$ or 105 N/mm ² respectively;						
If $d' > \left(1 - \frac{f_y}{735}\right)x$, use $700\left(1 - \frac{d'}{x}\right)$ in lieu of $0.95f_y$				$x = 0.5d$, $f_y \leq 460$		
				Foreword		
Back-analysis of x in $(0.67f_{cu}/1.5) \cdot [b \cdot h_f + (s - h_f) \cdot b_w] + (f_y/1.05) \cdot (A_{s,D} - A_s)$				N/A	mm	N/A
				N/A	mm	3.4.4.4 BC
				N/A	mm	3.4.4.4 BC
				N/A	mm	3.4.4.4 BC
Note for an under reinforced section, require $\epsilon_{st} = \epsilon_{cc}(d - x)/x \geq \epsilon_y$ for yielding of tension steel						
and $\epsilon_{sc} = \epsilon_{cc}(x - d')/x \geq \epsilon_y$ for yielding of compression steel, where $\epsilon_{cc} = 0.0035$ for $f_{cu} \leq 60$ N/mm ²						
and $\epsilon_{cc} = 0.0035 - (f_{cu} - 60)/50000$ for $f_{cu} > 60$ N/mm ² and $\epsilon_y = (f_y/1.05)/E_s$;						

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CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers			Job No.	Sheet No.	Rev.																																																																																																				
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Member Design - RC Beam					Made by	XX	Date	16/9/2026	Chd.																																																																																																		
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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, $v_{ult} = V_d/b_wd$ ($< 0.8f_{cu}^{0.5}$ & $\{5.0, 7.0\}N/mm^2$)					2.55	N/mm ²	3.4.5.2 BC																																																																																																				
Ultimate shear 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 stress utilisation					54%		OK																																																																																																				
Design shear stress, $v_d = V_d/b_wd$					2.55	N/mm ²																																																																																																					
Enhanced shear strength, $2d/a_v.v_c$ ($< 0.8f_{cu}^{0.5}$ & $\{5.0, 7.0\}N/mm^2$ x 1.00					0.79	N/mm ²	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	mm ²																																																																																																					
$\rho_w = 100A_{s,prov}/b_wd$					1.40	%																																																																																																					
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d)^{1/4}$					0.79	N/mm ²	cl.3.4.5.4																																																																																																				
$\rho_w = 100A_{s,prov}/b_wd \leq 3$					1.40	%	cl.3.4.5.4																																																																																																				
$f_{cu} = f_{cu} \leq 80N/mm^2$					35	N/mm ²	3.4.5.4 BC																																																																																																				
$(400/d)^{1/4} \geq (0.67 \text{ or } 1.00)$					1.00		cl.3.4.5.4																																																																																																				
Table 3.8 — Values of v_c design concrete shear stress <table><tr><th rowspan="2">$\frac{100A_s}{b_v d}$</th><th colspan="8">Effective depth mm</th></tr><tr><th>125</th><th>150</th><th>175</th><th>200</th><th>225</th><th>250</th><th>300</th><th>400</th></tr><tr><td></td><td>N/mm²</td><td>N/mm²</td><td>N/mm²</td><td>N/mm²</td><td>N/mm²</td><td>N/mm²</td><td>N/mm²</td><td>N/mm²</td></tr><tr><td>≤ 0.15</td><td>0.45</td><td>0.43</td><td>0.41</td><td>0.40</td><td>0.39</td><td>0.38</td><td>0.36</td><td>0.34</td></tr><tr><td>0.25</td><td>0.53</td><td>0.51</td><td>0.49</td><td>0.47</td><td>0.46</td><td>0.45</td><td>0.43</td><td>0.40</td></tr><tr><td>0.50</td><td>0.67</td><td>0.64</td><td>0.62</td><td>0.60</td><td>0.58</td><td>0.56</td><td>0.54</td><td>0.50</td></tr><tr><td>0.75</td><td>0.77</td><td>0.73</td><td>0.71</td><td>0.68</td><td>0.66</td><td>0.65</td><td>0.62</td><td>0.57</td></tr><tr><td>1.00</td><td>0.84</td><td>0.81</td><td>0.78</td><td>0.75</td><td>0.73</td><td>0.71</td><td>0.68</td><td>0.63</td></tr><tr><td>1.50</td><td>0.97</td><td>0.92</td><td>0.89</td><td>0.86</td><td>0.83</td><td>0.81</td><td>0.78</td><td>0.72</td></tr><tr><td>2.00</td><td>1.06</td><td>1.02</td><td>0.98</td><td>0.95</td><td>0.92</td><td>0.89</td><td>0.86</td><td>0.80</td></tr><tr><td>≥ 3.00</td><td>1.22</td><td>1.16</td><td>1.12</td><td>1.08</td><td>1.05</td><td>1.02</td><td>0.98</td><td>0.91</td></tr></table> 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: $0.79(100A_s/(b_v d))^{1/3}(400/d)^{1/4}/\gamma_m$ where $\frac{100A_s}{b_v d}$ should not be taken as greater than 3. $(\frac{400}{d})^{1/4}$ should not be taken as less than 0.67 for members without shear reinforcement. $(\frac{400}{d})^{1/4}$ should not be taken as less than 1 for members with shear reinforcement providing a design shear resistance of $\geq 0.4 N/mm^2$. 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.										$\frac{100A_s}{b_v d}$	Effective depth mm								125	150	175	200	225	250	300	400		N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	≤ 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
$\frac{100A_s}{b_v d}$	Effective depth mm																																																																																																										
	125	150	175	200	225	250	300	400																																																																																																			
	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²	N/mm ²																																																																																																			
≤ 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																																																																																																			
Minimum shear strength, $v_r = MAX(0.4, 0.4(f_{cu}/40)^{2/3})$, $f_{cu} \leq 80N/mm^2$					0.40	N/mm ²	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																																																																																																			
$(A_{sv}/S)_{nom} > v_r.b_w/(0.95f_{yv})$, $f_{yv} \leq 460N/mm^2$ i.e. $(A_{sv}/S)_{nom} >$					0.46	mm ² /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																																																																																																			
$A_{sv}/S > b_w(v_d - 2d/a_v.v_c)/(0.95f_{yv})$, $f_{yv} \leq 460N/mm^2$ i.e. $A_{sv}/S >$					2.01	mm ² /mm	cl.3.4.5.8,																																																																																																				
$V_{cap} = (A_{sv,prov}/S + A_{sv,prov,t}/S_t).(0.95f_{yv}).d + 2d/a_v.v_c.(b_wd)$, $f_{yv} \leq 460$					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	mm ²																																																																																																					
Tried $A_{sv,prov}/S + A_{sv,prov,t}/S_t$ value					3.14	mm ² /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 $2d/a_v.v_c$;																																																																																																											

