

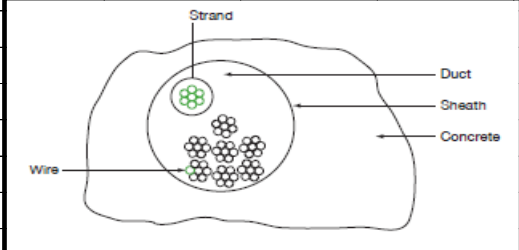
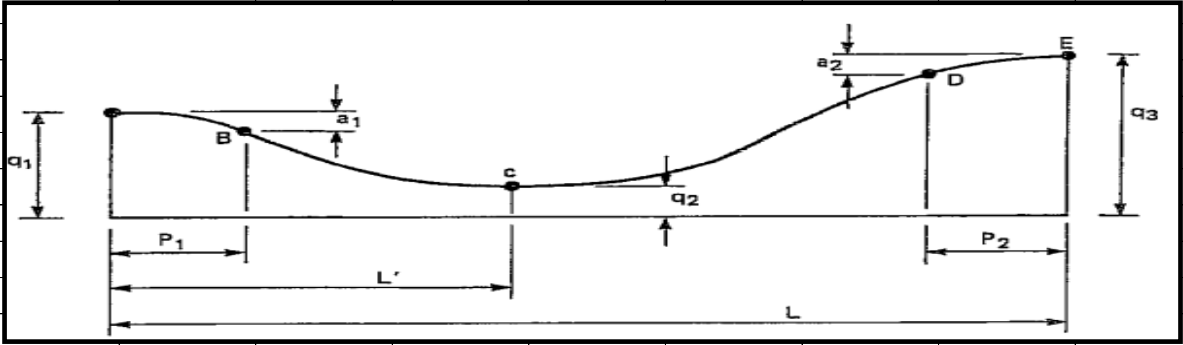
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.			
						jXXX	1				
						Member/Location					
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.					
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026		
						Chd.					
						BS8110					
Material Properties						BS8110 ▼					
Characteristic strength of concrete (PT beam and slab), $f_{cu} / f_{ck} f_c$						35	▼	28	▼	N/mm ²	OK
Note require $f_{cu} \geq 40\text{N/mm}^2$ (pre-T) or 35N/mm^2 (post-T) cl.4.1.8.1 BS8110, usually 40N/mm^2 , $\leq 105\text{N/mm}^2$											
Characteristic strength of concrete at transfer (PT beam and slab),						25	▼	20	▼	N/mm ²	OK
Note require $f_{ci} \geq 25\text{N/mm}^2$ cl.4.1.8.1 BS8110, usually 25N/mm^2 ;											
Characteristic strength of concrete (column), $f_{cu} / f_{ck} f_c' (f_{cu} \leq 105)$						40	▼	32	▼	N/mm ²	OK
Yield strength of longitudinal steel, f_y						Higher	▼	460	▼	N/mm ²	Foreword
Yield strength of shear link steel, f_{yv}						Higher	▼	460	▼	N/mm ²	cl.4.3.8.1
Type of concrete and density, ρ_c						Normal Weight 25kN/m ³		▼	25	kN/m ³	OK
Creep modulus factor, C_{MF}						Storage loading, $CMF=1/[1+f=2.0]$		▼			N/A
Beam and Slab Elastic Modulus	Uncracked, $E_{uncracked,28} = 20\text{kN/mm}^2 + 0.2f_{cu}$					100%	27.0	GPa	7.2 BS8110		
	Uncracked long term (creep), $E_{uncracked,28,cp} = C_{MF} \cdot E_{uncracked,28}$						9.0	GPa			
	Cracked, $E_{ck} = E_{uncracked,28} \cdot [0.5-1.0 \text{ beam, } 0.5-1.0 \text{ slab}]$					0% Crack ▼	27.0	GPa	8.3 BS8110		
	Cracked long term (creep), $E_{ck,cp} = E_{uncracked,28,cp} \cdot [0.5-1.0]$						9.0	GPa	8.3 BS8110		
Column Elastic Modulus	Uncracked, $E_{uncracked,28} = 20\text{kN/mm}^2 + 0.2f_{cu}$					100%	28.0	GPa	7.2 BS8110		
	Uncracked long term (creep), $E_{uncracked,28,cp} = C_{MF} \cdot E_{uncracked,28}$						9.3	GPa			
	Cracked, $E_{ck} = E_{uncracked,28} \cdot [0.5-1.0 \text{ column}]$					0% Crack ▼	28.0	GPa			
	Cracked long term (creep), $E_{ck,cp} = E_{uncracked,28,cp} \cdot [0.5-1.0]$						9.3	GPa			
TLS, SLS and ULS Load Combination Factors											
BS8110 ▼											
DL+SDL [G] and LL [Q] factors for ULS, k_G and k_Q						1.40	1.60	cl.2.4.3.1.1			
DL [S] and P' factors for TLS (E/L and P/E only, not S/E), k_S and k_P						1.00	1.15				
Pattern loading sag factor for ULS ($M_{SAG,ULS,E/E}$ for continuous only), k_{PAT}						1.00	cl.4.3.3				
Prestress Characteristics and Criteria											
BS8110 ▼											
Pre-tension or post-tension ?						Post-Tension	▼				
Prestress tendon(s) bonded or unbonded (post-tension only) ?						Bonded	▼	N/A			
Serviceability classification						Class 3 (Partial Prestressing)		▼	Note		
Flat slab hogging moment stress concentration						Beam, One-Way or Two-Way Slab		▼	N/A		
Class 1 No flexural tensile stresses (						N/A		▼	Note		
Class 2 Flexural tensile stresses, uncracked (no visible cracking);											
Class 3 Flexural tensile stresses, cracked (						$\leq 0.2\text{mm}$ normal crack widths		▼	N/A		
TLS permissible comp σ, f'_{max}						0.50	0.50	$f_{ci} / f_{ci}' =$	12.5	12.5	N/mm ²
All Classes $f'_{max} = 0.50f_{ci}$ or $\{0.24f_{ci} \text{ hog, } 0.33f_{ci} \text{ sag}\}$ for FTW-FS-DS											
TLS permissible tens σ, f'_{min}						-1.25	-1.25	$\sqrt{f_{ci}} / \sqrt{f_{ci}'}$	-6.3	-6.3	N/mm ²
Class 1 $f'_{min} = -1.0$									-1.0	N/mm ²	cl.4.3.5.2
Class 2 $f'_{min} = -0.45 \sqrt{f_{ci}}$ (pre-T), $-0.36 \sqrt{f_{ci}}$ (post-T)									-1.8	N/mm ²	cl.4.3.5.2
Class 3 $f'_{min} = -0.25f_{ci}$ or $-0.45 \sqrt{f_{ci}}$ for FTW-FS-DS									-6.3	N/mm ²	cl.4.3.5.2
SLS permissible comp σ, f_{max}						0.33	0.40	$f_{cu} / f_{c}' =$	11.6	14.0	N/mm ²
All Classes $f_{max} = 0.33f_{cu}$ (s/s, cont sag, cant), $0.40f_{cu}$ (cont hog) or $\{0.24f_{cu} \text{ hog, } 0.33f_{cu} \text{ sag}\}$											
SLS permissible tens σ, f_{min}						-0.45	-0.45	$\sqrt{f_{cu}} / \sqrt{f_{c}'}$	-2.7	-2.7	N/mm ²
Class 1 $f_{min} = -0.0$									0.0	N/mm ²	cl.4.3.4.3
Class 2 $f_{min,f_{cu} \leq 60\text{N/mm}^2} = -0.45 \sqrt{f_{cu}}$ (pre-T), $-0.36 \sqrt{f_{cu}}$ (post-T)									-2.1	N/mm ²	cl.4.3.4.3
									N/A	N/mm ²	cl.8.1 TR.49
Class 3 $f_{min,f_{cu} < 60\text{N/mm}^2} = \text{MAX}\{-0.25f_{cu}, -f(T.4.2, T.4.3)\}$									-2.7	-2.7	N/mm ²
									N/A	N/mm ²	cl.8.1 TR.49
Note by convention, positive stress is compressive and negative stress is tensile						Top	Bottom				
Note for flat slabs, if the full tributary width flat slab design strip (FTW-FS-DS) is employed, then f_{max} and f_{min} should account for the non-uniformity of bending moments across the panel width, whenever more onerous, adopt for											
(i) BD: permissible compressive stress $[f'_{max}, f_{max}] = \{0.24f_{ci/cu} \text{ hog, } 0.33f_{ci/cu} \text{ sag}\}$						T.2 TR.43					
BD: permissible tensile stress $[f'_{min}, f_{min}] = \{-0.45 \sqrt{f_{ci/cu}} \text{ hog, } -0.45 \sqrt{f_{ci/cu}} \text{ sag}\}$						T.2 TR.43					
(ii) Un-BD: permissible compressive stress $[f'_{max}, f_{max}] = \{0.24f_{ci/cu} \text{ hog, } 0.33f_{ci/cu} \text{ sag}\}$						T.2 TR.43					
Un-BD: permissible tensile stress $[f'_{min}, f_{min}] = \{-0.45 \sqrt{f_{ci/cu}} \text{ hog, } -0.45 \sqrt{f_{ci/cu}} \text{ sag}\}$ assume						T.2 TR.43					

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					Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab				Drg. Ref.			
Member Design - PC Beam and Slab					Made by	XX	Date	24/10/2026
								BS8110
Section Dimensions								BS8110 ▼
Span, L (usually $\geq 7.0\text{m}$ s/s or cont and $\geq 3.5\text{m}$ cant <i>cl.3.1 TR.43</i>)						10.000	m	OK
Available beam spacing, s_b						5.000	m	
<i>(effective width calcs, section properties flanged beam; usual spacing for interior beams; half for edge beams)</i>								
Section type at TLS						T - Section	▼	OK
Section type at (SLS/ULS)						T - Section	▼	OK
<i>(section type for section properties, bending calcs)</i>								
Support condition (and continuous end span moment ?) N/A					▼	Continuous	▼	Note
<i>(support condition for LTB restraint, effective width, prestress</i>					[x=0]	Continues	▼	
<i>force losses, action effects, deflection, longitudinal shear calcs)</i>					[x=L]	Continues	▼	
Design section hogging or sagging moment ?						Hogging Moment	▼	OK
<i>(incorporation of relevant action effects into and/or choice of equations for physical tendon profile, allowable range of P_0, max economic P_0, stress check equations, Magnel Diagram, allowable tendon profile, bending design; simply supported supports sagging moment only, continuous supports hogging and sagging moments and cantilever supports hogging moment only)</i>								
Section Type and Support Condition Option Selection								
Downstand Beam								
	Support	Effect	Slab	Type	Defl'n			
	S/S	Sag	Precast	Rect	Yes			
	S/S	Sag	Insitu	T/L	Yes			
	Cont.	Sag	Precast	Rect	Yes			
	Cont.	Sag	Insitu	T/L	Yes			
	Cont.	Hog	Precast	Rect	Yes			
	Cont.	Hog	Insitu	T/L ^{#1}	Yes			
	Cant.	Hog	Precast	Rect	Yes			
	Cant.	Hog	Insitu	T/L ^{#1}	Yes			
^{#1} Note that in the case that hogging with T/L- section is selected, the following parameters assume properties of a rect- section:- bending parameters $0.9x \leq h_f$ check and $F_{c,c}$;								
Overall depth, h (includes insitu slab thickness; {beam L/30, slab L/40} cont)						1000	mm	OK
Note minimum practical slab thickness to strand no.s are 130mm for 2, 140mm for 3 and 150mm for 4-5;								
Overall span-to-depth scheme suggested depth						500	mm	
Note s/s, cont $h \approx L/25+100\text{mm}$ ($L \leq 36\text{m}$), $h \approx L/20$ ($L > 36\text{m}$); Note cant $h \approx L/8$;								
Note usually $h \approx 70\%$ of equivalent non-prestressed member;								
Depth of flange, h_f						200	mm	
<i>(section properties flanged beam, bending flanged beam, longitudinal shear calcs)</i>								
Width (rectangular) or web width (flanged), b_w						500	mm	
Cover to all reinforcement, cover (usually 35 (C35) or 30 (C40) internal; 40 e						41	mm	T.4.8
Add cover (due to transverse steel layer(s)), cover _{add}						0	mm	
Column Section Dimensions (for Punching Shear Checks)								BS8110 ▼
Column section type, position and orientation					Rectangular	▼	Interior	▼
Design strip direction							Along h	▼
Depth, h (rect.) or dia., D (circ.)						800	mm	
Width, b (rect.) or N/A (circ.)						800	mm	
Column head dim. beyond column face, $l_{h\text{face}}$						0	mm	
Column head depth, d_h						0	mm	
13	Column head actual depth (rect.), $l_{h0,h} = h + (1 \text{ or } 2) \cdot l_{h\text{face}}$ or actual					800	mm	
-	Column head actual width (rect.), $l_{h0,b} = b + (1 \text{ or } 2) \cdot l_{h\text{face}}$ or N/A (					800	mm	
	Column head max. depth (rect.), $l_{h\text{max},h} = h + 2 \cdot (d_h - 40)$ or max. di					720	mm	
	Column head max. width (rect.), $l_{h\text{max},b} = b + 2 \cdot (d_h - 40)$ or N/A (cir					720	mm	
Column head eff. depth (rect.), $l_{h,h} = \text{MIN}(l_{h0,h}, l_{h\text{max},h})$ or eff. dia. (circ.), $l_{h,D} =$					800	mm		
Column head eff. width (rect.), $l_{h,b} = \text{MIN}(l_{h0,b}, l_{h\text{max},b})$ or N/A (circ.)					800	mm		