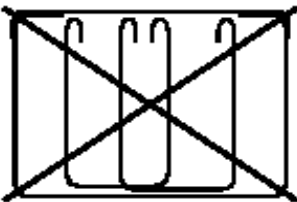
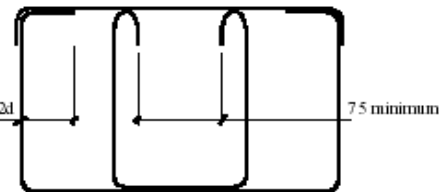
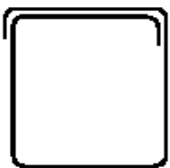
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.			Rev.																																																																																																			
						jXXX	16																																																																																																						
						Member/Location																																																																																																							
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.																																																																																																							
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.																																																																																																			
						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, $v_{ult} = V_d/b_w d$ ($< v_{Rd,max}(\theta=45^\circ)=0.18(1-f_{ck}/250).f_{ck}$, $f_{ck} \leq 50$ N/mm ²)						2.55	N/mm ²	3(3), cl.3.1																																																																																																					
Ultimate shear strength, $v_{Rd,max}(\theta=45^\circ)=0.18(1-f_{ck}/250).f_{ck}$, $f_{ck} \leq 50$ N/mm ²						4.48	N/mm ²	3(3), cl.3.1																																																																																																					
Ultimate shear stress utilisation						57%		OK																																																																																																					
Design shear stress, $v_d = V_d/b_w d$						2.55	N/mm ²																																																																																																						
Enhanced shear strength, $2d/a_v \cdot v_{Rd,c}$ ($< v_{Rd,max}(\theta=45^\circ)=0.18(1-f_{ck}/2) \times 1.00$)						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																																																																																																						
$\rho_w = 100A_{s,prov}/b_w d$						1.40	%																																																																																																						
$v_{Rd,c} = C_{Rd,c} \cdot k(\rho_w f_{ck})^{1/3} + k_1 \cdot \sigma_{cp} \geq v_{min} + k_1 \cdot \sigma_{cp}$; $v_{min} = 0.035k^{3/2} f_{ck}^{0.5}$, $k = 1 + \sqrt{200/d} \leq 2.0$						0.60	N/mm ²	2(1), cl.3.1																																																																																																					
$C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$						0.12		cl.6.2.2(1)																																																																																																					
$k = 1 + \sqrt{200/d} \leq 2.0$						1.47		cl.6.2.2(1)																																																																																																					
$\rho_w = 100A_{s,prov}/b_w d \leq 2$						1.40	%	cl.6.2.2(1)																																																																																																					
$\sigma_{cp} = F/b_w h \leq 0.2f_{cd} = 0.2\alpha_{cc} \cdot f_{ck}/\gamma_c = 0.2(1.0 \cdot f_{ck}/1.5)$, $f_{ck} \leq 50$ N/mm ²						0.00	N/mm ²	2(1), cl.3.1																																																																																																					
Table 8.2 Shear resistance of slabs without shear reinforcement $v_{Rd,c}$ N/mm ² (Class C30/37 concrete)																																																																																																													
<table border="1"> <tr> <th rowspan="2">$\rho_1 = A_s/bd$</th> <th colspan="9">Effective depth, d (mm)</th> </tr> <tr> <th>≤ 200</th> <th>225</th> <th>250</th> <th>300</th> <th>350</th> <th>400</th> <th>500</th> <th>600</th> <th>750</th> </tr> <tr> <td>0.25%</td> <td>0.54</td> <td>0.52</td> <td>0.50</td> <td>0.47</td> <td>0.45</td> <td>0.43</td> <td>0.40</td> <td>0.38</td> <td>0.36</td> </tr> <tr> <td>0.50%</td> <td>0.59</td> <td>0.57</td> <td>0.56</td> <td>0.54</td> <td>0.52</td> <td>0.51</td> <td>0.48</td> <td>0.47</td> <td>0.45</td> </tr> <tr> <td>0.75%</td> <td>0.68</td> <td>0.66</td> <td>0.64</td> <td>0.62</td> <td>0.59</td> <td>0.58</td> <td>0.55</td> <td>0.53</td> <td>0.51</td> </tr> <tr> <td>1.00%</td> <td>0.75</td> <td>0.72</td> <td>0.71</td> <td>0.68</td> <td>0.65</td> <td>0.64</td> <td>0.61</td> <td>0.59</td> <td>0.57</td> </tr> <tr> <td>1.25%</td> <td>0.80</td> <td>0.78</td> <td>0.76</td> <td>0.73</td> <td>0.71</td> <td>0.69</td> <td>0.66</td> <td>0.63</td> <td>0.61</td> </tr> <tr> <td>1.50%</td> <td>0.85</td> <td>0.83</td> <td>0.81</td> <td>0.78</td> <td>0.75</td> <td>0.73</td> <td>0.70</td> <td>0.67</td> <td>0.65</td> </tr> <tr> <td>2.00%</td> <td>0.94</td> <td>0.91</td> <td>0.89</td> <td>0.85</td> <td>0.82</td> <td>0.80</td> <td>0.77</td> <td>0.74</td> <td>0.71</td> </tr> <tr> <td>k</td> <td>2.000</td> <td>1.943</td> <td>1.894</td> <td>1.816</td> <td>1.756</td> <td>1.707</td> <td>1.632</td> <td>1.577</td> <td>1.516</td> </tr> </table>											$\rho_1 = A_s/bd$	Effective depth, d (mm)									≤ 200	225	250	300	350	400	500	600	750	0.25%	0.54	0.52	0.50	0.47	0.45	0.43	0.40	0.38	0.36	0.50%	0.59	0.57	0.56	0.54	0.52	0.51	0.48	0.47	0.45	0.75%	0.68	0.66	0.64	0.62	0.59	0.58	0.55	0.53	0.51	1.00%	0.75	0.72	0.71	0.68	0.65	0.64	0.61	0.59	0.57	1.25%	0.80	0.78	0.76	0.73	0.71	0.69	0.66	0.63	0.61	1.50%	0.85	0.83	0.81	0.78	0.75	0.73	0.70	0.67	0.65	2.00%	0.94	0.91	0.89	0.85	0.82	0.80	0.77	0.74	0.71	k	2.000	1.943	1.894	1.816	1.756	1.707	1.632	1.577	1.516
$\rho_1 = A_s/bd$	Effective depth, d (mm)																																																																																																												
	≤ 200	225	250	300	350	400	500	600	750																																																																																																				
0.25%	0.54	0.52	0.50	0.47	0.45	0.43	0.40	0.38	0.36																																																																																																				
0.50%	0.59	0.57	0.56	0.54	0.52	0.51	0.48	0.47	0.45																																																																																																				
0.75%	0.68	0.66	0.64	0.62	0.59	0.58	0.55	0.53	0.51																																																																																																				
1.00%	0.75	0.72	0.71	0.68	0.65	0.64	0.61	0.59	0.57																																																																																																				
1.25%	0.80	0.78	0.76	0.73	0.71	0.69	0.66	0.63	0.61																																																																																																				
1.50%	0.85	0.83	0.81	0.78	0.75	0.73	0.70	0.67	0.65																																																																																																				
2.00%	0.94	0.91	0.89	0.85	0.82	0.80	0.77	0.74	0.71																																																																																																				
k	2.000	1.943	1.894	1.816	1.756	1.707	1.632	1.577	1.516																																																																																																				
Table 8.3 Concrete strength modification factor																																																																																																													
<table border="1"> <tr> <th>f_{ck} (N/mm²)</th> <th>25</th> <th>30</th> <th>35</th> <th>40</th> <th>45</th> <th>50</th> </tr> <tr> <td>Modification factor</td> <td>0.94</td> <td>1.00</td> <td>1.05</td> <td>1.10</td> <td>1.14</td> <td>1.19</td> </tr> </table>											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																																																																																					
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, $v_r = 0.08/\gamma_s \cdot f_{ck}^{0.5}$, $\gamma_s = 1.15$, $f_{ck} \leq 50$ N/mm ²						0.37	N/mm ²	2(5), cl.3.1																																																																																																					
Check $v_d < 2d/a_v \cdot v_{Rd,c}$ for no links (minor elements)						INVALID	0.60	cl.6.2.1(3)																																																																																																					
4.5.8	Concrete shear capacity $2d/a_v \cdot v_{Rd,c} \cdot (b_w d)$					274	kN	2(1), cl.6.2.3(8)																																																																																																					
Check $0.0 < v_d < v_r$ for nominal links						0.00	N/A	0.37	cl.6.2.1(5)																																																																																																				
4.5.1	$(A_{sv}/S)_{nom} > v_r \cdot b_w / (0.87f_{yv})$, $f_{yv} \leq 600$ N/mm ² i.e. $(A_{sv}/S)_{nom} >$					0.42	mm ² /mm	3(3), cl.3.2																																																																																																					
4.5.8	$V_{cap,nom} = 2d/a_v \cdot (A_{sv}/S)_{nom} \cdot (0.87f_{yv}) \cdot 0.9d \cdot \cot\theta$, $f_{yv} \leq 600$ N/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	$A_{sv}/S > b_w \cdot v_d / (0.9(0.87f_{yv}) \cdot \cot\theta) / (2d/a_v)$, $f_{yv} \leq 600$ N/mm ² i.e. $A_{sv}/S >$					1.32	mm ² /mm	cl.6.2.3(8)																																																																																																					
cl.3.4.5.1	$V_{cap} = 2d/a_v \cdot (A_{sv,prov}/S + A_{sv,prov,t}/S_t) \cdot (0.87f_{yv}) \cdot 0.9d \cdot \cot\theta$, $f_{yv} \leq 600$ N/mm ²					2792	kN	cl.6.2.3(8)																																																																																																					
Comp. strut angle, $\theta = 22^\circ \leq 0.5\sin^{-1}\{v_d/v_{Rd,max}(\theta=45^\circ)\} \leq 45^\circ$						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 $2d/a_v \cdot v_{Rd,c}$;																																																																																																													

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.										
				jXXX	17											
				Member/Location												
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.												
Member Design - RC Beam				Made by	XX	Date 16/9/2026										
						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, $v_{ult} = V_d^*/b_wd$ ($< \phi(0.66+0.17\lambda)\sqrt{f'_c}$, $f'_c \leq 70\text{N/mm}^2$)				2.55	N/mm ²	cl.22.5.5.1										
Ultimate shear strength, $\phi(0.66+0.17\lambda)\sqrt{f'_c}$, $f'_c \leq 70\text{N/mm}^2$				3.29	N/mm ²	cl.22.5.5.1										
Ultimate shear stress utilisation				77%		OK										
Design shear stress, $v_d = V_d^*/b_wd$				2.55	N/mm ²											
Enhanced shear strength, $d/a_v.v_c$ ($< \phi(0.66+0.17\lambda)\sqrt{f'_c}$, $f'_c \leq 70\text{N/mm}^2$) $\times 1.00$				0.74	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											
$\rho_w = 100A_{s,prov}/b_wd$				1.40	%											
9.2(2)	$v_c = \phi V_c/b_wd$		Note $f'_c \leq 70\text{N/mm}^2$	Detailed	0.74	N/mm ²										
		$V_c = 0.17\lambda\sqrt{f'_c}b_wd$	[Simplified] $\phi V_c =$	309	kN	5.5.1, cl.22										
9.2(2)	where $\lambda = \{1.00 \text{ NWC}, 0.75 \text{ LWC}\}$		Normal weight													
Table 22.5.5.1—Detailed method for calculating V_c <table><tr><td colspan="2">V_c</td><td></td></tr><tr><td rowspan="3">Least of (a), (b), and (c):</td><td>$\left(0.16\lambda\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right)b_wd$ ⁽¹⁾</td><td>(a)</td></tr><tr><td>$(0.16\lambda\sqrt{f'_c} + 17\rho_w)b_wd$</td><td>(b)</td></tr><tr><td>$0.29\lambda\sqrt{f'_c}b_wd$</td><td>(c)</td></tr></table> ⁽¹⁾ M_u occurs simultaneously with V_u at the section considered.							V_c			Least of (a), (b), and (c):	$\left(0.16\lambda\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right)b_wd$ ⁽¹⁾	(a)	$(0.16\lambda\sqrt{f'_c} + 17\rho_w)b_wd$	(b)	$0.29\lambda\sqrt{f'_c}b_wd$	(c)
V_c																
Least of (a), (b), and (c):	$\left(0.16\lambda\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right)b_wd$ ⁽¹⁾	(a)														
	$(0.16\lambda\sqrt{f'_c} + 17\rho_w)b_wd$	(b)														
	$0.29\lambda\sqrt{f'_c}b_wd$	(c)														
where $\lambda = \{1.00 \text{ NWC}, 0.75 \text{ LWC}\}$				Normal weight												
Strength reduction factor for shear, $\phi = 0.75$				0.75		cl.21.2										
Minimum shear strength, $v_r = \text{MAX} (0.35, 0.062\sqrt{f'_c})$, $f'_c \leq 70\text{N/mm}^2$				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	0.37	cl.9.6.3.1										
9.2(6)	Concrete shear capacity $d/a_v.v_c.(b_wd)$			339	kN	5.5.1, cl.9.										
Check $0.0 < v_d < \phi v_r + d/a_v.v_c$ for nominal links				0.00	N/A	1.00										
9.2(3)	$(A_{sv}/S)_{nom} > v_r.b_w/f_{yv}$, $f_{yv} \leq 420\text{N/mm}^2$ i.e. $(A_{sv}/S)_{nom} >$			0.42	mm ² /mm	5.10.5, cl.2										
cl.3.2.2(3) $V_{cap,nom} = (\phi d/a_v.v_r + d/a_v.v_c).(b_wd)$					459	kN										
Check $v_d > \phi v_r + d/a_v.v_c$ for design links				1.00	VALID	3.29										
cl.3.2.2(3) $A_{sv}/S > b_w(v_d - d/a_v.v_c)/(\phi f_{yv})/(d/a_v)$, $f_{yv} \leq 420\text{N/mm}^2$ i.e. $A_{sv}/S >$					2.88	mm ² /mm										
cl.3.2.2(3) $V_{cap} = \phi d/a_v.(A_{sv,prov}/S + A_{sv,prov,t}/S_t).f_{yv}.d + d/a_v.v_c.(b_wd)$, $f_{yv} \leq 420\text{N/mm}^2$					1246	kN										
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				94%		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 $d/a_v.v_c$;																

[illegible]

[illegible]

[illegible]

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	21			
						Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.				
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.
						BS8110				
Detailing Requirements Rectangular or Flanged Beam										
All detailing requirements met ?						OK				
Min tension steel reinforcement diameter, ϕ_t ($\geq 12\text{mm}$)						32		mm		OK
Min tension steel reinforcement pitch ($b_w - 2 \cdot \text{cover} - 2 \cdot \text{MAX}(\phi_{\text{link}}, \phi_{\text{link},t}) - \phi_t$) / (n_t/n)						133		mm		OK
Max tension steel reinforcement pitch ($b_w - 2 \cdot \text{cover} - 2 \cdot \text{MAX}(\phi_{\text{link}}, \phi_{\text{link},t}) - \phi_t$) / (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 ($\geq 12\text{mm}$)						N/A		mm		N/A
Min compression steel reinforcement pitch (b_w or $b - 2 \cdot \text{cover} - 2 \cdot \text{MAX}(\phi_{\text{link}}, \phi_{\text{link},t})$)						N/A		mm		N/A
Max compression steel reinforcement pitch (b_w or $b - 2 \cdot \text{cover} - 2 \cdot \text{MAX}(\phi_{\text{link}}, \phi_{\text{link},t})$)						N/A		mm		N/A
Min shear link diameter, ϕ_{link} ($\geq 6\text{mm}$, $\geq \phi_c/4$)						10		mm		OK
Min torsion link diameter, $\phi_{\text{link},t}$ ($\geq 6\text{mm}$, $\geq \phi_c/4$)						N/A		mm		N/A
Shear link pitch, S ($\leq 0.75d$, $\leq 12\phi_c$, $\leq 300\text{mm}$, $\geq \text{MAX}(100\text{mm}, 50 + 12 \cdot \phi_c)$)						100		mm		OK
Torsion link pitch, S_t ($\leq 0.75d$, $\leq 12\phi_c$, $\leq 300\text{mm}$, $\geq \text{MAX}(100\text{mm}, 50 + 12 \cdot \phi_c)$)						N/A		mm		N/A
$A_{sv,prov} / (b_w \cdot S) + A_{sv,prov,t} / (b_w \cdot S_t)$ ($> 0.10\%$ G460; $> 0.17\%$ G250)						0.63		%		OK
$A_{sv,prov} / (b_w \cdot S) + A_{sv,prov,t} / (b_w \cdot S_t)$ ($< 4.00\%$)						0.63		%		OK
Note require an overall enclosing link; Note require additional restraining links for each alternate longitudinal										
										
5.3.1, A pattern which overlaps links as shown below should not be used.										
										
Open links may be used for beam and slab construction using 'L' hooks where the width of rib is 450mm or more. In such circumstance a top locking link is also used.										
										
Where links are used for torsion the bar shape 99 should be used to describe the shape shown below.										
										
Note lacer bars of 16mm are required at the sides of beams more than 750mm deep at 250mm pitch;										

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.										
				jXXX	22											
				Member/Location												
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.												
Member Design - RC Beam				Made by	XX	Date										
					16/9/2026	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 multipliers (C ₁): C ₁ = 0.8 for flanged beams with b _w /b < 0.3 C ₁ = 10/span(m) for spans beyond 10m C ₁ = 0.9 for flat slabs (use longer span)																
NOTE: For two-way slabs on continuous support, use shorter span.																
<table><tr><th colspan="2">Basic span/effective depth ratios for rectangular beams</th></tr><tr><th>Support conditions</th><th>Rectangular sections</th></tr><tr><td>Cantilever</td><td>7</td></tr><tr><td>Simple supported</td><td>20</td></tr><tr><td>Continuous</td><td>26</td></tr></table>							Basic span/effective depth ratios for rectangular beams		Support conditions	Rectangular sections	Cantilever	7	Simple supported	20	Continuous	26
Basic span/effective depth ratios for rectangular beams																
Support conditions	Rectangular sections															
Cantilever	7															
Simple supported	20															
Continuous	26															
Tension reinforcement modification factor (C₂)⁴ f _s = service stress in reinforcement																
Compression reinforcement modification factor (C₃)																
Shrinkage For normal situations, assume long term shrinkage strain of 300 x 10 ⁻⁶																
Creep For normal situations, assume creep coefficient of ϕ = 2																
Hence long term E value: E = $\frac{E_{28}}{1+\phi}$																

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.			Rev.	
						jXXX	23				
						Member/Location					
Job Title		Member Design - Reinforced Concrete Beam BS8110,				Drg. Ref.					
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.	
						BS8110					
Span						10.000 m					
Span / effective depth ratio						10.9					
Basic span / effective depth ratio criteria						26.0					
Multiplier C _{1,rect} or flanged						1.00 cl.3.4.6.3					
Multiplier C _{1,span} more or less than 10m						Include	▼	1.00		cl.3.4.6.4	
Modification factor for tension C ₂											
M/(b _w or b).d ²						4.40		N/mm ²		cl.3.4.6.2	
$f_s = \frac{2f_{y,A_s\text{ req}}}{3A_{s\text{ prov}}} \times \frac{1}{\beta_b}$						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 or b).d						N/A				cl.3.4.6.2	
Modification						$1 + \frac{100A'_{s\text{ prov}}}{bd} \bigg/ \left(3 + \frac{100A'_{s\text{ prov}}}{bd}\right) \leq 1.5$		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:

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

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:

$$f_s = \frac{2f_{y,A_s\text{ req}}}{3A_{s\text{ prov}}} \times \frac{1}{\beta_b}$$

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 2/3f_y.

Table 3.11 — Modification factor for compression reinforcement

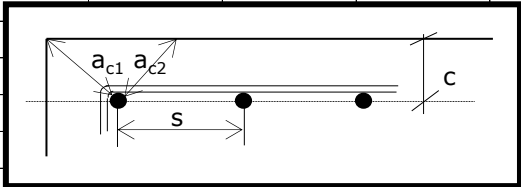
$100 \frac{A'_{s\text{ prov}}}{bd}$	Factor
0.00	1.00
0.15	1.05
0.25	1.08
0.35	1.10
0.50	1.14
0.75	1.20
1.0	1.25
1.5	1.33
2.0	1.40
2.5	1.45
≥ 3.0	1.50

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

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

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.