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers					Job No.	Sheet No.		Rev.
						jXXX	3		
						Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.			
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026
									BS8110
Type of Construction									BS8110 ▼
Type of construction						Type IV - Insitu Beam ▼			
						Type I	Type II	Type III	Type IV
						Insitu Slab	Insitu Transfer Slab	Precast Bridge Beam	Insitu Beam
						Type V			
						Insitu Transfer Beam			
Input Item									
Concrete grade (cube) at TLS and (SLS/ULS)						≥25MPa	≥25MPa	≥35MPa	≥25MPa
						≥35MPa	≥35MPa	≥35MPa	≥35MPa
Section type at TLS and (SLS/ULS)						Rect or T / L	Rect or T / L	Rect	T / L
Creep modulus factor, C _{MF}						Normal Loading	Storage Loading	Normal Loading	Normal Loading
Banding of prestress tendons and/or longitudinal steel (hogging and sagging)						Banded Flat Slab	Banded Flat Slab	Not Banded	Not Banded
Dead load, DL (on plan), DL _h						- or DL _h	DL _{v,STG(i-1)} or DL _h + DL _{v,STG(i-1)}	-	DL _h
									DL _h + DL _{v,STG(i-1)}
Superimposed dead load, SDL (on plan), SDL _h						SDL _h	SDL _{v,STG(i-1)}	DL _h +SDL _h	SDL _h
									SDL _{v,STG(i-1)}
Live load, LL (on plan), LL _h						LL _h	LL _h + LL _{v,STG(i-1)}	LL _h	LL _h
									LL _h + LL _{v,STG(i-1)}
Dead load, DL (on plan), DL _v						-	DL _{v,STG(i)}	-	-
									DL _{v,STG(i)}
Superimposed dead load, SDL (on plan), SDL _v						-	SDL _{v,STG(i)}	-	-
									SDL _{v,STG(i)}
Live load LL (on plan), LL _v						-	LL _{v,STG(i)}	-	-
									LL _{v,STG(i)}
Longitudinal shear between web and flange ?						Ignore or Consider	Ignore or Consider	Ignore	Consider
Note STG(i) refers to prestressing stage(i) where i=1,2,3...; Note STG(0) refers to nothing;									
Dual-Cast and Multi-Stage Stressing Construction (Insitu Transfer Slab Without Slab Band						BS8110 ▼			
Note dual-cast and/or multi-stage stressing construction may also apply to Insitu Transfer Slab									
flat slab with slab band and Insitu Transfer Beam, these however not illustrated herein;									
Single-cast or dual-cast construction						N/A ▼	N/A ▼		
Additional bottom compressive stress at TLS and (SLS/ULS)						0.0	0.0	N/mm ²	N/A
Input Item		Casting Sequence Stressing Stage		First-Cast, C1	Second-Cast, C2	Stage 1			
						Stage 2,3..			
Concrete grade (cube) at TLS				≥25MPa	≥25MPa	≥35MPa			
Concrete grade (cube) at (SLS/ULS)				≥25MPa	≥35MPa	≥35MPa			
Creep modulus factor, C _{MF}				Normal Loading	Storage Loading	Storage Loading			
Overall depth, h				h _{C1} ≈ h _{C2} /3	h _{C2}	h _{C2}			
Tendons				[N _T x N _s] _{C1}	[N _T x N _s] _{C2,STG(1)}	Σ[N _T x N _s] _{STG(1,2,3..)}			
Tendon profile				Within h _{C1}	Within h _{C2}	Within h _{C2}			
Additional bottom compressive stress				0N/mm ²	≥0N/mm ²	≥0N/mm ²			
DL (on plan), DL _h				-	-	DL _{v,STG(1,2..)}			
SDL (on plan), SDL _h				(([DL _b] _{C2} -[DL _b] _{C1})/t _w	SDL _h	SDL _h + SDL _{v,STG(1,2..)}			
LL (on plan), LL _h				1.5kPa	LL _h	LL _h + LL _{v,STG(1,2..)}			
DL (on plan), DL _v				-	DL _{v,STG(1)}	DL _{v,STG(2,3..)}			
SDL (on plan), SDL _v				-	SDL _{v,STG(1)}	SDL _{v,STG(2,3..)}			
LL (on plan), LL _v				-	LL _{v,STG(1)}	LL _{v,STG(2,3..)}			
Note if only single-cast, refer to second-cast, C2 only;									
Note if only single-stage stressing, refer to stage 1 stressing only;									

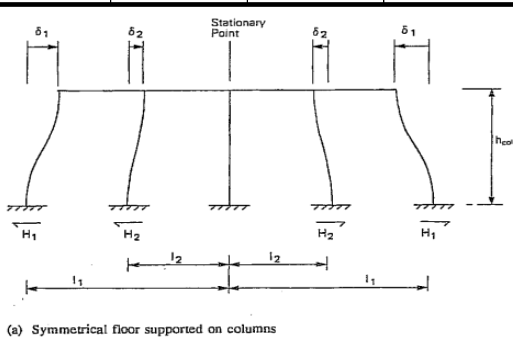
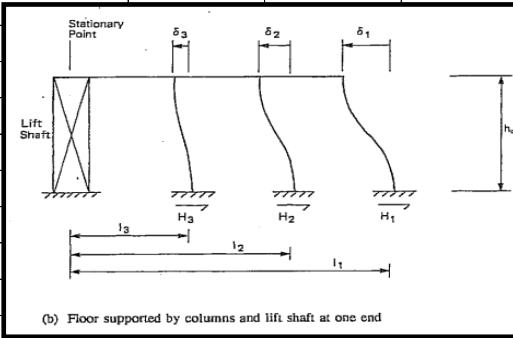
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				Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab			Drg. Ref.		
Member Design - PC Beam and Slab				Made by	XX	Date
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				BS8110		
Code of Practice				BS8110 ▼		
Code of practice adopted				BS8110 ▼		
Design and Critical Section Definition						
Note in this spreadsheet, unless noted otherwise, design section refers to the (moment) design section and not the (shear) design section which in general is at a different location. This design section is located at the mid-span for simply supported beams, the LHS support or at / near the mid-span for continuous beams and the LHS support for cantilever beams. The critical section on the other hand, refers to the (shear) critical section which is the LHS support for all simply-supported, continuous and cantilever beams;						
Limitations						
1 Section properties do not consider the transformed section.						
2 Flanged option only caters for downstand sections, not upstand sections.						
3 Untensioned reinforcement is always exterior to the prestressed tendon(s);						
Material Stress-Strain Curves						
Figure 2.1 — Short term design stress-strain curve for normal-weight concrete						
NOTE f_y is in N/mm ² .						
Figure 2.2 — Short term design stress-strain curve for reinforcement						
NOTE f_{pu} is in N/mm ² .						
Figure 2.3 — Short term design stress-strain curve for prestressing tendons						

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
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Member Design - PC Beam and Slab			Made by	XX	Date 24/10/2026 Chd.
			BS8110		
Prestress Reinforcement and Physical Tendon Profile			BS8110 ▼		
			Note that bonded tendons are placed in metal ducts which are cement grouted to ensure bond and corrosion protection. Unbonded tendons are protected with a layer of grease for corrosion protection inside a plastic sheath (note PVC should not be used as the plastic sheath) (cl.4.2.2 TR.43);		
			Wires → Strands	Tendon = Duct + Strands	
Banding of prestress tendons			100%	tendons within	1.00 b _w OK
Number of prestress tendon(s), N _T			1		
Prestress tendon(s) size (maximum no. of strands)			12	▼	
Number of prestress strands per prestress tendon, N _s			12	▼	
Note N _s could be 12 for PT transfer beams or PT transfer slabs whilst is usually 3 to 5 for PT slabs;					
Total number of prestress strands, N _T .N _s			12		
Duct (external) diameter, D _{T,H} and D _{T,V}			100%	100%	87 mm
Prestress strands code, grade and ϕ _s			[ASTM A416] Grade 270 d = 15.24mm ▼		
Note usually [BS5896] 7-wire super d=12.9mm / 15.7mm or [ASTM A416] Grade 270 d=12.7mm / 15.2mm;					
Prestress strands nominal diameter, ϕ _s			15.24 mm		
Prestress strands nominal area, A _s			140.00 mm ²		
Elastic modulus of prestress strand, E _p			186.0 GPa		
Ultimate (characteristic) tensile strength of prestress strand, f _{pk}			1860 N/mm ²		
Proof (0.1%) strength of prestress strand, f _{p,0.1}			1670 N/mm ²		
Ultimate (characteristic) tensile load of prestress strand, F _{pk}			260.7 kN		
Proof (0.1%) load of prestress strand, F _{p,0.1}			234.6 kN		
Number of layers of prestress tendon(s), n _{layers,PT}			1 layer(s)		
Spacer for prestress tendon(s), s _{r,PT} = MAX (2D _{T,V} pre-T or D _{T,V} post-T, 40mm)			87 mm 12.4.3 BS8		
Top limit of (negative) physical eccentricity of prestress tendon(s), e _{min,t}			-197 mm		
Note e _{min,t} = -(X _{c,(SLS/ULS)} - cover - MAX(ϕ _{link} , cover _{add})) - [D _{T,V} + (n _{layers,PT} - 1)(D _{T,V} + s _{r,PT})] / 2 - [ϕ _t + (n _{layers,tens} - 1)(ϕ _t + s _{r,tens})]					
Bottom limit of (positive) physical eccentricity of prestress tendon(s), e _{max,b}			525 mm		
Note e _{max,b} = h - X _{c,(SLS/ULS)} - cover - ϕ _{link} - [D _{T,V} + (n _{layers,PT} - 1)(D _{T,V} + s _{r,PT})] / 2 - [ϕ _t + (n _{layers,tens} - 1)(ϕ _t + s _{r,tens})] [exte					
Note by convention, e is positive downwards, measured from the c			TLS SLS/ULS)		
Physical eccentricity of prestress tendon(s) at design section, e _{HOG}			-196	-196	mm
Physical eccentricity of prestress tendon(s) at design section, e _{SAG}			524	524	mm
Note by convention, e is positive downwards, measured from the centroid of the TLS/(SLS/ULS) section;					
Note ensure (e _{min,t} ≤ e _{HOG} and e _{SAG} ≤ e _{max,b});					
Physical eccentricity of prestress tendon(s) at design section utilisation			100% OK		
					
Dimension, q ₁			Continues ▼	840	mm
Dimension, q ₂				120	mm
Dimension, q ₃ (N/A if cantilever)			Continues ▼	100%	840 mm
Dimension, L				10000	mm
Dimension, p ₁			10%L	1000	mm
Dimension, p ₂ (N/A if cantilever)			10%L	1000	mm
Goal Seek q1, q2, q3					

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						BS8110 ▼			
Coefficient, $l = q_1 - q_3$ (note $l = N/A$ if cantilever)						0 mm			
Coefficient, $m = (p_2 - 2L) \cdot (q_1 - q_2) + p_1 \cdot (q_3 - q_2)$ (note $m = N/A$ if cantilever)						-1.3E+07 mm ²			
Coefficient, $n = (q_1 - q_2) \cdot (L - p_2) \cdot L$ (note $n = N/A$ if cantilever)						6.5E+10 mm ³			
Dimension, $L' = [-m - \sqrt{(m^2 - 4l \cdot n)}] / (2l)$ (note $L' = L/2$ if $l = 0$ or L if car)						5000 mm			
Dimension, $a_1 = (q_1 - q_2) \cdot p_1 / L'$						144 mm			
Dimension, $a_2 = (q_3 - q_2) \cdot p_2 / (L - L')$ (note $a_2 = N/A$ if cantilever)						144 mm			
Physical Eccentricity of Prestress Tendon(s) at All Sections, e_{var}									
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m	
e_{var}	-196	-160	-52	175	346	460	517	mm	
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m	
e_{var}	517	460	346	175	-52	-160	-196	mm	
Note by convention, e_{var} is positive downwards, measured from the centroid of the (SLS/ULS) section;									
Note tendon profile equations are as follows: -									
if $x < p_1$ then $e_{var} = a_1 / p_1^2 \cdot (x)^2 + h - q_1 - x_{c(SLS/ULS)}$;									
if $x \leq L'$ then $e_{var} = -(q_1 - a_1 - q_2) / (L' - p_1)^2 \cdot (L' - x)^2 + h - q_2 - x_{c(SLS/ULS)}$;									
if $x \leq L - p_2$ then $e_{var} = -(q_3 - a_2 - q_2) / (L - L' - p_2)^2 \cdot (x - L')^2 + h - q_2 - x_{c(SLS/ULS)}$;									
if $x > L - p_2$ then $e_{var} = a_2 / p_2^2 \cdot (L - x)^2 + h - q_3 - x_{c(SLS/ULS)}$;									
Physical Tendon Profile Distance, x (m) - - - • e (at design section) — evar (at all sections)									
Physical eccentricity of prestress tendon(s) at all sections, MIN (e_{HOG} , e_{var})						-196		mm	
Physical eccentricity of prestress tendon(s) at all sections, MAX (e_{SAG} , e_{var})						524		mm	
Note by convention, e is positive downwards, measured from the centroid of the (SLS/ULS) section;									
Note ensure ($e_{min,t} \leq MIN(e_{var})$) and ($MAX(e_{var}) \leq e_{max,b}$);									
Physical eccentricity of prestress tendon(s) at all sections utilisation						100%		OK	
Longitudinal and Shear Reinforcement Details						BS8110 ▼			
						HOG		SAG	
Elastic modulus of longitudinal reinforcement, E_s						200.0		GPa	
Banding of longitudinal steel (hogging)				100%	rebar	within	1.00	b_w	OK
Banding of longitudinal steel (sagging)				100%	rebar	within	1.00	b_w	OK
Untensioned steel reinforcement diameter, ϕ_t						20	▼ 25	▼	mm
Untensioned steel reinforcement number, n_t						10	5		
Untensioned steel area provided, $A_{s,prov} = n_t \cdot \pi \cdot \phi_t^2 / 4$						3142	2454	mm ²	
Number of layers of untensioned steel, $n_{layers,tens}$						2	1	layer(s)	
Spacer for untensioned steel, $s_{r,tens} = MAX(\phi_t, 25mm)$						25	25	mm	
Shear link diameter, ϕ_{link}						10	▼	mm	
Number of links in a cross section, i.e. number of legs, n_{leg}						4			
Area provided by all links in a cross-section, $A_{sv,prov} = \pi \cdot \phi_{link}^2 / 4 \cdot n_{leg}$						314	mm ²		
Pitch of links, S						100	mm		
No., $n_{l,2/3}$ area, $A_{sv,prov,2/3} = n_{l,2/3} \cdot \pi \cdot \phi_{link}^2 / 4$				N/A	N/A	N/A	N/A	mm ²	
No., $n_{l,4/5}$ area, $A_{sv,prov,4/5} = n_{l,4/5} \cdot \pi \cdot \phi_{link}^2 / 4$				N/A	N/A	N/A	N/A	mm ²	