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.		
				jXXX	24			
				Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.				
Member Design - RC Beam				Made by	XX	Date	16/9/2026	Chd.
								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, $\rho_w = A_{s,prov}/b_w d$						0.01		
Factor, $\alpha_e \cdot \rho_w$						0.21		
Depth of neutral axis, $x = \alpha_e \cdot \rho_w \cdot [(1+2/(\alpha_e \rho_w))^{1/2} - 1] \cdot d$						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 $f_{s2} = \alpha_e \cdot (A_{s,prov} + A_{s,prov}')/[\alpha_e \cdot (A_{s,prov} + A_{s,prov}') + (b_w \cdot h - A_{s,prov} - A_{s,prov}')] \cdot F_{sls}/(A_{s,prov} + A_{s,prov}')$;								
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 $F_{sls} > 0$, $f_c = 2(M_{sls}/z)/(b_w x)$, else $f_c = 2(M_{sls}/z)/(b_w x) + (b_w \cdot h - A_{s,prov} - A_{s,prov}')/[\alpha_e \cdot (A_{s,prov} + A_{s,prov}') + (b_w \cdot h - A_{s,prov} - A_{s,prov}')] \cdot ABS(F_{sls})/(b_w \cdot h - A_{s,prov} - A_{s,prov}')$;								
Concrete compressive service stress utilisation, $f_c / (0.45f_{cu}) < 1.00$						101%		NOT OK
Strain at tension face, $\epsilon_1 = (h-x)/(d-x) \cdot (f_{s1} + f_{s2})/E_s$						1.55		x10 ⁻³
Strain stiffening, $\epsilon_2 = b_w(h-x)^2/[3E_s A_{s,prov}(d-x)]$						0.09		x10 ⁻³
Mean strain, $\epsilon_m = \epsilon_1 - \epsilon_2$						1.47		x10 ⁻³
Distance to face of extreme rebar, $c_f = \text{cover} + \text{MAX}(\phi_{link}, \phi_{link,t})$						35		mm
Distance to centroid of extreme rebar, $c = c_f + \phi_t/2$						51		mm
Distance, $s = (b_w - 2 \cdot \text{cover} - 2 \cdot \text{MAX}(\phi_{link}, \phi_{link,t}) - \phi_t)/(n_t/n_{layers,tens} - 1)$						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, $a_{cr} = \text{MAX}(a_{c1}, a_{c2})$						68		mm
Note a_{c1} is not applicable for continuous width sections, i.e. slabs or walls;								
								
Maximum crack width, $w_{max} = 3a_{cr}\epsilon_m/[1+2 \cdot (a_{cr}-c_f)/(h-x)]$						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

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
				jXXX	25	
				Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,			Drg. Ref.		
Member Design - RC Beam				Made by	XX	Date 16/9/2026
						Chd. EC2
Longitudinal Shear Between Web and Flange Rectangular or Flanged Beam (EC2)						
<i>Note that this check is performed for both rectangular and flanged section designs, although theoretically only applicable in the latter case;</i>						
Longitudinal shear stress, $K_S \cdot V_{Ed}$				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 , $\Delta F_d = K_B \cdot (M^* - 0) / z$				606	kN	
<i>Note conservatively factor, $K_B = 0.5(b_{eff} - b_w) / b_{eff}$ employed even if neutral axis within web;</i>						
Lever arm, z				0.763	m	
<i>Note if neutral axis within web, for simplicity, $z = d - h_f / 2$;</i>						
Thickness of the flange at the junctions, h_f				200	mm	
Length under consideration, Δx				2500	mm	
<i>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 $\sim M/2 - 0$ 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.</i>						cl.6.2.4
Shear stress distribution factor, K_S				1.33		
<i>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;</i>						
Effective width, $b_{eff} = \text{MIN}(b_w + \text{function (span, section, structure)})$				1000	mm	
<i>Note for rectangular sections, b_{eff} equivalent to that of T-sections assumed;</i>						
Width (rectangular) or web width (flanged), b_w				500	mm	
Longitudinal shear stress limit to prevent crushing,				$v_{cd} \sin \theta_f \cos \theta_f$	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 $\alpha_{cc} = 1.0, \gamma_c = 1.5$				cl.3.1.6
Strength reduction factor for concrete cracked in shear, v				0.533		
$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$						cl.6.2.2
Longitudinal shear stress limit to prevent crushing utilisation, $(K_S \cdot v_{Ed}) / (v_{cd} \sin \theta_f \cos \theta_f)$				32%		OK
Longitudinal shear stress limit for no transverse reinforcement, $0.4 f_{ctd}$				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 $\alpha_{ct} = 1.0, \gamma_c = 1.5$				cl.3.1.6
$f_{ctk,0.05} = 0.7 \times f_{ctm}$				1.94	N/mm ²	T.3.1
$f_{ctm} = 0.30 \times f_{ck}^{(2/3)} \leq C50/60$		$f_{ctm} = 2.12 \cdot \ln(1 + (f_{cm}/10)) > C50/60$		2.77	N/mm ²	T.3.1
$f_{cm} = f_{ck} + 8 \text{ (MPa)}$				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, $(K_S \cdot v_{Ed}) / (v_{cd} \sin \theta_f \cos \theta_f)$				312%		NOT OK
Required design transverse reinforcement per unit length, $A_{sf}/s_f >$				741	mm ² /m	
$(A_{sf} f_{yd} / s_f) \geq v_{Ed} \cdot h_f / \cot \theta_f$						
<i>Note area of transverse steel to be provided should be the greater of $1.0 A_{sf} / s_f$ and $0.5 A_{sf} / s_f + \text{area required for slab bending}$; Note K_S factored onto v_{Ed} herein;</i>						cl.6.2.4
Design yield strength of reinforcement, $f_{yd} = f_y / \gamma_s, \gamma_s = 1.15, f_y \leq 60$				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	
$1.0 \leq \cot \theta_f \leq 2.0$ for compression flanges ($45^\circ \geq \theta_f \geq 26.5^\circ$) $1.0 \leq \cot \theta_f \leq 1.25$ for tension flanges ($45^\circ \geq \theta_f \geq 38.6^\circ$)						cl.6.2.4
Provided transverse reinforcement per unit length, A_e				785	mm ² /m	
Required design transverse reinforcement per unit length utilisation, $(A_{sf}/s_f) / (A_e/s_f)$				94%		OK

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.	
						jXXX	26			
						Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.				
Member Design - RC Beam						Made by	XX	Date	16/9/2026	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 , $\Delta F_d = K_B \cdot (M^* - 0) / z$						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, $b_{eff} = \text{MIN}(b_w + \text{function (span, section, structure)})$						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, $V_{1,limit}$										
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, $f_y \leq 460 \text{N/mm}^2$					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			

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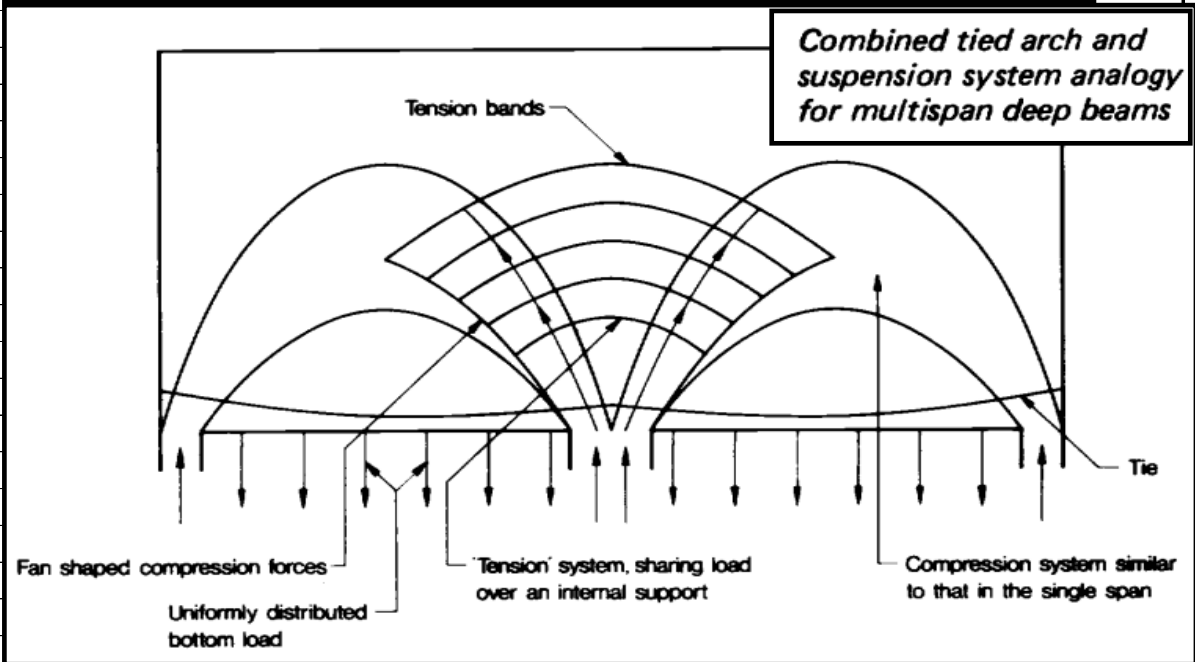
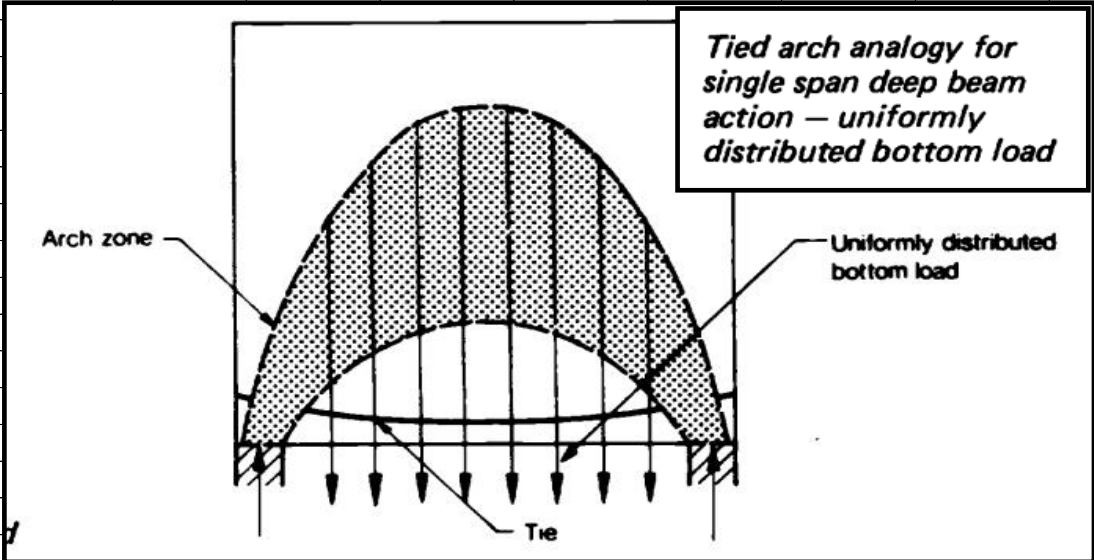
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CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.	
						jXXX	32			
						Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.				
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.
									BS8110	
Deep Beam Bending									CIRIA	
									Guide 2	
Design bending moment, M							N/A	kNm		
Note the ultimate bending moment limit is $0.12f_{cu}b_w h^2$;									cl.2.4.1	
Ultimate bending moment utilisation							N/A		N/A	
Tension steel (deep beam), $A_{s,db} = M / [0.95f_y z]$, $f_y \leq 460 \text{ N/mm}^2$							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, $z = 0.2 \times \text{span}_{lim} + 0.4h$						N/A	mm	cl.2.4.1	
LF	Continuous, $z = 0.2 \times \text{span}_{lim} + 0.3h$						N/A	mm	cl.2.4.1	
	Cantilever, $z = 0.4 \times \text{span}_{lim} + 0.4h$						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 $0.5(\text{span}_{lim}/h-1)A_{s,db}$, $0.2h-0.8h$ 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 $\min\{b_w \cdot h \cdot v_u, 2b_w \cdot h^2 v_c k_s / x_e\}$ where v_u is the ultimate concrete shear strength from CP 110 T.6 and T.26 replaced by $\min\{0.8f_{cu}^{0.5}, \{5.0, 7.0\} \text{ N/mm}^2\}$ and v_c is the design concrete shear strength from CP 110 T.5 and T.25 replaced by v_c ;									cl.2.4.2	
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 $\{\text{span}/4 \text{ s/s and cont, span}/2 \text{ cant}\}$ 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, $\lambda_1 = \{0.44 \text{ NWC, } 0.32 \text{ LWC}\}$							N/A		cl.3.4.2	
Empirical coefficient, $\lambda_2 = \{0.85 \text{ plain round bars, } 1.95 \text{ deformed bars}\}$							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, $V_r = 100\lambda_2 \Sigma A_{sv,prov,h} \cdot \gamma_r \cdot \sin^2 \theta / h$							N/A	kN	cl.3.4.2	
Note the summation of the depths at which the horizontal shear links intersect the diagonal crack, $\Sigma \gamma_r$ is calculated as $S_h \cdot n \cdot (n+1)/2$ 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, $V (\leq 1.3\lambda_1 \sqrt{f_{cu}} \cdot b_w \cdot h)$							N/A	kN	cl.3.4.2	
Note $V = \text{MAX}[0, \lambda_1 \cdot (h - 0.35x_e) \cdot \sqrt{f_{cu}} \cdot b_w] + 100\lambda_2 \cdot A_{s,prov} \cdot d \cdot \sin^2 \theta / h$;									cl.3.4.2	
Note require $[100\lambda_2 \cdot A_{s,prov} \cdot d \cdot \sin^2 \theta / h] / V \geq 0.20$;							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 $V_r + V (\leq \lambda_1 \cdot \sqrt{f_{cu}} \cdot b_w \cdot h)$							N/A	kN	cl.3.4.2	
Note $v_{max} = 1.3 \sqrt{f_{cu}}$;									T.5	
Note require $[V_r + 100\lambda_2 \cdot A_{s,prov} \cdot d \cdot \sin^2 \theta / h] / [V_r + V] \geq 0.20$;							N/A		N/A	
Design shear resistance (deep beam) utilisation							N/A		N/A	

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.	
						jXXX	33			
						Member/Location				
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.				
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.

BS8110

Detailing Instructions (Deep Beam)



CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	34	
		Member/Location		
Job Title	Member Design - Reinforced Concrete Beam BS8110,	Drg. Ref.		
Member Design - RC Beam		Made by	XX	Date
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				Chd.
				BS8110