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.	
						jXXX	8		
						Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.			
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026
						Chd.			
						BS8110			
External Loading						BS8110 ▼			
Note for UDLs (DL, SDL and LL) only uniform loading considered, no pattern loading considered;									
External loading tributary width, t_w						5.000		m	
						{h}		{v}	
Dead load (on plan), $\{DL_h, DL_v\}$						4.50		0.00 kPa	
Superimposed dead load (on plan), $\{SDL_h, SDL_v\}$						15.00		0.00 kPa	
Live load (on plan), $\{LL_h, LL_v\}$						10.00		0.00 kPa	
Dead load (point load), $\{DL_{point,h}, DL_{point,v}\}$						0		0 kN	
Distance to DL_{point} from LHS, $\{a_h, a_v\}$						0.000		0.000 m	
Note that $DL_{point,h}$ complements TLS beam loading whilst $DL_{point,v}$ does not;									
Dead load of beam, $DL_b = h \cdot b_w \cdot \rho_c$						12.5		kN/m	
TLS beam loading, $\omega_{TLS,E/E} = k_s \cdot [DL_h] \cdot t_w + k_s \cdot DL_b$						35.0		kN/m	
DL+SDL beam loading, $\omega_{DL+SDL} = [DL_h + DL_v + SDL_h + SDL_v] \cdot t_w + DL_b$						110.0		kN/m	
LL beam loading, $\omega_{LL} = [LL_h + LL_v] \cdot t_w$						50.0		kN/m	
SLS beam loading, $\omega_{SLS,E/E} = [DL_h + DL_v + SDL_h + SDL_v + LL_h + LL_v] \cdot t_w + DL_b$						160.0		kN/m	
ULS beam loading, $\omega_{ULS,E/E} = [k_G \cdot (DL_h + DL_v + SDL_h + SDL_v) + k_Q \cdot (LL_h + LL_v)]$						77.2		234.0 kN/m	
								OK	
Prestress Force at SLS (With Restraint, With Long Term Losses)						BS8110 ▼			
Prestress force at SLS (w. restraint, w. LT losses), KP_0						1820		kN	
Prestress force at transfer (w. restraint, w.o. ST losses), P_0						2346		kN	
Prestress force losses factor, K						0.78			
Effective (long-term) stress, $f_{se} = \% \cdot K f_{pk}$						1082		N/mm ²	
Note f_{se} usually 1100 to 1200 N/mm ² for bonded and unbonded tendons respectively (Aalami, 2014);									
Percentage of Load Balancing at SLS						BS8110 ▼			
SLS equivalent load, $\omega_{SLS,E/L}$						-131.0		kN/m	
S/S. $\omega_{SLS,E/L} = -8KP_0 \cdot e_d / s^2$						N/A		kN/m	
Cont. $\omega_{SLS,E/L} = -8KP_0 \cdot e_d / s^2$						-131.0		kN/m	
Cant. $\omega_{SLS,E/L} = -2KP_0 \cdot e_d / s^2$						N/A		kN/m	
Note that the equivalent load calculation includes the support peak tendon reverse curvature;									
Percentage of load balancing at SLS basis						SLS		▼	
Percentage of load balancing at SLS, $ \omega_{SLS,E/L} / \omega_{TLS,E/E} + DL_{point,h}/L + DL_{point,v}/L$						kN/m		%	
of TLS beam loading, $\omega_{TLS,E/E} + DL_{point,h}/L$						35.0		374%	
of DL+SDL beam loading, $\omega_{DL+SDL} + DL_{point,h}/L + DL_{point,v}/L$						110.0		119%	
of SLS beam loading, $\omega_{SLS,E/E} + DL_{point,h}/L + DL_{point,v}/L$						160.0		82%	
						L-Sup		Span	
						2.000		8.000	
Distance between points of inflexion, s						2.000		2.000 m	
S/S. $s = \{2p_1 \text{ (l-sup)}, L - p_1 - p_2 \text{ (span)}, 2p_2 \text{ (r-sup)}\}$						N/A		N/A m	
Cont. $s = \{2p_1 \text{ (l-sup)}, L - p_1 - p_2 \text{ (span)}, 2p_2 \text{ (r-sup)}\}$						2.000		8.000 m	
Cant. $s = \{2p_1 \text{ (l-sup)}, L - p_1 \text{ (span)}, N/A \text{ (r-sup)}\}$						N/A		N/A m	
Total drape between points of inflexion, e_d						144		576 mm	
S/S. $e_d = \{a_1 \text{ (l-sup)}, e_c - [e_B + e_D]/2 \text{ (span)}, a_2 \text{ (r-sup)}\}$						N/A		N/A mm	
Cont. $e_d = \{a_1 \text{ (l-sup)}, e_c - [e_B + e_D]/2 \text{ (span)}, a_2 \text{ (r-sup)}\}$						144		576 mm	
Cant. $e_d = \{a_1 \text{ (l-sup)}, e_c - e_B \text{ (span)}, N/A \text{ (r-sup)}\}$						N/A		N/A mm	
Eccentricity, $e_A = e_{var}(x=0)$								-196 mm	
Eccentricity, $e_B = e_{var}(x=p_1)$								-52 mm	
Eccentricity, $e_C = e_{var}(x=L')$ ($e_{var}(x=L)$ if cantilever)								524 mm	
Eccentricity, $e_D = e_{var}(x=L-p_2)$ (N/A if cantilever)								-52 mm	
Eccentricity, $e_E = e_{var}(x=L)$ (N/A if cantilever)								-196 mm	
Percentage of load balancing at SLS utilisation, $ \omega_{SLS,E/L} / \omega_{TLS,E/E} + DL_{point,h}/L + DL_{point,v}/L$						82%		OK	

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.				
		jXXX	9					
		Member/Location						
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.						
Member Design - PC Beam and Slab		Made by	XX	Date 24/10/2026 Chd.				
				BS8110				
Prestress Force at TLS (With Restraint, With Short Term Losses)				BS8110 ▼				
Prestress force at transfer (w. restraint, w. ST losses), P'			2116 kN					
Note max allowable prestress force at transfer (w. restraint, w. ST losses), $P' \leq 75\% \cdot (N_T \cdot N_s \cdot F_{pk})$;				cl.4.7.1				
Max allowable prestress force at transfer (w. restraint, w. ST losses) utilisation			90%	OK				
Prestress Force at TLS (With Restraint, Without Short Term Losses)				BS8110 ▼				
Prestress force at transfer (w.o. restraint, w.o. ST losses), $P_{0,free}$			2346 kN					
Note prestress force at transfer (w.o. restraint, w.o. ST losses), $P_{0,free} = \% \cdot (N_T \cdot N_s \cdot F_{pk})$;								
Percentage of tensile capacity, % ($P_{0,free} \leq 80\% \cdot (N_T \cdot N_s \cdot F_{pk})$ cl.4.7.1)			75.0 %	OK				
Total restraint force, ΣH_i		Exclude ▼	0 kN					
Note calculate UTs for cases with / without restraint to prestress force;								
 (a) Symmetrical floor supported on columns  (b) Floor supported by columns and lift shaft at one end								
Note the total tension in the floor due to the restraint to shortening is the sum of all the column forces to one side of the stationary point, i.e. in (a) $H_1 + H_2$ and in (b) $H_1 + H_2 + H_3$ (cl.3.3 TR.43); Note restraint force is due to floor shortening which is a result of elastic shortening due to the prestress force, creep shortening due to the prestress force and concrete $H_i = \frac{12E_s I_i \delta_i}{(h_{col})^3}$ $\delta_i = \epsilon_{LT} \times l_i$								
Total long term strain, $\epsilon_{LT} = \epsilon_{es} + \epsilon_{cp} + \epsilon_{sh}$			1486 $\times 10^{-6}$	cl.3.3 TR.43				
Elastic shortening strain, ϵ_{es}			396 $\times 10^{-6}$					
Note $\epsilon_{es} = \sigma_{es} / E_{uncracked,28} = [(P_{0,free} / A_{TLS}) \cdot (1 + e^2 A_{TLS} / I_{TLS})] / E_{uncracked,28}$;				MOSLEY				
noting that e above is taken as MAX[e _{HOG} , e _{SAG}];								
Creep strain, ϵ_{cp}			990 $\times 10^{-6}$					
Note creep strain, $\epsilon_{cp} = 2.5 \epsilon_{es}$;				cl.3.3 TR.43				
Shrinkage strain, ϵ_{sh}			100 $\times 10^{-6}$					
Note ϵ_{sh} usually 100×10^{-6} for UK outdoor exposure conditions;				cl.4.8.4				
Note ϵ_{sh} usually 300×10^{-6} for UK indoor exposure conditions;				cl.4.8.4				
Column Restraints								
	l_i (m)	$\delta_{i,es}$ (m)	$\delta_{i,cp+sh}$ (m)	I_i (m ⁴)	$E_{c,es}$ (GPa)	$E_{c,cp+sh}$ (GPa)	h_{col} (m)	H_i (kN)
No.1	13.300	0.005	0.015	0.0500	28.0	9.3	8.000	331
No.2	13.300	0.005	0.015	0.0500	28.0	9.3	8.000	331
No.3	0.000	0.000	0.000	0.0000	28.0	9.3	0.000	0
No.4	0.000	0.000	0.000	0.0000	28.0	9.3	0.000	0
No.5	0.000	0.000	0.000	0.0000	28.0	9.3	0.000	0
No.6	0.000	0.000	0.000	0.0000	28.0	9.3	0.000	0
Prestress force at transfer (w. restraint, w.o. ST losses), P_0			2346 kN					
Note prestress force at transfer (w. restraint, w.o. ST losses), $P_0 = \text{MAX}(P_{0,free} - \Sigma H_i, 0)$;								

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			jXXX	10	
			Member/Location		
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Member Design - PC Beam and Slab			Made by	Date	Chd.
			XX	24/10/2026	
					BS8110
Prestress Force Losses					BS8110 ▼
Prestress force (total) losses factor, $K = (P_0 - P_L) / P_0$			100%	0.78	Goal Seek Losses Factor, K
Note prestress force (total) losses factor, K is usually 0.70 i.e. 30% to 0.80 i.e. 20% (cl.6.8 TR.43);					
Prestress force (total) loss, $P_L = P_{L,DF} + P_{L,ES} + P_{L,CC} + P_{L,TR} + P_{L,CS}$			Include ▼	526 kN	
Prestress force loss due to duct friction, $P_{L,DF}$				199 kN	
Prestress force loss due to elastic shortening, $P_{L,ES}$				32 kN	
Prestress force loss due to concrete creep, $P_{L,CC}$				109 kN	
Prestress force loss due to tendon relaxation, $P_{L,TR}$				156 kN	
Prestress force loss due to concrete shrinkage, $P_{L,CS}$				31 kN	
Prestress force (short term) losses factor, $(P_0 - P_{L,DF} - P_{L,ES}) / P_0$				0.90	Goal Seek ST Losses Factor
Note prestress force (short term) losses factor, $(P_0 - P_{L,DF} - P_{L,ES}) / P_0$ is usually 0.90 i.e. 10% (cl.4.9.4.3)					
Prestress force (short term) loss, $P_{L,ST} = P_{L,DF} + P_{L,ES}$			Include ▼	231 kN	
Prestress force loss due to duct friction, $P_{L,DF}$				199 kN	
Prestress force loss due to elastic shortening, $P_{L,ES}$				32 kN	
Prestress Force Loss due to Duct Friction (Short Term) (Post-Tension Only)					
Prestress force loss due to duct friction, $P_{L,DF}$			100%	199 kN	cl.4.9
$P_{L,DF} = P_0 \left(1 - e^{-(\mu x / r_{ps} + kx)} \right)$					
Note $x = \{L/2 \text{ simply supported, } L/2 \text{ continuous, } L \text{ cantilever}\}$					
Coefficient of friction, μ (usually 0.25 BD, 0.07 un-BD (Aalami, 2014))			0.25	/rad	cl.4.9.4.3
Lightly-rusted strand in galvanized steel duct: 0.25			▼		
Wobble factor, k (usually 46×10^{-4} rad/m (Aalami, 2014))			33	$\times 10^{-4}$ rad/m	cl.4.9.3.3
General: 33×10^{-4}			▼		
Tendon curvature, C				28.800×10^{-6}	
$C = \begin{cases} \text{simply supported} & \text{cantilever} \\ \text{continuous} & \\ \text{hogging / sagging} & \text{hogging / sagging} \\ \frac{ \text{MAX}(e_{\text{SAG}}, e_{\text{var}}) - \text{MIN}(e_{\text{HOG}}, e_{\text{var}}) }{(L/2)^2} & \frac{ \text{MAX}(e_{\text{SAG}}, e_{\text{var}}) - \text{MIN}(e_{\text{HOG}}, e_{\text{var}}) }{L^2} \end{cases}$					
Tendon radius of curvature, r_{ps}				17.4 m	
$r_{ps} \approx \frac{1}{d^2 y / dx^2} = \frac{1}{2C}$					
Prestress Force Loss due to Elastic Shortening of Concrete (Short Term)					
Prestress force loss due to elastic shortening of concrete, $P_{L,ES}$			100%	32 kN	cl.4.8.3
$P_{L,ES} = P_0 - \frac{P_0}{1 + \text{factor} \cdot \frac{E_p}{E_{\text{uncracked,28,transfer}}} \cdot \frac{N_T \cdot N_s \cdot A_s}{A_{\text{TLS}}} \left(1 + \frac{(5 \cdot \text{MAX}(e_{\text{HOG}} , e_{\text{SAG}}) / 8)^2 \cdot A_{\text{TLS}}}{I_{\text{TLS}}} \right)}$					
$\text{factor} = \begin{cases} 1.0 & \text{Pre-Tension} \\ 0.5 & \text{Post-Tension} \end{cases}$					
Beam and slab uncracked elastic modulus at transfer, E			100%	25.0 GPa	7.2 BS8110
Reduced prestress force after ST losses, $P' = P_0 - (P_{L,DF} + P_{L,ES})$				2116 kN	

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		jXXX	11	
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Member Design - PC Beam and Slab		Made by	XX	Date 24/10/2026
		Chd.	BS8110	
			BS8110 ▼	
Prestress Force Loss due to Concrete Creep Under Sustained Compression (Long Term)				
Prestress force loss due to concrete creep, $P_{L,CC}$			109 kN	cl.4.8.5
$P_{L,CC} = E_p \cdot \frac{N_T \cdot N_s \cdot A_s}{A} \cdot P' \cdot \left(1 + \frac{(5 \cdot \text{MAX}(e_{HOG} , e_{SAG}) / 8)^2 \cdot A_{(SLS/ULS)}}{I_{(SLS/ULS)}} \right) \cdot \frac{\phi}{E_{\text{uncracked},28}}$				
Creep coefficient, ϕ			2.0	
Note ϕ usually 1.8 for 3-day transfer, 1.4 for 28-day transfer for UK outdoor exposure conditions				
		Note the effective thickness is taken as twice the cross sectional area divided by the		
Prestress Force Loss due to Tendon Relaxation Under Sustained Tension (Long Term)				
Prestress force loss due to tendon relaxation, $P_{L,TR}$			156 kN	cl.4.8.2
$P_{L,TR} = \text{MAX} \left(0.0, 0.08 - 0.08 \frac{70\% - P'/(N_T \cdot N_s \cdot F_{pk}) \times 100\%}{70\% - 40\%} \right) \cdot P'$				
Prestress Force Loss due to Concrete Shrinkage (Long Term)				
Prestress force loss due to concrete shrinkage, $P_{L,CS}$			31 kN	cl.4.8.4
$P_{L,CS} = \epsilon_{sh} \cdot E_p \cdot N_T \cdot N_s \cdot A_s$				
Shrinkage strain, ϵ_{sh}			100 $\times 10^{-6}$	
Note ϵ_{sh} usually 100×10^{-6} for UK outdoor exposure conditions;				
Note ϵ_{sh} usually 300×10^{-6} for UK indoor exposure conditions;				
		Note the effective thickness is taken as twice the cross sectional area divided by the		

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					jXXX	12			
					Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab				Drg. Ref.				
Member Design - PC Beam and Slab					Made by	XX	Date	24/10/2026	Chd.
								BS8110	
Input Summary								BS8110 ▼	
Item									
Job Title					BS110 (EC2), ACI318, AS3600 v2024.05.xlsm				
Calc Title					Member Design - PC Beam and Slab				
					TLS SLS/ULS)				
Concrete Grade (Cube)					C25	C35			
Concrete Grade (Cylinder)					C20	C28			
Pre-T or Post-T ? Bonded or Unbonded ?					Post-Tension	Bonded			
Serviceability Class					Class 3 (Partial Prestressing)				
Span and Available Beam Spacing					10.000	5.000	m		
Support Condition					Continuous				
					TLS SLS/ULS)				
Type of Construction					Type IV - Insitu Beam				
Section Type					T -	T -	section		
Section					500 x 1000 mm				
Flange Thickness					200	200	mm		
Flange Effective Width					1901	1901	mm		
					HOG SAG				
Prestress Tendon(s)					1 layer(s) x 1 tendons x 12T15.24 Unbanded				
Prestress Tendon(s) Jacking %						75.0	%		
Prestress Force at TLS, $P_{0,free}$ and P_0					2346	2346	kN		
Prestress Force at TLS, P'					10% losses	2116	4231	kN kN/m	
Prestress Force at SLS, KP_0					22% losses	1820	3640	kN kN/m	
Tendon Rebar Quantity					26.4	kg/m ²	161.8	kg/m ²	
Tendon(s) Profile, q_1 , q_2 and q_3					840	120	840	mm	
Tendon(s) Profile, p_1 and p_2						10%L	10%L		
Tendon Termination at $e_{var}(x=0/L)$?						No	No		
					$\omega_{TLS,E/E}$	ω_{DL+SDL}	$\omega_{SLS,E/E}$		
% of Load Balancing at SLS, $ \omega_{SLS,E/L} $					374%	119%	82%		
					HOG SAG				
Longitudinal Steel					2x5T20	Unbanded	1x5T25	Unbanded	
Shear and Punching Shear Links No.					2T10-100	N/A	N/A	N/A	
Shear and Punching Shear Links Area					3,142	N/A	N/A	N/A	
End Block Links, Width, Zone Length					2T16-150	500	1000	mm	
Flange Transverse Reinforcement							785	mm ² /m	
Column Section Type and Size					Rectangular: -		800 x 800	mm	
Column Head $d_h \times l_{hface}$							0 x 0	mm	
Column Position and Orientation							Interior		
Design Strip Direction							Along h		
Punching Shear $M_0V_{ult}/ M_{ult} $?							Include		
DL+SDL [G] and LL [Q] Factors for ULS					1.40	1.60			
DL [S] and P' Factors for TLS					1.00	1.15			
Pattern Loading Sag Factor for ULS						1.00			
					{h} {v}				
Dead Load					4.50	0.00	kPa		
Superimposed Dead Load					15.00	0.00	kPa		
Live Load					10.00	0.00	kPa		
Loading Tributary Width						5.000	m		
Elastic or Redistributed Effects					Elastic Effects				

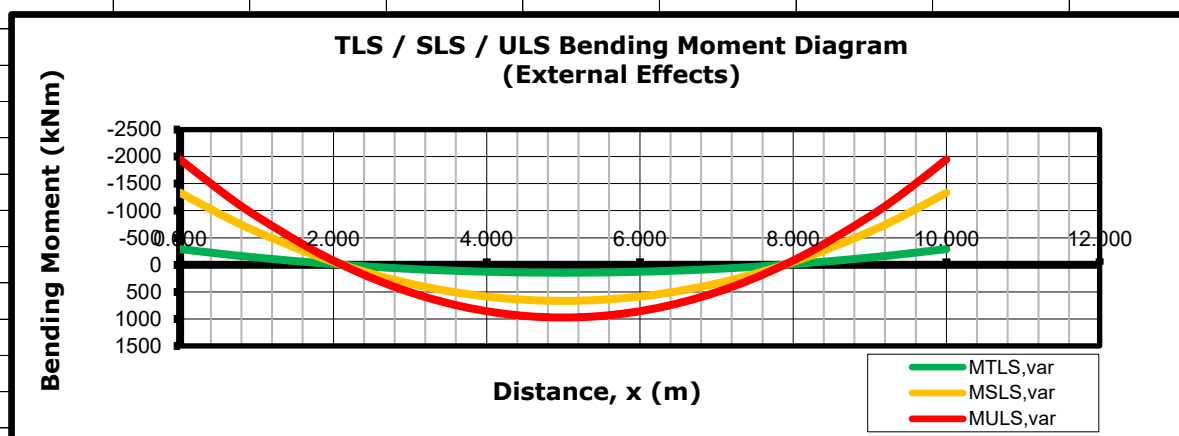
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.			
						jXXX	13				
						Member/Location					
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.					
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026		
						BS8110					
Utilisation Summary						BS8110 ▼					
Item						UT	Status	Overall			
Min beam top elastic section modulus at design section, Z_t						45%	OK	100%			
Min beam bottom elastic section modulus at design section, Z_b						54%	OK				
Physical eccentricity of prestress tendon(s) at design section, e						100%	OK				
Physical eccentricity of prestress tendon(s) at all sections, e_{var}						100%	OK				
Percentage of load balancing at SLS						82%	OK				
Max allowable prestress force at transfer (w. restraint, w. ST losses), P'						90%	OK				
Rect. or flgd. beam allowable range of P_0 (for given e) at design section						72%	OK				
Rect. or flgd. beam SLS stress at top at design section, f_t						54%	OK				
Rect. or flgd. beam TLS stress at top at design section, f'_t						44%	OK				
Rect. or flgd. beam SLS stress at bottom at design section, f_b						45%	OK				
Rect. or flgd. beam TLS stress at bottom at design section, f'_b						82%	OK				
Rect. or flgd. beam TLS and SLS minimum average precompression						54%	OK				
Rect. or flgd. beam TLS and SLS maximum average precompression						29%	OK				
Rect. or flgd. beam allowable range of ecc. (for given P_0) at design section, e						79%	OK				
Rect. or flgd. beam allowable range of ecc. (for given P_0) at all sections, e_{var}						79%	OK				
Rect. or flgd. beam end block design						83%	OK				
Rect. or flgd. beam deflection requirements						21%	OK				
Rect. or flgd. beam bending design capacity						Ductile	Converged			84%	OK
Rect. or flgd. beam bending design capacity						Ductile				98%	OK
Rect. or flgd. beam bending design capacity						Not Ductile	Converged			63%	OK
Rect. or flgd. beam bending design capacity						Not Ductile				78%	OK
Rect. or flgd. beam bending design capacity at all sections						Converged				63%	OK
Rect. beam shear ultimate stress at critical section										64%	OK
Rect. beam shear design capacity at (shear) design section										72%	OK
Rect. beam shear design capacity at all sections										72%	OK
Rect. beam punching shear at column face perimeter										N/A	OK
Rect. beam punching shear at column 1st shear perimeter						N/A				N/A	N/A
Rect. beam punching shear at column 2nd shear perimeter						N/A				N/A	N/A
Rect. beam punching shear at column 3rd shear perimeter						N/A				N/A	N/A
Rect. beam punching shear at column 4th shear perimeter						N/A				N/A	N/A
Detailing requirements								OK			
Note calculate UTs for cases with / without restraint to prestress force;											
Overall utilisation						Hogging Moment ▼	100%		Bending		
Inclusion of restraint to prestress force, ΣH_i						Exclude ▼			Pch. Shear		
Inclusion of prestress force losses, K						Include ▼			OK		
Inclusion of secondary effects ?						Include ▼			OK		
% Tensioned reinforcement (rectangular or flanged)							0.34	%			
$7850 \cdot [(N_T \cdot N_s \cdot A_s) / b_w \cdot h];$											
% Untensioned reinforcement (rectangular or flanged) hog / sag						0.63	0.49	%			
$7850 \cdot [(A_{s,prov,h} + A_{s,prov,s}) / b_w \cdot h + (A_{sv,prov} \cdot (h + b_w) / S) / b_w \cdot h];$ No curtailment; No laps;											
Estimated tensioned reinforcement quantity							26	kg/m ³			
Estimated untensioned reinforcement quantity						49	39	74	162		
[Note that steel quantity in kg/m ³ can be obtained from 78.5 x % tendon/rebar];											
Estimated tendon steel reinforcement quantity (slabs 25 25kg/m ³ , transfer slabs 25 50kg/m ³ , beams 50 50kg/m ³)											
Material concrete, c						315	units/m ³	tendon, t	12500		
								steel, s	3600		
Reinforced concrete material cost = $[c + (\text{est. tendon quant}) \cdot t + (\text{est. rebar quant}) \cdot s]$							614	units/m			
Degree of partial prestressing, $PI = N_T \cdot N_s \cdot A_s \cdot f_{pk} / [N_T \cdot N_s \cdot A_s \cdot f_{pk} + A_{s,prov} \cdot f_y]$							68%				
Degree of partial prestressing, $PPR = M_{u,PT} / M_{u,PT+RC}$							79%				
Max LTB stability (compression flange) restraints spacing, L_{LTB}							114.0	m	4.1.6 BS8.		
Note s/s / cont $L_{LTB} = \text{MIN} (60(b_w \text{ or } b), 250(b_w \text{ or } b)^2 / h)$ and cant $L_{LTB} = \text{MIN} (25b_w, 100b_w^2 / h);$											

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.	
						jXXX	14		
						Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.			
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026
						BS8110			
						BS8110 ▼			
Additional Longitudinal Shear Rectangular or Flanged Beam Utilisation Summary									
Longitudinal shear between web and flange						Consider	▼	OK	
Longitudinal shear within web						Consider	▼	OK	
Length under consideration, Δx (span/2 s/s, ~span/4 cont, span cant)						2778	mm		
Applicability of longitudinal shear design						Applicable			
Longitudinal Shear Between Web and Flange (EC2)									
Longitudinal shear stress limit to prevent crushing						48%		OK	
Longitudinal shear stress limit for no transverse reinforcement						404%		NOT OK	
Required design transverse reinforcement per unit length						77%		OK	
Longitudinal Shear Between Web and Flange (BS5400-4)									
Longitudinal shear force limit per unit length						83%		OK	
Required nominal transverse reinforcement per unit length						38%		OK	
Longitudinal Shear Between Web and Flange Mandatory Criteria						83%		OK	
Longitudinal Shear Within Web (EC2)									
Longitudinal shear stress limit						89%		OK	
Longitudinal Shear Within Web (BS8110)									
Longitudinal shear stress limit for no nominal / design vertical reinforcement						96%		OK	
Required nominal vertical reinforcement per unit length						24%		OK	
Required design vertical reinforcement per unit length						0%		OK	
Longitudinal Shear Within Web (BS5400-4)									
Longitudinal shear force limit per unit length						69%		OK	
Required nominal vertical reinforcement per unit length						24%		OK	
Longitudinal Shear Within Web Mandatory Criteria						89%		OK	
Additional Input Parameters Requirements Rectangular or Flanged Beam									
Characteristic strength of concrete (PT beam and slab), f_{cu} and f_{ci}						OK			
Characteristic strength of concrete (column), f_{cu}						OK			
Type of concrete and density, ρ_c						OK			
Creep modulus factor, C_{MF}						N/A			
Prestress tendon(s) bonded or unbonded (post-tension only) ?						N/A			
Flat slab hogging moment stress concentration						N/A			
Flexural tensile stresses, cracked (internal building) crack width						N/A			
Span, L						OK			
Section type at TLS and (SLS/ULS)						OK			
Design section hogging or sagging moment ?						OK			
Overall depth, h						OK			
Additional bottom compressive stress						N/A			
Banding of prestress tendons						OK			
Banding of longitudinal steel (hogging/sagging)						OK			
Number of layers of untensioned steel, $n_{layers,tens}$						OK			
Load (on plan), $\{DL_h, DL_v, SDL_h, SDL_v, LL_h, LL_v\}$ and UDL, $\{ULS_{construction}\}$						OK			
Percentage of tensile capacity, %						OK			
Inclusion of prestress force losses, K						OK			
Inclusion of secondary effects ?						OK			
Longitudinal shear between web and flange						OK			
Longitudinal shear within web						OK			
Horizontal anchorage edge distance and spacing						OK			
Vertical anchorage edge distance and spacing						OK			
110									