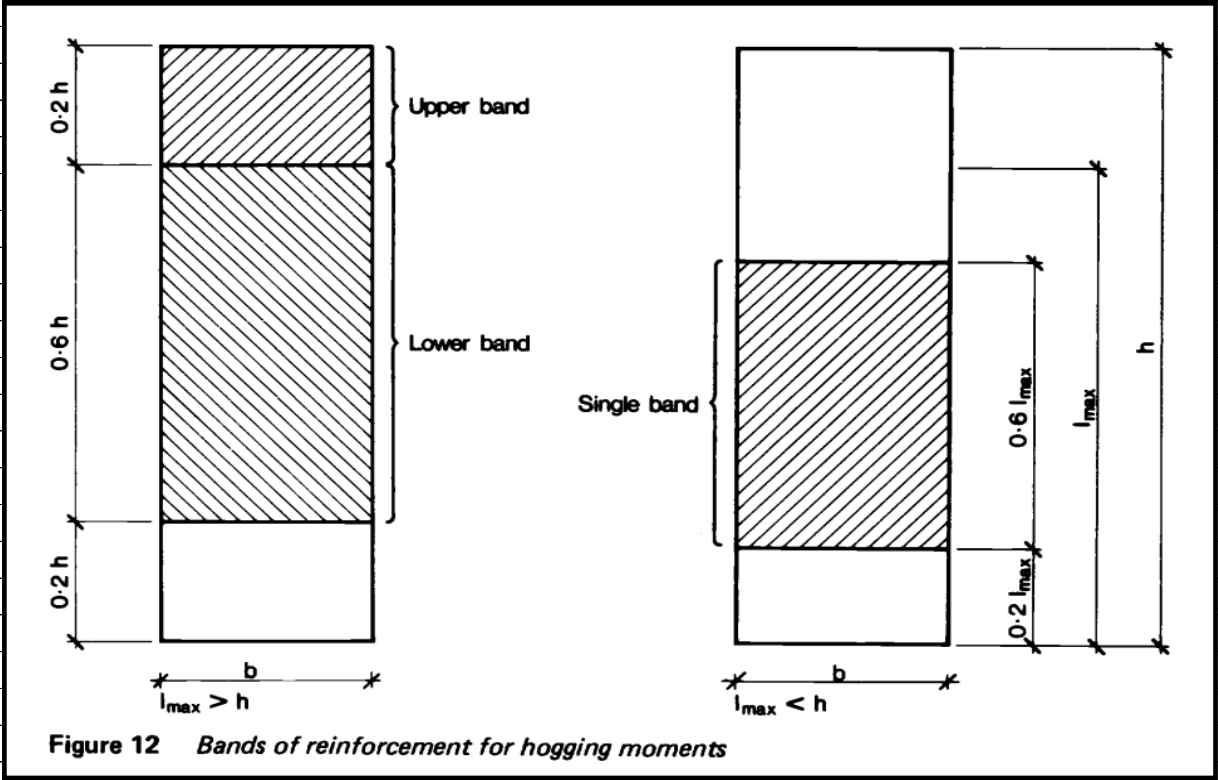
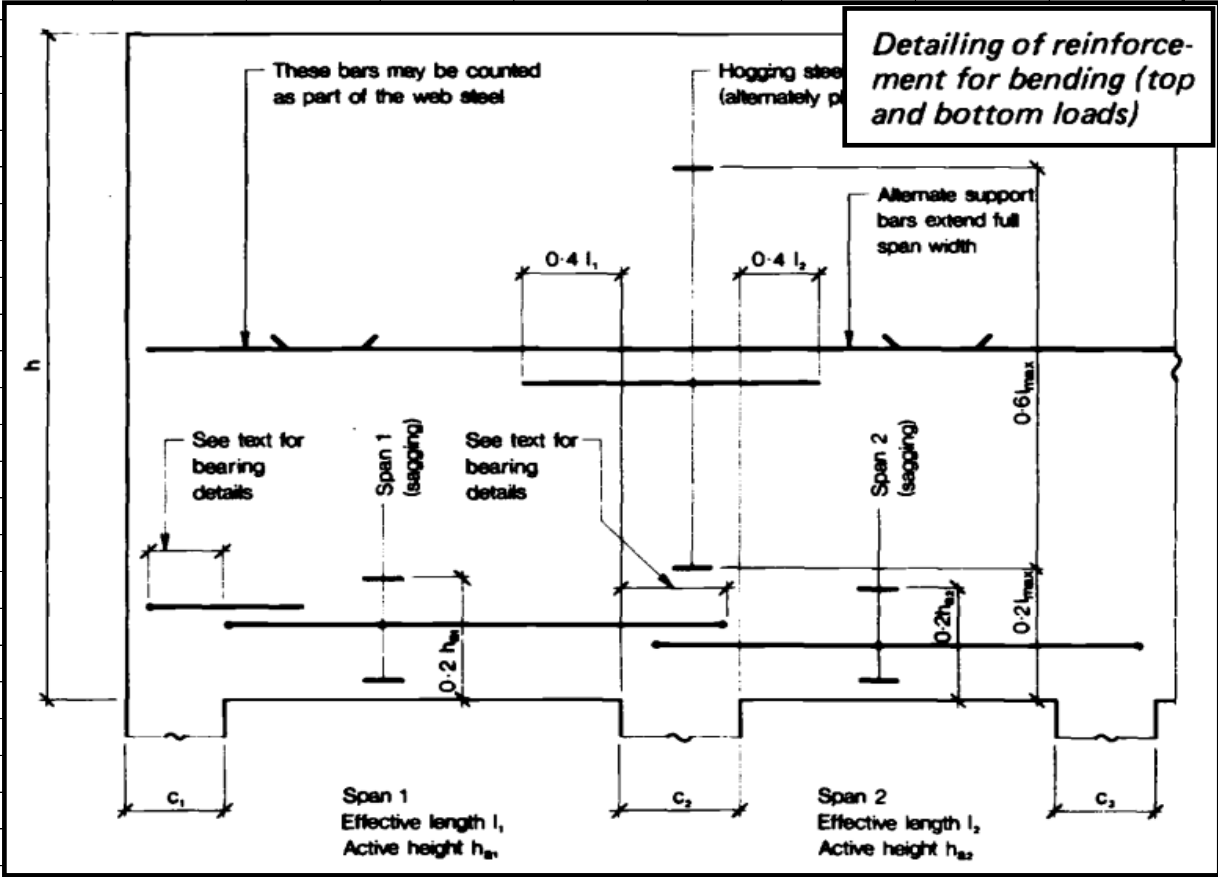
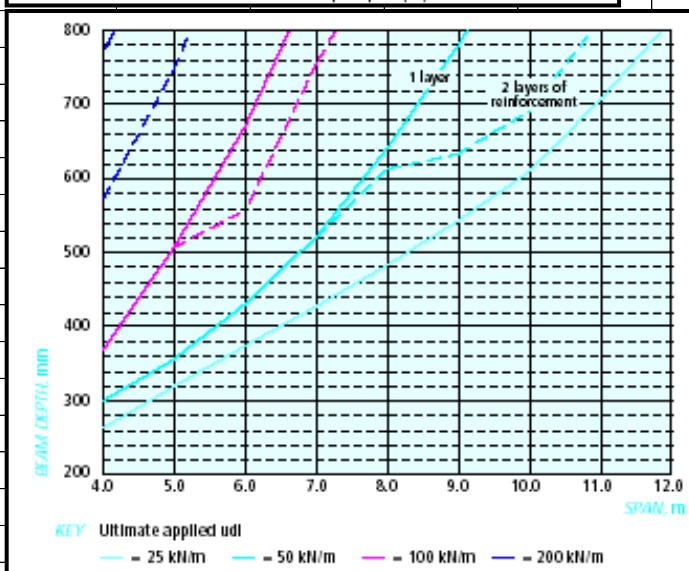
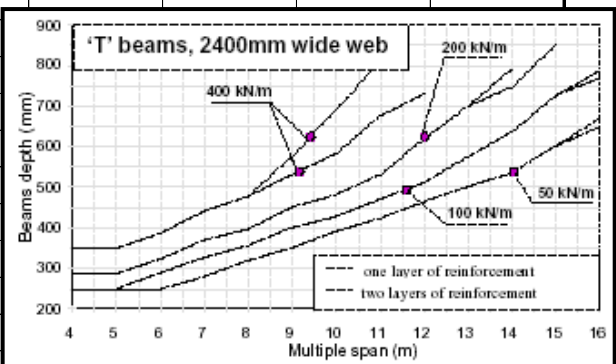
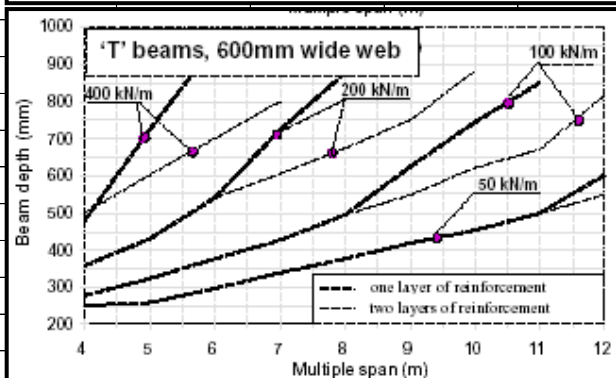
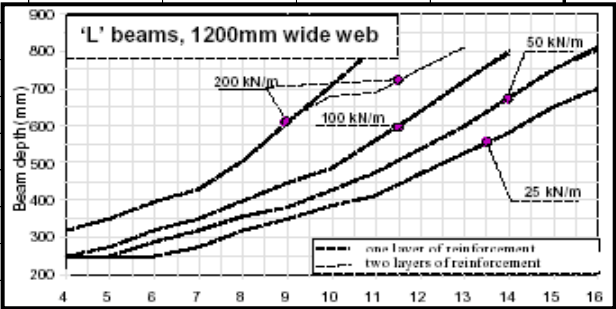
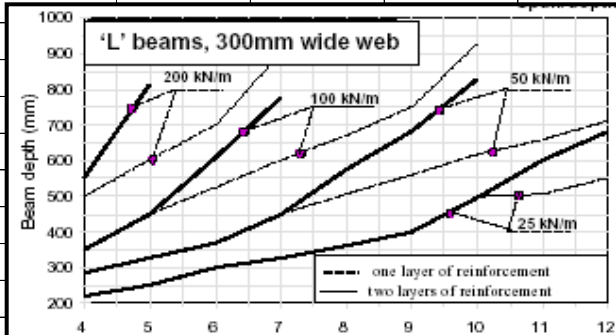
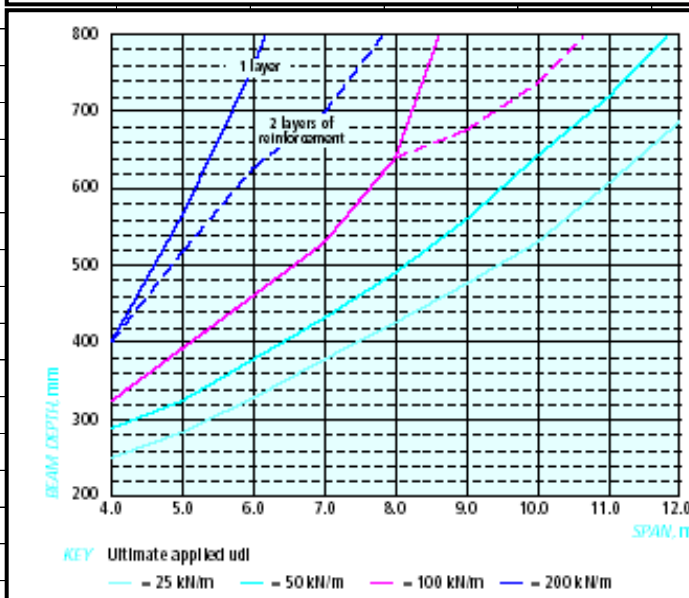