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	15			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
						BS8110				
Action Effects From Structural Analysis (External Effects)						BS8110 ▼				
Note that moment redistribution to cl.4.2.3 BS8110 is not performed herein; Note ω positive downwards;										
Span, L						10.000		m		
TLS beam loading, $\omega_{TLS,E/E}$						35.0		kN/m		
SLS beam loading, $\omega_{SLS,E/E}$						160.0		kN/m		
ULS beam loading, $\omega_{ULS,E/E}$						234.0		kN/m		
Simply Supported						N/A				
TLS	$M_{HOG,TLS,E/E}=0$					N/A		kNm		
	$M_{SAG,TLS,E/E}=[0.125(\omega_{TLS,E/E})] \cdot L^2 + DL_{point,h} \cdot a_h \cdot (L-a_h)/L$					N/A		kNm		
	$V_{TLS,E/E}=[0.500(\omega_{TLS,E/E})] \cdot L + DL_{point,h} \cdot (L-a_h)/L$					N/A		kN		
SLS	$M_{HOG,SLS,E/E}=0$					N/A		kNm		
	$M_{SAG,SLS,E/E}=[0.125(\omega_{SLS,E/E})] \cdot L^2 + DL_{point,h} \cdot a_h \cdot (L-a_h)/L + DL_{point,v} \cdot (L-a_v)/L$					N/A		kNm		
	$V_{SLS,E/E}=[0.500(\omega_{SLS,E/E})] \cdot L + DL_{point,h} \cdot (L-a_h)/L + DL_{point,v} \cdot (L-a_v)/L$					N/A		kN		
ULS	$M_{HOG,ULS,E/E}=0$					N/A		kNm		
	$M_{SAG,ULS,E/E}=[0.125(\omega_{ULS,E/E})] \cdot L^2 + k_G \cdot DL_{point,h} \cdot a_h \cdot (L-a_h)/L + k_G \cdot DL_{point,v} \cdot (L-a_v)/L$					N/A		kNm		
	$V_{ULS,E/E}=[0.500(\omega_{ULS,E/E})] \cdot L + k_G \cdot DL_{point,h} \cdot (L-a_h)/L + k_G \cdot DL_{point,v} \cdot (L-a_v)/L$					N/A		kN		
Continuous (Infinitely, Encastre)						Continues ▼	Continues ▼	VALID		Note
TLS	$M_{HOG,TLS,E/E}=-[\% \times 0.083(\omega_{TLS,E/E})] \cdot L^2 - [\% \times f] \cdot (L/2)^2/2$					100%	-292	kNm		
	$M_{SAG,TLS,E/E}=M_{HOG,TLS,E/E} + V_{TLS,E/E} \cdot [L/2] - \omega_{TLS,E/E} \cdot (L/2)^2/2$						146	kNm		
	$V_{TLS,E/E}=[\% \times 0.500(\omega_{TLS,E/E})] \cdot L + [100\%] \cdot (L/2)$					100%	175	kN		
SLS	$M_{HOG,SLS,E/E}=-[\% \times 0.083(\omega_{SLS,E/E})] \cdot L^2 - [\% \times f] \cdot (L/2)^2/2$					100%	-1333	kNm		Goal Seek BMD
	$M_{SAG,SLS,E/E}=M_{HOG,SLS,E/E} + V_{SLS,E/E} \cdot [L/2] - \omega_{SLS,E/E} \cdot (L/2)^2/2$						667	kNm		
	$V_{SLS,E/E}=[\% \times 0.500(\omega_{SLS,E/E})] \cdot L + [100\%] \cdot (L/2)$					100%	800	kN		
ULS	$M_{HOG,ULS,E/E}=-[\% \times 0.083(\omega_{ULS,E/E})] \cdot L^2 - [\% \times k] \cdot (L/2)^2/2$					100%	-1950	kNm		Goal Seek Shear
	$M_{SAG,ULS,E/E}=k_{PAT} \cdot [M_{HOG,ULS,E/E} + V_{ULS,E/E} \cdot [L/2] - \omega_{ULS,E/E} \cdot (L/2)^2/2]$						975	kNm		
	$V_{ULS,E/E}=[\% \times 0.500(\omega_{ULS,E/E})] \cdot L + [100\%] \cdot (L/2)$					100%	1170	kN		
Cantilever						N/A				
TLS	$M_{HOG,TLS,E/E}=-[0.500(\omega_{TLS,E/E})] \cdot L^2 - DL_{point,h} \cdot a_h$					N/A		kNm		
	$M_{SAG,TLS,E/E}=0$					N/A		kNm		
	$V_{TLS,E/E}=[(\omega_{TLS,E/E})] \cdot L + DL_{point,h}$					N/A		kN		
SLS	$M_{HOG,SLS,E/E}=-[0.500(\omega_{SLS,E/E})] \cdot L^2 - DL_{point,h} \cdot a_h - DL_{point,v} \cdot a_v$					N/A		kNm		
	$M_{SAG,SLS,E/E}=0$					N/A		kNm		
	$V_{SLS,E/E}=[(\omega_{SLS,E/E})] \cdot L + DL_{point,h} + DL_{point,v}$					N/A		kN		
ULS	$M_{HOG,ULS,E/E}=-[0.500(\omega_{ULS,E/E})] \cdot L^2 - k_G \cdot DL_{point,h} \cdot a_h - k_G \cdot DL_{point,v} \cdot a_v$					N/A		kNm		
	$M_{SAG,ULS,E/E}=0$					N/A		kNm		
	$V_{ULS,E/E}=[(\omega_{ULS,E/E})] \cdot L + k_G \cdot DL_{point,h} + k_G \cdot DL_{point,v}$					N/A		kN		
Design section hogging or sagging moment ?						Hogging Moment				
TLS bending moment at design section, $M_{HOG/SAG,TLS,E/E}$						-292		kNm		
SLS bending moment at design section, $M_{HOG/SAG,SLS,E/E}$						-1333		kNm		
ULS bending moment at design section, $M_{HOG/SAG,ULS,E/E}$						-1950		kNm		
Note that unlike shear force, the bending moment is presented for the design section be it hogging or sagging. Note by convention, a negative bending moment indicates hogging moment;										
TLS shear force at critical section, $V_{TLS,E/E}$						175		kN		
SLS shear force at critical section, $V_{SLS,E/E}$						800		kN		
ULS shear force at critical section, $V_{ULS,E/E}$						1170		kN		
Note that unlike bending moment, the shear force is presented for the critical section irrespective of whether the design section is hogging or sagging. Note an arbitrary sign convention applicable;										

TLS / SLS / ULS Bending Moment Diagram (kNm)								
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
M _{TLS,E/E,var}	-292	-209	-134	-24	59	115	142	kNm
M _{SLS,E/E,var}	-1333	-953	-613	-108	272	524	651	kNm
M _{ULS,E/E,var}	-1950	-1394	-897	-157	397	767	952	kNm
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
M _{TLS,E/E,var}	142	115	59	-24	-134	-209	-292	kNm
M _{SLS,E/E,var}	651	524	272	-108	-613	-953	-1333	kNm
M _{ULS,E/E,var}	952	767	397	-157	-897	-1394	-1950	kNm

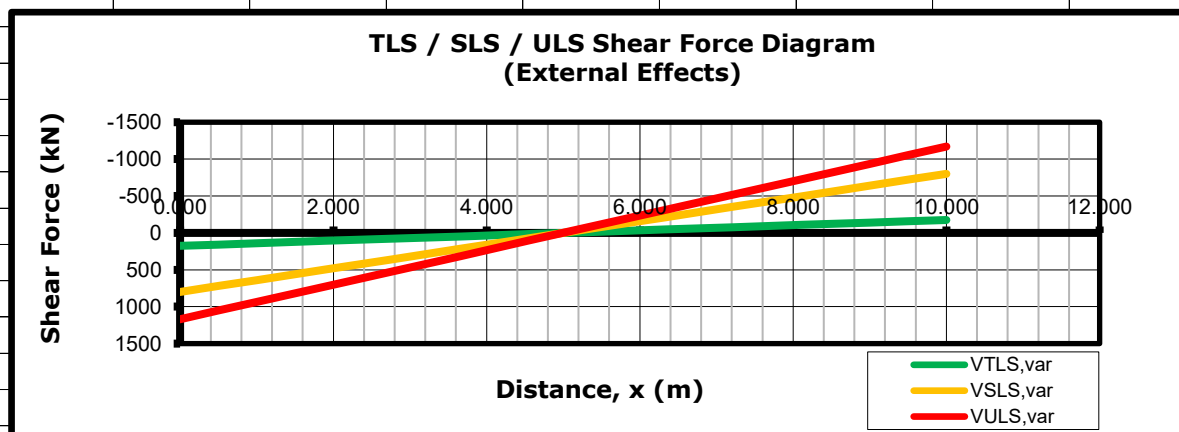
Note by convention, a negative bending moment indicates hogging moment;



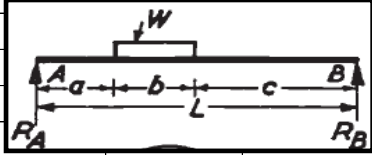
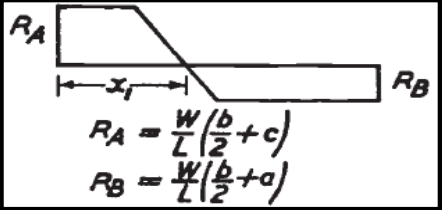
Note by convention, a negative bending moment indicates hogging moment;

	TLS / SLS / ULS Shear Force Diagram (kN)							
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
V _{TLS,E/E,var}	175	158	140	109	78	47	16	kN
V _{SLS,E/E,var}	800	720	640	498	356	213	71	kN
V _{ULS,E/E,var}	1170	1053	936	728	520	312	104	kN
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
V _{TLS,E/E,var}	-16	-47	-78	-109	-140	-158	-175	kN
V _{SLS,E/E,var}	-71	-213	-356	-498	-640	-720	-800	kN
V _{ULS,E/E,var}	-104	-312	-520	-728	-936	-1053	-1170	kN

Note an arbitrary shear force sign convention is employed;



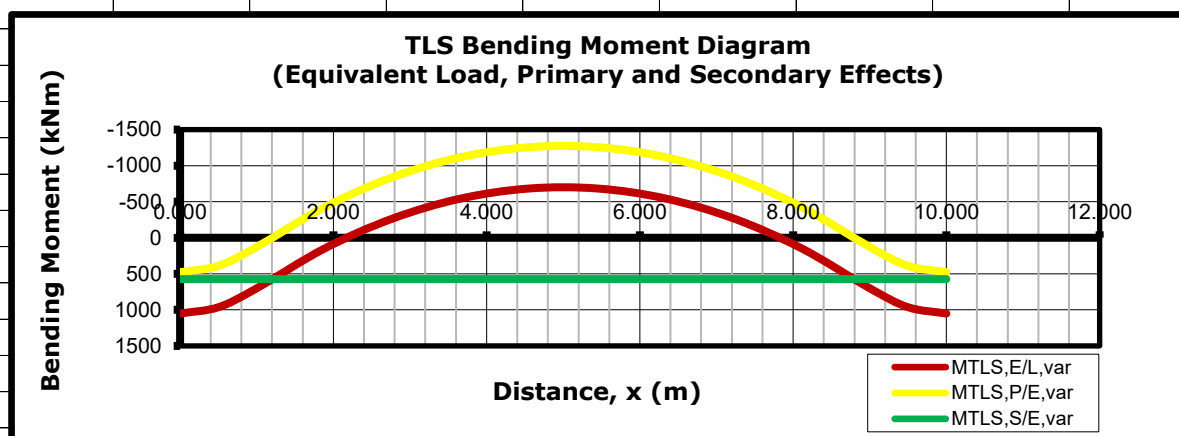
Note an arbitrary shear force sign convention is employed;

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
			jXXX	17	
			Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab		Drg. Ref.		
Member Design - PC Beam and Slab			Made by	XX	Date 24/10/2026 Chd.
					BS8110
Action Effects From Structural Analysis (Equivalent Load, Primary and Secondary Effects)					BS8110 ▼
Note that moment redistribution is not applicable herein; Note ω positive downwards;					
Span, L				10.000	m
			L-Sup	Span	R-Sup
Distance between points of inflexion, s		2.000	8.000	2.000	m
Total drap between points of inflexion, e_d		144	576	144	mm
TLS equivalent load, $\omega_{TLS,E/L} = \pm[8 2]k_p P' \cdot e_d / s^2$		700.7	-175.2	700.7	kN/m
SLS equivalent load, $\omega_{SLS,E/L} = \pm[8 2]K P_0 \cdot e_d / s^2$		524.1	-131.0	524.1	kN/m
Note that the equivalent load calculation includes the support peak tendon reverse curvature;					
Dimensions, $\{p_1, L-p_1-p_2, p_2\}$		1.000	8.000	1.000	m
Σ SLS equivalent load, $\Sigma\{p_1, L-p_1-p_2, p_2\} \cdot \omega_{SLS,E/L}$		524	-1048	524	kN
Inclusion of secondary effects ?				Include ▼	
Simply Supported, Continuous (Infinitely, Encastre), Cantilever					
Note primary effects P/E and secondary effects S/E equations: -					
P/E	$M_{HOG,TLS,P/E} = -k_p P' \cdot e_{HOG}$	$M_{HOG,SLS,P/E} = -K P_0 \cdot e_{HOG}$			
	$M_{SAG,TLS,P/E} = -k_p P' \cdot e_{SAG}$	$M_{SAG,SLS,P/E} = -K P_0 \cdot e_{SAG}$			
	$V_{TLS,P/E} \approx dM_{TLS,P/E} / dx$	$V_{SLS,P/E} \approx dM_{SLS,P/E} / dx$			
S/E	$M_{HOG,TLS/SLS,S/E} = M_{HOG,TLS/SLS,E/L} - M_{HOG,TLS,S/P/E}$				
	$M_{SAG,TLS/SLS,S/E} = M_{SAG,TLS/SLS,E/L} - M_{SAG,TLS,S/P/E}$				
	$V_{TLS/SLS,S/E} = V_{TLS/SLS,E/L} - V_{TLS/SLS,P/E}$				
Note method of calculating S/E from the reactions of E/L not adopted herein;					Note
Simply Supported					N/A
Note statically determinate structures do not exhibit secondary effects;					
			E/L	P/E	S/E
TLS	$M_{HOG,TLS}$		N/A	N/A	N/A kNm
	$M_{SAG,TLS}$		N/A	N/A	N/A kNm
	V_{TLS}		N/A	N/A	N/A kN
SLS / ULS	$M_{HOG,SLS}$		N/A	N/A	N/A kNm
	$M_{SAG,SLS}$		N/A	N/A	N/A kNm
	V_{SLS}		N/A	N/A	N/A kN
Note equivalent load effects E/L equations: -					
E/L	$M_{HOG,TLS/SLS,E/L} = 0 - [k_p P' \text{ or } K P_0] \cdot e_{var}(x=0)$				
	$M_{SAG,TLS/SLS,E/L} = 0 + V_{TLS/SLS,E/L} \cdot L/2 - f[\omega_{TLS/SLS,E/L}, x=L/2] - [k_p P' \text{ or } K P_0] \cdot e_{var}(x=0) + [k_p P' \text{ or } K P_0] \cdot e_{var}(x=L)/L$				
	$V_{TLS/SLS,E/L} = f[\omega_{TLS/SLS,E/L}, x=0] + [k_p P' \text{ or } K P_0] \cdot e_{var}(x=0)/L - [k_p P' \text{ or } K P_0] \cdot e_{var}(x=L)/L$				
					