Figure 12 Bands of reinforcement for hogging moments

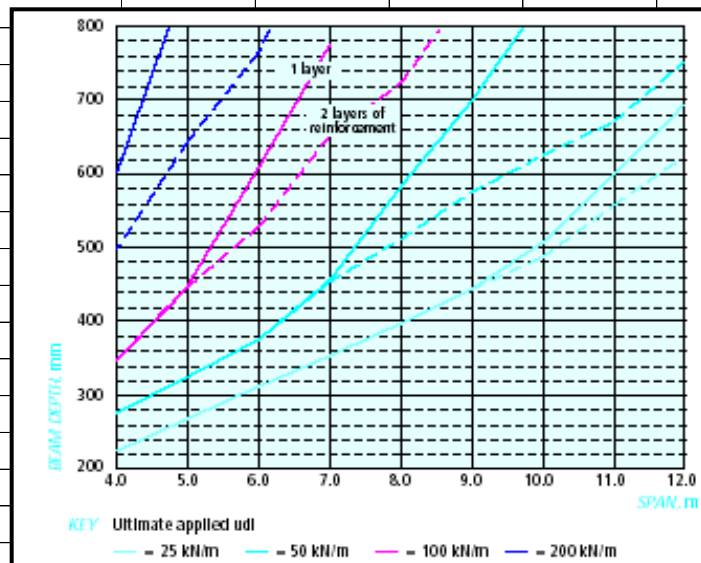
Scheme Design



Rectangular beams
300 mm
wide
single span

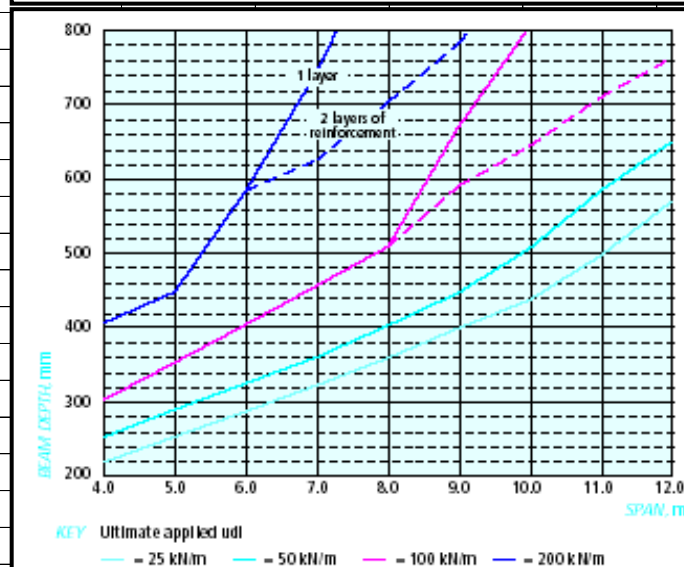


Rectangular beams
600 mm
wide
single span

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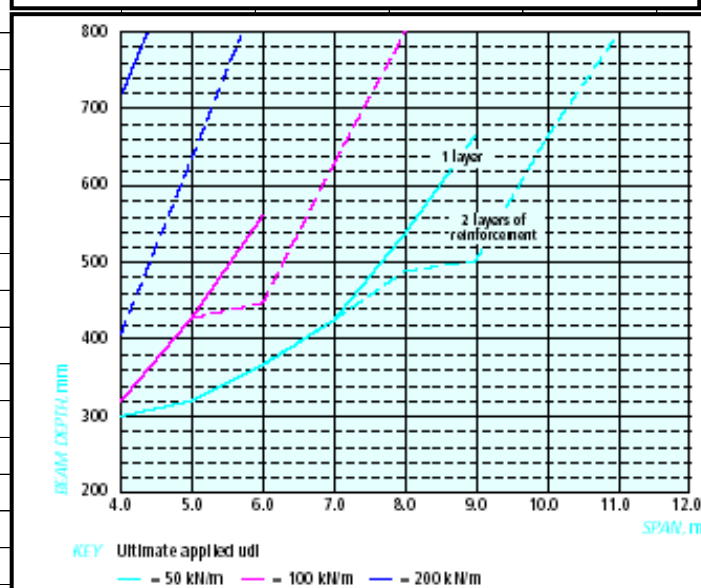
Rectangular beams
300 mm
wide

multiple span



Rectangular beams
600 mm
wide

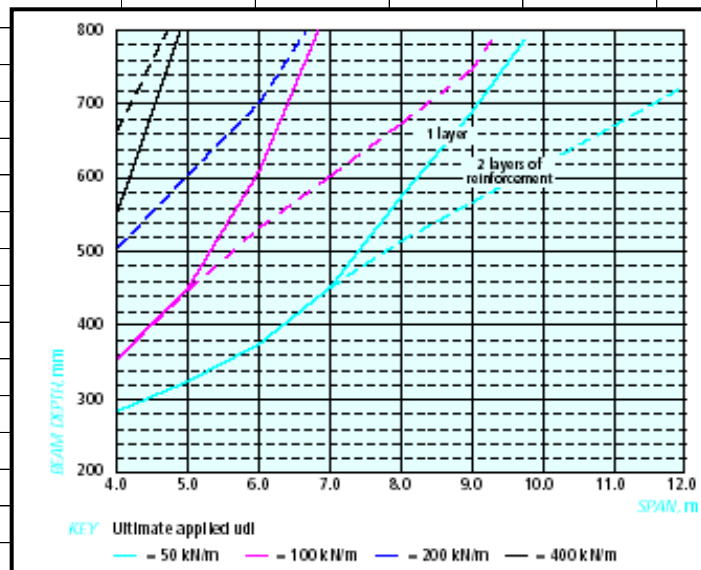
multiple span



T' beams
300 mm
wide web

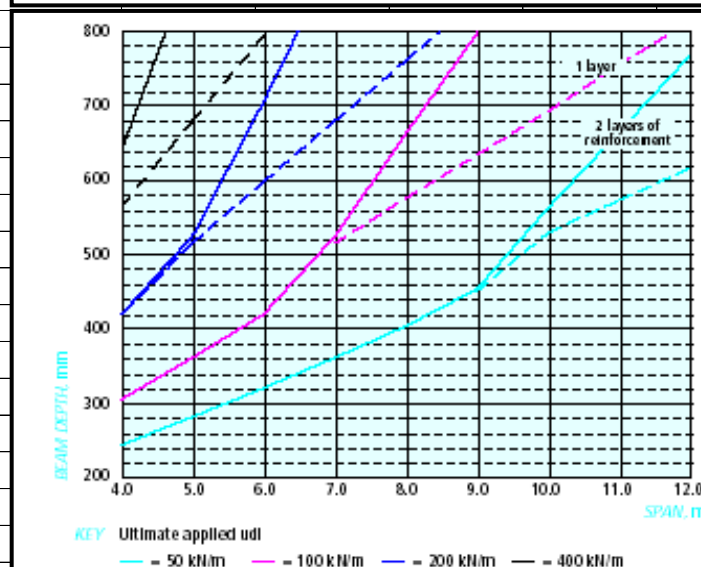
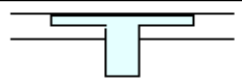
single span



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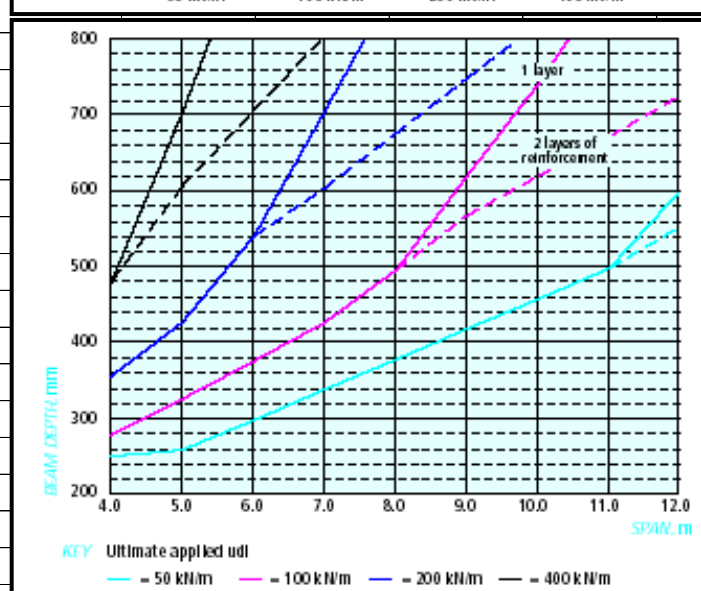
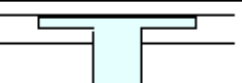
T beams
300 mm
wide web

multiple span



'T' beams
450 mm
wide web

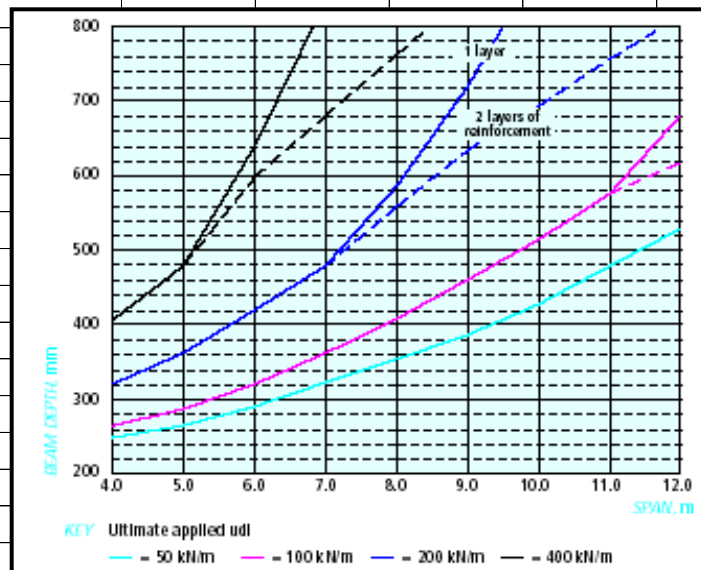
multiple span



T beams
600 mm
wide web

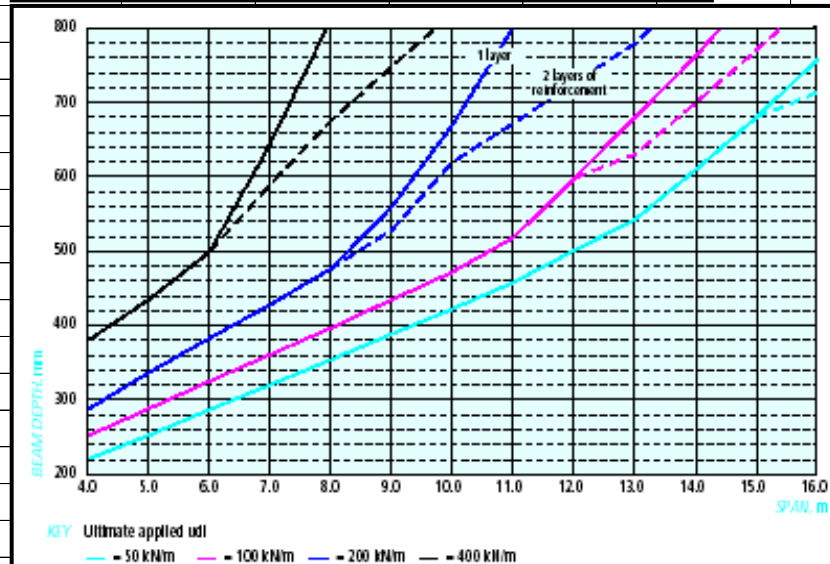
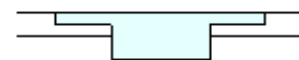
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BS8110