Note for simplicity, E/L effects due to any change of section not computed;					Note
Continuous (Infinitely, Encastre)					VALID
Note statically indeterminate structures do exhibit secondary effects;					
			E/L	P/E	S/E
TLS	$M_{HOG,TLS}$	100%	1051	478	573 kNm
	$M_{SAG,TLS}$		-701	-1274	573 kNm
	V_{TLS}	100%	0	-175	175 kN
SLS / ULS	$M_{HOG,SLS}$	100%	786	357	429 kNm
	$M_{SAG,SLS}$		-524	-953	429 kNm
	V_{SLS}	100%	0	-131	131 kN
					Goal Seek BMD

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
			jXXX	18	
			Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab		Drg. Ref.		
Member Design - PC Beam and Slab			Made by	Date	Chd.
			XX	24/10/2026	
					BS8110
					BS8110 ▼
Note equivalent load effects E/L equations: -					
E/L	$M_{HOG,TLS/SLS,E/L} = -\% \times f[\omega_{TLS/SLS,E/L}, X=0] - [k_p P' \text{ or } KP_0].e_{var}(X=0)$				
	$M_{SAG,TLS/SLS,E/L} = M_{HOG,TLS/SLS,E/L} + V_{TLS/SLS,E/L} \cdot L/2 - f[\omega_{TLS/SLS,E/L}, X=L/2] - [k_p P' \text{ or } KP_0].e_{var}(X=L/2)$				
	$V_{TLS/SLS,E/L} = \% \times f[\omega_{TLS/SLS,E/L}, X=0] + [k_p P' \text{ or } KP_0].e_{var}(X=0)/L - [k_p P' \text{ or } KP_0].e_{var}(X=L)/L$				
Tendon termination at x=0 ?			Continues ▼		
Tendon termination at x=L ?			Continues ▼		
$R_1 = \frac{qd}{L^3} \left[(2a+L)b^2 + \left(\frac{a-b}{4}\right)d^2 \right]$ $R_2 = \frac{qd}{L^3} \left[(2b+L)a^2 - \left(\frac{a-b}{4}\right)d^2 \right]$ $R_A = r_A + \frac{MA}{L} \quad R_B = r_B - \frac{MA}{L}$ Where r_A and r_B are the simple support reactions for the beam (MA being considered positive)					
Note for simplicity, E/L effects due to any change of section not computed;					Note
Cantilever			N/A		
Note statically determinate structures do not exhibit secondary effects;					
			E/L	P/E	S/E
TLS	$M_{HOG,TLS}$		N/A	N/A	N/A kNm
	$M_{SAG,TLS}$		N/A	N/A	N/A kNm
	V_{TLS}		N/A	N/A	N/A kN
SLS / ULS	$M_{HOG,SLS}$		N/A	N/A	N/A kNm
	$M_{SAG,SLS}$		N/A	N/A	N/A kNm
	V_{SLS}		N/A	N/A	N/A kN
Note equivalent load effects E/L equations: -					
E/L	$M_{HOG,TLS/SLS,E/L} = -f[\omega_{TLS/SLS,E/L}, X=0]$				
	$M_{SAG,TLS/SLS,E/L} = M_{HOG,TLS/SLS,E/L} + V_{TLS/SLS,E/L} \cdot L - f[\omega_{TLS/SLS,E/L}, X=L] - [k_p P' \text{ or } KP_0].e_{var}(X=L)$				
	$V_{TLS/SLS,E/L} = f[\omega_{TLS/SLS,E/L}, X=0] - [k_p P' \text{ or } KP_0].e_{var}(X=0)/L - [k_p P' \text{ or } KP_0].e_{var}(X=L)/L$				
$M_{max.} = W(a + \frac{b}{2})$ $R_A = W$					
Note for simplicity, E/L effects due to any change of section not computed;					Note
Design section hogging or sagging moment ?			Hogging Moment		
TLS S/E bending moment at design section, $M_{HOG/SAG,TLS,S/E}$			573 kNm		
SLS S/E bending moment at design section, $M_{HOG/SAG,SLS,S/E}$			429 kNm		
Note that unlike shear force, the bending moment is presented for the design section be it hogging or sagging. Note by convention, a negative bending moment indicates hogging moment;					
TLS S/E shear force at critical section, $V_{TLS,S/E}$			175 kN		
SLS S/E shear force at critical section, $V_{SLS,S/E}$			131 kN		
Note that unlike bending moment, the shear force is presented for the critical section irrespective of whether the design section is hogging or sagging. Note an arbitrary sign convention applicable;					

[illegible]

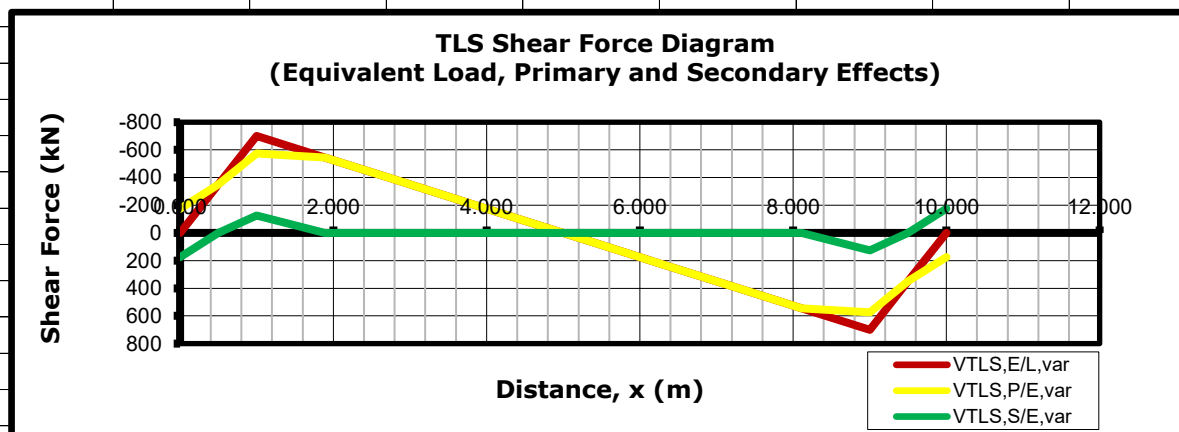
Note by convention, a negative bending moment indicates hogging moment;



Note by convention, a negative bending moment indicates hogging moment;

TLS Shear Force Diagram (kN)								
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$V_{\text{TLS,E/L,var}}$	0	-350	-701	-545	-389	-234	-78	kN
$V_{\text{TLS,P/E,var}}$	-175	-350	-574	-545	-389	-234	-78	kN
$V_{\text{TLS,S/E,var}}$	175	0	-127	0	0	0	0	kN
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
$V_{\text{TLS,E/L,var}}$	78	234	389	545	701	350	0	kN
$V_{\text{TLS,P/E,var}}$	78	234	389	545	574	350	175	kN
$V_{\text{TLS,S/E,var}}$	0	0	0	0	127	0	-175	kN

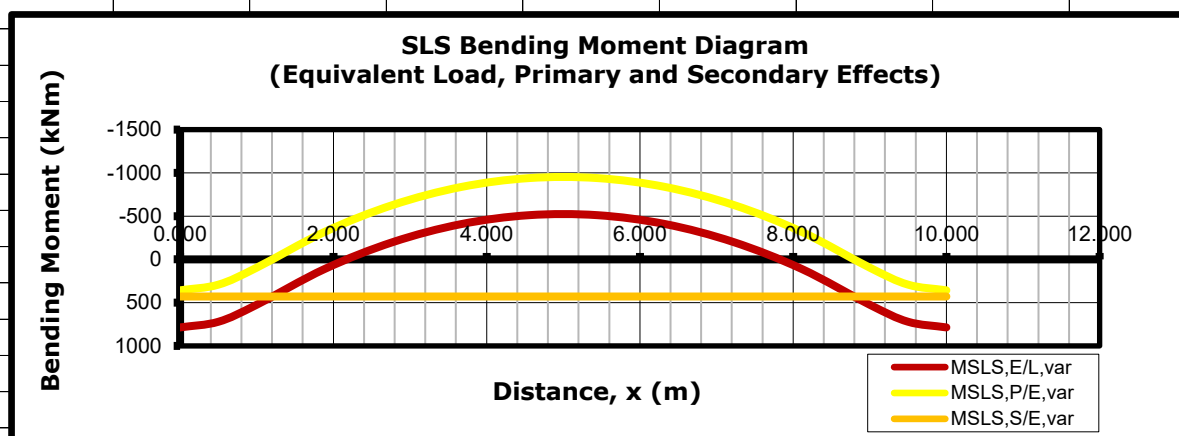
Note an arbitrary shear force sign convention is employed;



Note an arbitrary shear force sign convention is employed;

[illegible]

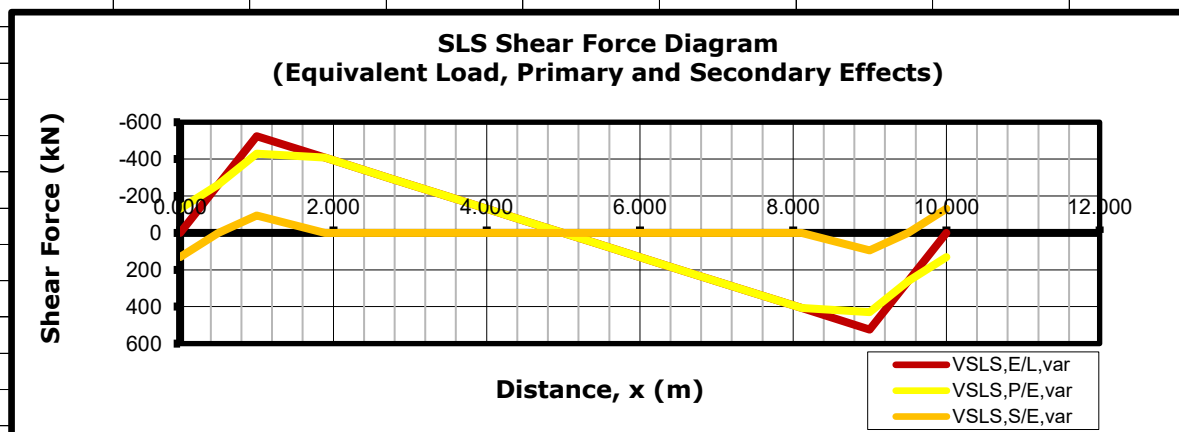
Note by convention, a negative bending moment indicates hogging moment;



Note by convention, a negative bending moment indicates hogging moment;

	SLS Shear Force Diagram (kN)							
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$V_{SLS,E/L,var}$	0	-262	-524	-408	-291	-175	-58	kN
$V_{SLS,P/E,var}$	-131	-262	-430	-408	-291	-175	-58	kN
$V_{SLS,S/E,var}$	131	0	-95	0	0	0	0	kN
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
$V_{SLS,E/L,var}$	58	175	291	408	524	262	0	kN
$V_{SLS,P/E,var}$	58	175	291	408	430	262	131	kN
$V_{SLS,S/E,var}$	0	0	0	0	95	0	-131	kN

Note an arbitrary shear force sign convention is employed;



Note an arbitrary shear force sign convention is employed;

Job Title	Member Design - Prestressed Concrete Beam and Slab
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Member Design - PC Beam and Slab

Drg. Ref.	
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Made by	xx
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Date	24/10/2026
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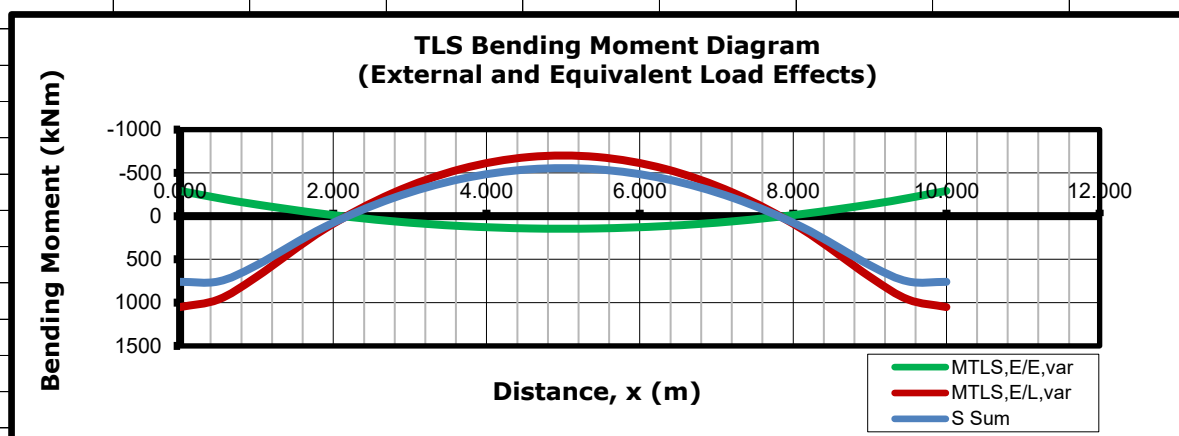
BS8110

Action Effects From Structural Analysis (External, Equivalent Load and Secondary Effects)

BS8110

		TLS Bending Moment Diagram (kNm)						
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$M_{TLS,E/E,var}$	-292	-209	-134	-24	59	115	142	kNm
$M_{TLS,E/L,var}$	1051	963	701	147	-268	-545	-683	kNm
Σ Sum	759	755	566	124	-209	-430	-541	kNm
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
$M_{TLS,E/E,var}$	142	115	59	-24	-134	-209	-292	kNm
$M_{TLS,E/L,var}$	-683	-545	-268	147	701	963	1051	kNm
Σ Sum	-541	-430	-209	124	566	755	759	kNm

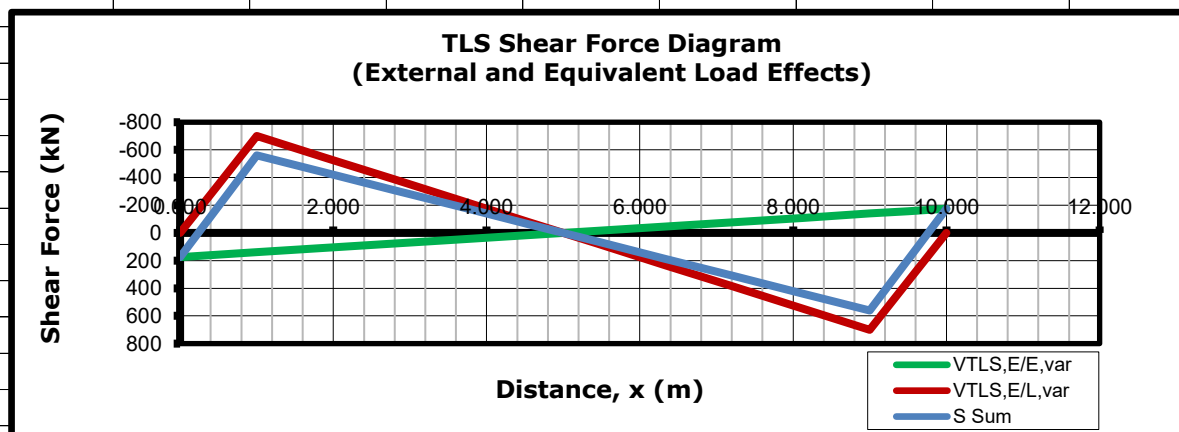
Note by convention, a negative bending moment indicates hogging moment;



Note by convention, a negative bending moment indicates hogging moment;

		TLS Shear Force Diagram (kN)						
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$V_{\text{TLS,E/E,var}}$	175	158	140	109	78	47	16	kN
$V_{\text{TLS,E/L,var}}$	0	-350	-701	-545	-389	-234	-78	kN
Σ Sum	175	-193	-561	-436	-311	-187	-62	kN
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
$V_{\text{TLS,E/E,var}}$	-16	-47	-78	-109	-140	-158	-175	kN
$V_{\text{TLS,E/L,var}}$	78	234	389	545	701	350	0	kN
Σ Sum	62	187	311	436	561	193	-175	kN

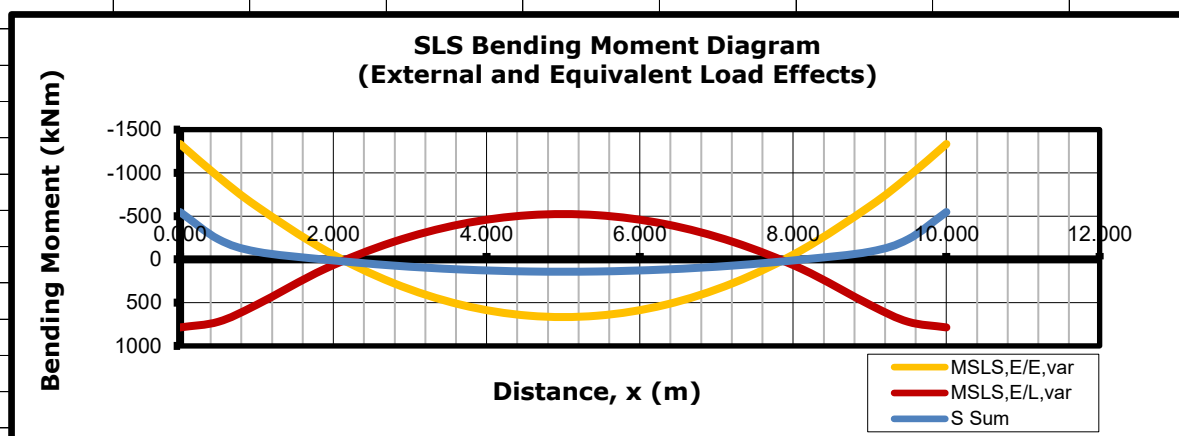
Note an arbitrary shear force sign convention is employed;



Note an arbitrary shear force sign convention is employed;

		SLS Bending Moment Diagram (kNm)						
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
M _{SLS,E/E,var}	-1333	-953	-613	-108	272	524	651	kNm
M _{SLS,E/L,var}	786	721	524	110	-201	-408	-511	kNm
Σ Sum	-547	-233	-89	2	71	117	140	kNm
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
M _{SLS,E/E,var}	651	524	272	-108	-613	-953	-1333	kNm
M _{SLS,E/L,var}	-511	-408	-201	110	524	721	786	kNm
Σ Sum	140	117	71	2	-89	-233	-547	kNm

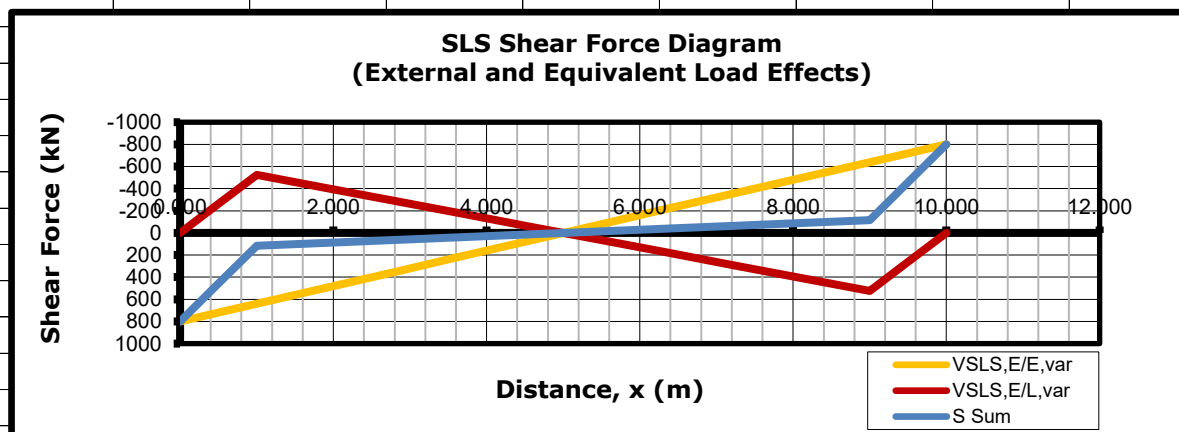
Note by convention, a negative bending moment indicates hogging moment;



Note by convention, a negative bending moment indicates hogging moment;

SLS Shear Force Diagram (kN)								
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$V_{SLS,E/E,var}$	800	720	640	498	356	213	71	kN
$V_{SLS,E/L,var}$	0	-262	-524	-408	-291	-175	-58	kN
Σ Sum	800	458	116	90	64	39	13	kN
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
$V_{SLS,E/E,var}$	-71	-213	-356	-498	-640	-720	-800	kN
$V_{SLS,E/L,var}$	58	175	291	408	524	262	0	kN
Σ Sum	-13	-39	-64	-90	-116	-458	-800	kN

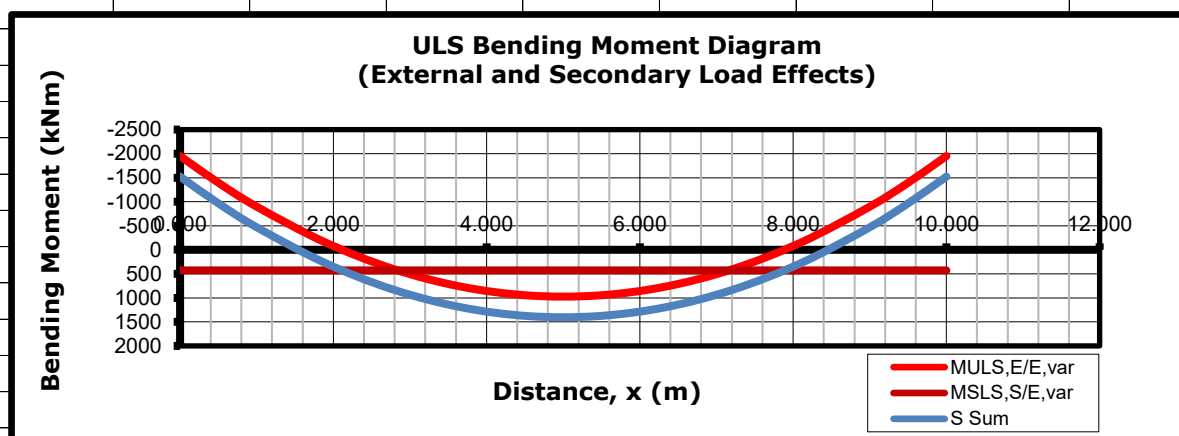
Note an arbitrary shear force sign convention is employed;



Note an arbitrary shear force sign convention is employed;

		ULS Bending Moment Diagram (kNm)						
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
M _{ULS,E/E,var}	-1950	-1394	-897	-157	397	767	952	kNm
M _{SLS,S/E,var}	429	429	429	429	429	429	429	kNm
Σ Sum	-1521	-965	-468	271	826	1196	1381	kNm
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
M _{ULS,E/E,var}	952	767	397	-157	-897	-1394	-1950	kNm
M _{SLS,S/E,var}	429	429	429	429	429	429	429	kNm
Σ Sum	1381	1196	826	271	-468	-965	-1521	kNm

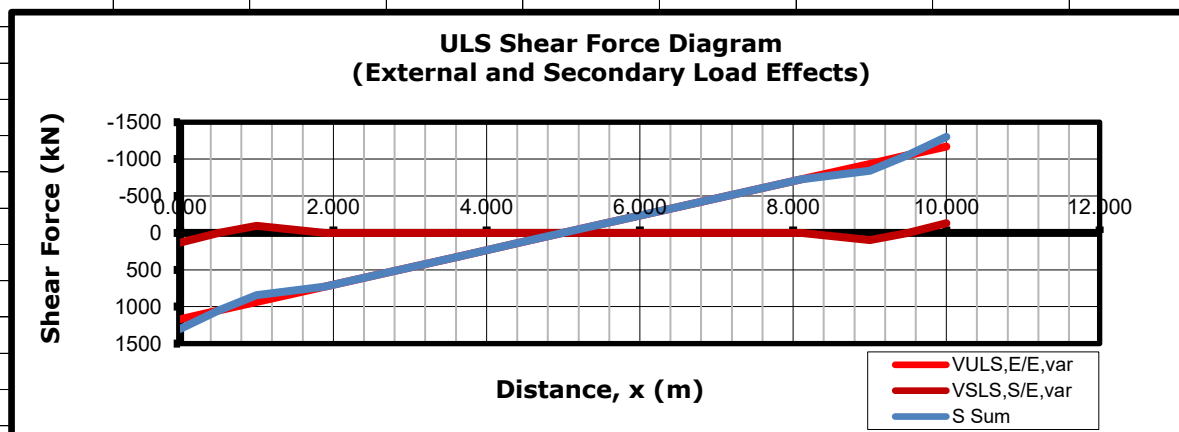
Note by convention, a negative bending moment indicates hogging moment;



Note by convention, a negative bending moment indicates hogging moment;

	ULS Shear Force Diagram (kN)							
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$V_{ULS,E/E,var}$	1170	1053	936	728	520	312	104	kN
$V_{SLS,S/E,var}$	131	0	-95	0	0	0	0	kN
Σ Sum	1301	1053	841	728	520	312	104	kN
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m
$V_{ULS,E/E,var}$	-104	-312	-520	-728	-936	-1053	-1170	kN
$V_{SLS,S/E,var}$	0	0	0	0	95	0	-131	kN
Σ Sum	-104	-312	-520	-728	-841	-1053	-1301	kN

Note an arbitrary shear force sign convention is employed;



Note an arbitrary shear force sign convention is employed;

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.	
						jXXX	24		
						Member/Location			
Job Title		Member Design - Prestressed Concrete Beam and Slab				Drg. Ref.			
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026
						Chd. BS8110			
Allowable Range of Prestress Force at Transfer (for Given Eccentricity) at Design Section BS8110 ▼									
		Sagging Moment		Invalid		Hogging Moment		Valid	
P_0	$\geq$	$\frac{(Z_t f_{t_{\max}} - M_{\max})}{K(Z_t / A - e_{\text{SAG}})}$		$\frac{(Z_t f_{t_{\min}} - M_{\max})}{K(Z_t / A - e_{\text{HOG}})}$		MIN		1414 kN	
Note A and Z_t in the above inequality refer to $A_{(SLS/ULS)}$ and $Z_{t,(SLS/ULS)}$ respectively;									
P_0	$\leq$	$\frac{(Z_t f_{t_{\min}}^I - M_{\min})}{(Z_t / A - e_{\text{SAG}}) k_P} + (P_{L,DF} + P_{L,ES})$				MAX		5920 kN	
		$\frac{(Z_t f_{t_{\max}}^I - M_{\min})}{(Z_t / A - e_{\text{HOG}}) k_P} + (P_{L,DF} + P_{L,ES})$							
Note A and Z_t in the above inequality refer to A_{TLS} and $Z_{t,TLS}$ respectively;									
P_0	$\geq$	$\frac{(Z_b f_{b_{\min}} + M_{\max})}{K(Z_b / A + e_{\text{SAG}})}$		$\frac{(Z_b f_{b_{\max}} + M_{\max})}{K(Z_b / A + e_{\text{HOG}})}$		MIN		-7116 kN	
Note A and Z_b in the above inequality refer to $A_{(SLS/ULS)}$ and $Z_{b,(SLS/ULS)}$ respectively;									
P_0	$\leq$	$\frac{(Z_b f_{b_{\max}}^I + M_{\min})}{(Z_b / A + e_{\text{SAG}}) k_P} + (P_{L,DF} + P_{L,ES})$				MAX		3280 kN	
		$\frac{(Z_b f_{b_{\min}}^I + M_{\min})}{(Z_b / A + e_{\text{HOG}}) k_P} + (P_{L,DF} + P_{L,ES})$							
Note A and Z_b in the above inequality refer to A_{TLS} and $Z_{b,TLS}$ respectively;									
Note by convention, e is positive downwards, measured from the centroid of the TLS/(SLS/ULS) section;									
Note that in the above inequalities, $M_{\min} = M_{TLS,E/E} + M_{TLS,S/E}$ and $M_{\max} = M_{SLS,E/E} + M_{SLS,S/E}$;									
Note that $k_P P' = k_P [P_0 - (P_{L,DF} + P_{L,ES})]$;									
Note that in the above inequalities, should the denominator be negative, the inequality is flipped;									
Allowable range of P_0 (for given e)		1414	$\leq$	2346	$\leq$	3280	kN		
Allowable range of P_0 (for given e) at design section utilisation		72%						OK	
Maximum Economic Upper Limit to Prestress Force at Transfer at Design Section Rectangu BS8110 ▼									
Max economic upper limit to prestress force at transfer (w. restraint, w.o. ST		5915						kN	
		Sagging Moment		Invalid		Hogging Moment		Valid	
$P_0 = \frac{f_{\max} Z_t + f_{\min} Z_b}{K \left(\frac{Z_b + Z_t}{A} \right)}$				$P_0 = \frac{f_{\min} Z_t + f_{\max} Z_b}{K \left(\frac{Z_b + Z_t}{A} \right)}$					
Note A, Z_t and Z_b in the above equation refer to $A_{(SLS/ULS)}$, $Z_{t,(SLS/ULS)}$ and $Z_{b,(SLS/ULS)}$ respectively;									
Eccentricity of prestress tendon(s) at $P_{0,ecomax}$, e_{ecomax}		62						mm	
Note by convention, e is positive downwards, measured from the centroid of the (SLS/ULS) section;									