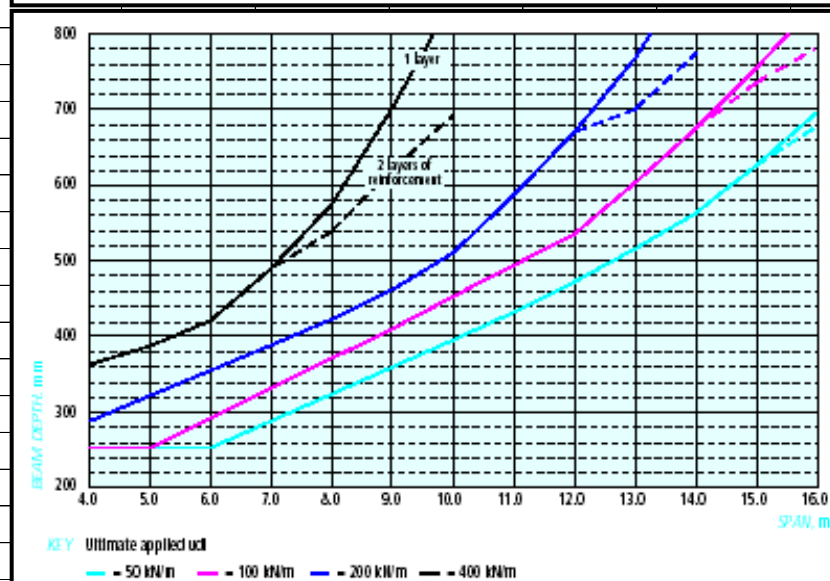
'T' beams
900 mm
wide web

multiple span



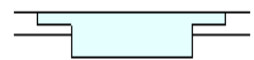
T' beams
1200 mm
wide web

multiple span



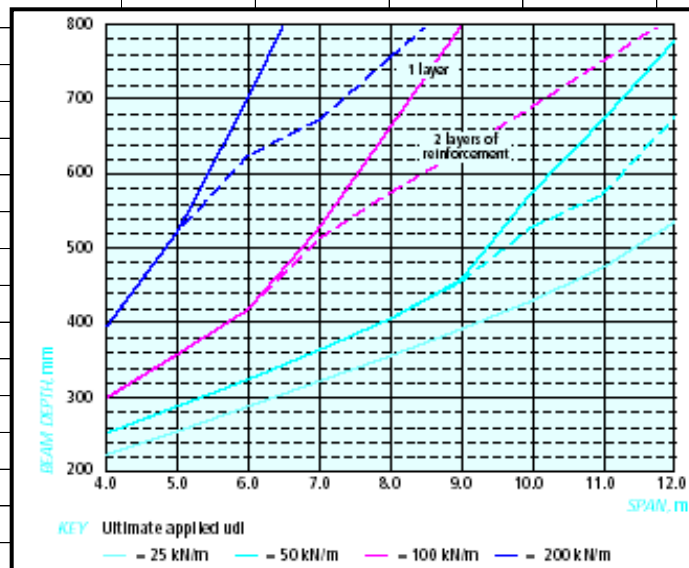
'T' beams
1800 mm
wide web

multiple span



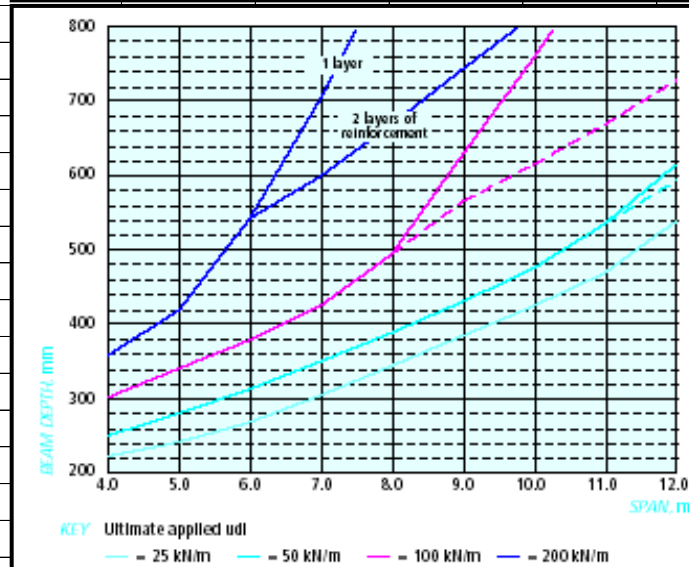
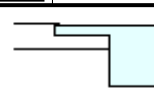
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						jXXX	40													
						Member/Location														
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.														
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.										
										BS8110										

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.												
						jXXX	41													
						Member/Location														
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.														
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.										
										BS8110										

BS8110

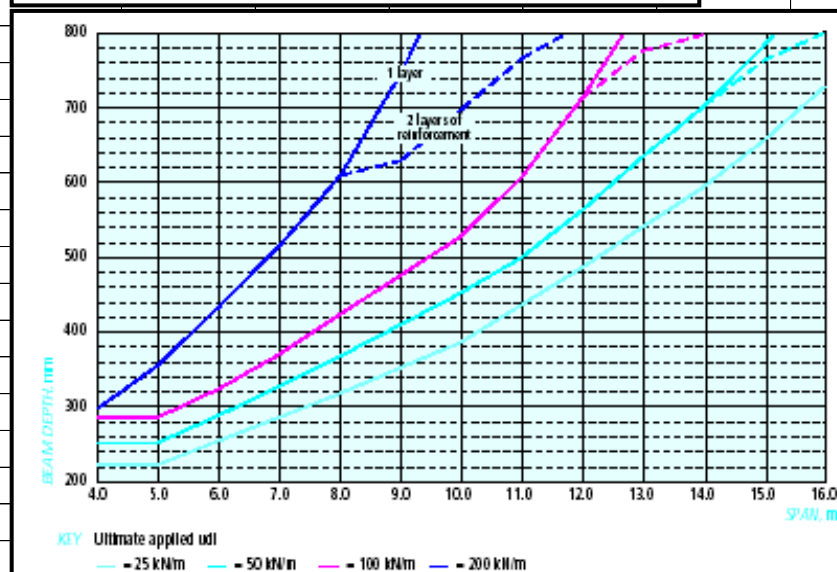
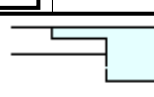
Inverted 'L' beams
450 mm
wide web

multiple span



Inverted 'L' beams
600 mm
wide web

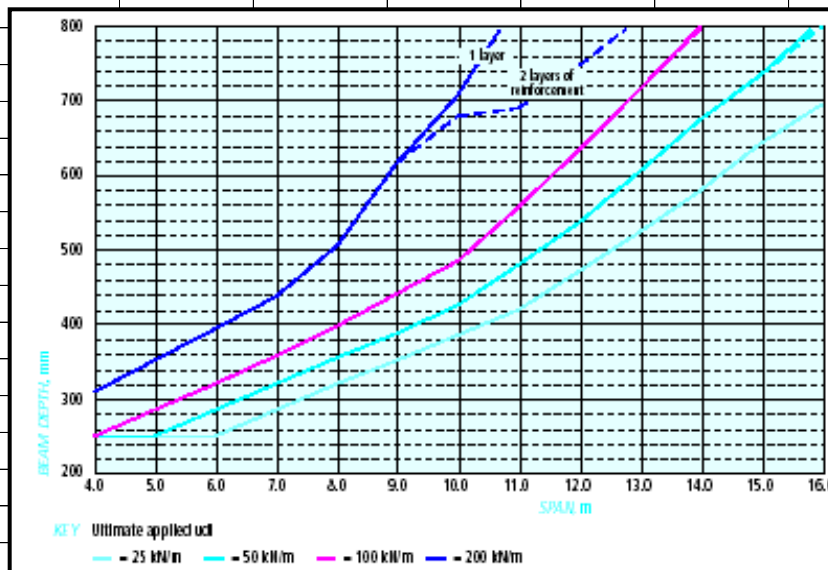
multiple span



Inverted 'L' beams
900 mm
wide web

multiple span



BS8110

Inverted 'L' beams
1200 mm
wide web

multiple span

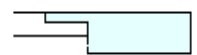


Table 3.1 — Allowable steel stresses in direct or flexural tension for serviceability limit states

Design crack width	Allowable stress	
	Plain bars ^a	Deformed bars ^b
mm	N/mm ²	N/mm ²
0.1	85	100
0.2	115	130

* 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;

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.		
						jXXX	44				
						Member/Location					
Job Title	Member Design - Reinforced Concrete Beam BS8110,					Drg. Ref.					
Member Design - RC Beam						Made by	XX	Date	16/9/2026	Chd.	
										BS8110	
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 Handbook)											
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	

[illegible]