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						jXXX	25			
						Member/Location				
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Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
						BS8110				
TLS and SLS Top and Bottom Stresses at Design Section Rectangular or Flanged Beam						BS8110 ▼				
SLS stress at top		$\sqrt{f_{cu}} / \sqrt{f_c'}$	-0.45	≤	-0.24	≤	0.33	f_{cu} / f_c'		
at design section, f_t			-2.7	≤	-1.4	≤	11.6	N/mm ²		
$f_{min} \leq \left[f_t = \frac{K P_0}{A} - \frac{K P_0 e}{Z_t} + \frac{M_{SLS,E/E}}{Z_t} + \frac{M_{SLS,S/E}}{Z_t} \right] \leq f_{max}$				1.3 - -1.8 + -6.7 + 2.1				N/mm ²		
Note A and Z_t in the above inequality refer to $A_{(SLS/ULS)}$ and $Z_{t,(SLS/ULS)}$ respectively;										
SLS stress at top at design section utilisation						54%		OK		
TLS stress at top		$\sqrt{f_{ci}} / \sqrt{f_{ci}'}$	-1.25	≤	0.22	≤	0.50	f_{ci} / f_{ci}'		
at design section, f_t'			-6.3	≤	5.5	≤	12.5	N/mm ²		
$f_{min}' \leq \left[f_t' = \frac{k_P P'}{A} - \frac{k_P P' e}{Z_t} + \frac{M_{TLS,E/E}}{Z_t} + \frac{M_{TLS,S/E}}{Z_t} \right] \leq f_{max}'$				1.7 - -2.4 + -1.5 + 2.9				N/mm ²		
Note A and Z_t in the above inequality refer to A_{TLS} and $Z_{t,TLS}$ respectively;										
TLS stress at top at design section utilisation						44%		OK		
SLS stress at bottom		$\sqrt{f_{cu}} / \sqrt{f_c'}$	-0.45	≤	0.18	≤	0.40	f_{cu} / f_c'		
at design section, f_b			-2.7	≤	6.2	≤	14.0	N/mm ²		
$f_{min} \leq \left[f_b = \frac{K P_0}{A} + \frac{K P_0 e}{Z_b} - \frac{M_{SLS,E/E}}{Z_b} - \frac{M_{SLS,S/E}}{Z_b} \right] \leq f_{max}$				1.3 + -3.2 - -12.0 - 3.9				N/mm ²		
Note A and Z_b in the above inequality refer to $A_{(SLS/ULS)}$ and $Z_{b,(SLS/ULS)}$ respectively;										
SLS stress at bottom at design section utilisation						45%		OK		
TLS stress at bottom		$\sqrt{f_{ci}} / \sqrt{f_{ci}'}$	-1.25	≤	-1.02	≤	0.50	f_{ci} / f_{ci}'		
at design section, f_b'			-6.3	≤	-5.1	≤	12.5	N/mm ²		
$f_{min}' \leq \left[f_b' = \frac{k_P P'}{A} + \frac{k_P P' e}{Z_b} - \frac{M_{TLS,E/E}}{Z_b} - \frac{M_{TLS,S/E}}{Z_b} \right] \leq f_{max}'$				1.7 + -4.3 - -2.6 - 5.2				N/mm ²		
Note A and Z_b in the above inequality refer to A_{TLS} and $Z_{b,TLS}$ respectively;										
TLS stress at bottom at design section utilisation						82%		OK		
Note in the preceding equations, e refers to either e_{HOG} or e_{SAG} as relevant;										
Note by convention, positive stress is compressive and negative stress is tensile;										
Note by convention, e is positive downwards, measured from the centroid of the TLS/(SLS/ULS) section;										
Note by convention, a negative bending moment indicates hogging moment;										
TLS and SLS Average Precompression Rectangular or Flanged Beam						BS8110 ▼				
TLS average precompression, $k_P P$		0.9	≤	1.7	≤	6.0	N/mm ²			
SLS average precompression, $K P$		0.7	≤	1.3	≤	4.5	N/mm ²			
TLS and SLS minimum average precompression utilisation						54%		OK		
TLS and SLS maximum average precompression utilisation						29%		OK		
Slab										
Average precompression should be at least 0.7N/mm ² (cl.2.4.1 TR.43) to 0.9N/mm ² (cl.8.6.2.1 ACI318) to be										
Average precompression usually vary from 0.7N/mm ² to 2.5N/mm ² for solid slabs;						cl.1.3 TR.43				
Average precompression usually vary from 1.4N/mm ² to 2.5N/mm ² for slabs;						IStructE Exam Sc				
When the average precompression exceeds 2.0N/mm ² or the floor is very long, the effects						cl.3.3 TR.43				
of restraint to slab shortening by supports become important, otherwise they may be ignored;										
Beam										
Average prestress levels occasionally vary up to 6.0N/mm ² for ribbed or waffle slabs;						cl.1.3 TR.43				
Average precompression usually vary from 2.5N/mm ² to 4.5N/mm ² for beams;						IStructE Exam Sc				

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Member Design - PC Beam and Slab		Made by	XX	Date	24/10/2026	Chd.
				BS8110		
Magnet Diagram at Design Section Rectangular or Flanged Beam				BS8110 ▼		

Magnet Diagram (at Design Section)

10⁶/P₀ (kN)

Eccentricity, e (mm)

EQ1

elimit

EQ2

eHOG or eSAG

EQ3

10⁶/P₀

EQ4

Drawing Limits	Equation	e _{min,t,magnet}	e _{max,b,magnet}	
		Equation 1, EQ.1	-207	143
	Equation 2, EQ.2	-207	143	mm
	Equation 3, EQ.3	-79	59	mm
	Equation 4, EQ.4	-207	-79	mm

Note by convention, e is positive downwards, measured from the centroid of the (SLS/ULS) section;

	Sagging Moment	Invalid	Hogging Moment	Valid
Equation 1	$\frac{1}{P_0} \geq \frac{(1/A_{(SLS/ULS)} - e/Z_{t,(SLS/ULS)})}{(f_{max} - M_{max}/Z_{t,(SLS/ULS)})/K}$		$\frac{1}{P_0} \leq \frac{(1/A_{(SLS/ULS)} - e/Z_{t,(SLS/ULS)})}{(f_{min} - M_{max}/Z_{t,(SLS/ULS)})/K}$	
Equation 2	$\frac{1}{k_P P'} \leq \frac{(1/A_{TLS} - e/Z_{t,TLS})}{(f_{min}^I - M_{min}/Z_{t,TLS})}$		$\frac{1}{k_P P'} \geq \frac{(1/A_{TLS} - e/Z_{t,TLS})}{(f_{max}^I - M_{min}/Z_{t,TLS})}$	
Equation 3	$\frac{1}{P_0} \leq \frac{(1/A_{(SLS/ULS)} + e/Z_{b,(SLS/ULS)})}{(f_{min} + M_{max}/Z_{b,(SLS/ULS)})/K}$		$\frac{1}{P_0} \geq \frac{(1/A_{(SLS/ULS)} + e/Z_{b,(SLS/ULS)})}{(f_{max} + M_{max}/Z_{b,(SLS/ULS)})/K}$	
Equation 4	$\frac{1}{k_P P'} \geq \frac{(1/A_{TLS} + e/Z_{b,TLS})}{(f_{max}^I + M_{min}/Z_{b,TLS})}$		$\frac{1}{k_P P'} \leq \frac{(1/A_{TLS} + e/Z_{b,TLS})}{(f_{min}^I + M_{min}/Z_{b,TLS})}$	

Note that in the above inequalities, $M_{min} = M_{TLS,E/E} + M_{TLS,S/E}$ and $M_{max} = M_{SLS,E/E} + M_{SLS,S/E}$;
Note that $k_P P' = k_P [P_0 - (P_{L,DF} + P_{L,ES})]$; Note that $e_{TLS} = e_{(SLS/ULS)} + x_{c,(SLS/ULS)} - x_{c,TLS}$;

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.																																																														
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		Member/Location																																																																
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.																																																																
Member Design - PC Beam and Slab		Made by	XX	Date 24/10/2026																																																														
				Chd. BS8110																																																														
Allowable Tendon Profile (for Given Prestress Force at Transfer) at All Sections Rectangul				BS8110 ▼																																																														
Sagging Moment		Invalid	Hogging Moment	Valid																																																														
$e_{var} \geq \left[\frac{Z_t}{A} - \frac{f_{max} Z_t}{K P_0} \right] + \frac{M_{max,var}}{K P_0}$		$e_{var} \leq \left[\frac{Z_t}{A} - \frac{f_{min} Z_t}{K P_0} \right] + \frac{M_{max,var}}{K P_0}$		Note A is $A_{(SLS/ULS)}$ and Z_t is $Z_{t,(SLS/ULS)}$																																																														
$e_{var} \leq \left[\frac{Z_t}{A} - \frac{f_{min}^I Z_t}{k_p P'} \right] + \frac{M_{min,var}}{k_p P'}$		$e_{var} \geq \left[\frac{Z_t}{A} - \frac{f_{max}^I Z_t}{k_p P'} \right] + \frac{M_{min,var}}{k_p P'}$		Note A is A_{TLS} and Z_t is $Z_{t,TLS}$																																																														
$e_{var} \geq \left[-\frac{Z_b}{A} + \frac{f_{min} Z_b}{K P_0} \right] + \frac{M_{max,var}}{K P_0}$		$e_{var} \leq \left[-\frac{Z_b}{A} + \frac{f_{max} Z_b}{K P_0} \right] + \frac{M_{max,var}}{K P_0}$		Note A is $A_{(SLS/ULS)}$ and Z_b is $Z_{b,(SLS/ULS)}$																																																														
$e_{var} \leq \left[-\frac{Z_b}{A} + \frac{f_{max}^I Z_b}{k_p P'} \right] + \frac{M_{min,var}}{k_p P'}$		$e_{var} \geq \left[-\frac{Z_b}{A} + \frac{f_{min}^I Z_b}{k_p P'} \right] + \frac{M_{min,var}}{k_p P'}$		Note A is A_{TLS} and Z_b is $Z_{b,TLS}$																																																														
Note by convention, e_{var} is positive downwards, measured from the centroid of the (SLS/ULS) section;																																																																		
<table border="1"> <thead> <tr> <th colspan="8">Allowable Range of Eccentricity at All Sections</th> </tr> <tr> <th>Dist, x</th> <th>0.000</th> <th>0.500</th> <th>1.000</th> <th>1.889</th> <th>2.778</th> <th>3.667</th> <th>4.556</th> <th>m</th> </tr> </thead> <tbody> <tr> <td>$e_{min,var}$</td> <td>-248</td> <td>-214</td> <td>-183</td> <td>-65</td> <td>144</td> <td>283</td> <td>352</td> <td>mm</td> </tr> <tr> <td>$e_{max,var}$</td> <td>-62</td> <td>147</td> <td>334</td> <td>612</td> <td>750</td> <td>773</td> <td>784</td> <td>mm</td> </tr> <tr> <th>Dist, x</th> <th>5.444</th> <th>6.333</th> <th>7.222</th> <th>8.111</th> <th>9.000</th> <th>9.500</th> <th>10.000</th> <th>m</th> </tr> <tr> <td>$e_{min,var}$</td> <td>352</td> <td>283</td> <td>144</td> <td>-65</td> <td>-183</td> <td>-214</td> <td>-248</td> <td>mm</td> </tr> <tr> <td>$e_{max,var}$</td> <td>784</td> <td>773</td> <td>750</td> <td>612</td> <td>334</td> <td>147</td> <td>-62</td> <td>mm</td> </tr> </tbody> </table>					Allowable Range of Eccentricity at All Sections								Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m	$e_{min,var}$	-248	-214	-183	-65	144	283	352	mm	$e_{max,var}$	-62	147	334	612	750	773	784	mm	Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m	$e_{min,var}$	352	283	144	-65	-183	-214	-248	mm	$e_{max,var}$	784	773	750	612	334	147	-62	mm
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Note that in the above inequalities, $M_{min,var} = M_{TLS,E/E,var} + M_{TLS,S/E,var}$ and $M_{max,var} = M_{SLS,E/E,var} + M_{SLS,S/E,var}$;																																																																		
Note that $k_p P' = k_p [P_0 - (P_{L,DF} + P_{L,ES})]$; Note that $e_{var,TLS} = e_{var,(SLS/ULS)} + x_{c,(SLS/ULS)} - x_{c,TLS}$;																																																																		
Note that all 8 inequalities are simultaneously employed as hogging and sagging are interchangeable along the member in structural systems with certain support conditions (e.g. continuous);																																																																		
Allowable Tendon Profile (at All Sections) Legend: — $e_{min,var}$ — $e_{max,var}$ —◆— eHOG or eSAG (at design section) — evar (at all sections)																																																																		
Allowable range of eccentricity (for given P_0) at design section, e utilisation		-248	≤	-196	≤	-62	mm	OK																																																										
Allowable range of eccentricity (for given P_0) at all sections, e_{var} utilisation						79%		OK																																																										

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.														
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			Member/Location																
Job Title	Member Design - Prestressed Concrete Beam and Slab		Drg. Ref.																
Member Design - PC Beam and Slab			Made by	XX	Date 24/10/2026 Chd.														
			BS8110																
End Block Design Rectangular or Flanged Beam			BS8110 ▼																
<table border="1"> <tr> <td>y_{po}/y_o</td> <td>0.2</td> <td>0.3</td> <td>0.4</td> <td>0.5</td> <td>0.6</td> <td>0.7</td> </tr> <tr> <td>F_{bst}/P_o</td> <td>0.23</td> <td>0.23</td> <td>0.20</td> <td>0.17</td> <td>0.14</td> <td>0.11</td> </tr> </table>						y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7	F_{bst}/P_o	0.23	0.23	0.20	0.17	0.14	0.11
y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7													
F_{bst}/P_o	0.23	0.23	0.20	0.17	0.14	0.11													
End block width (rectangular) or web width (flanged), $b_{w,e}$			100%	500	mm														
Number of rows of anchorages, N_R				1															
Number of anchorages per row, N_A				1															
Vertical anchorage (group) (bottom) edge distance, A_{VED}				500	mm														
Vertical anchorage spacing, A_{VS}				1000	mm														
Horizontal anchorage edge distance, $(b_{w,e}/N_A)/2$			250	$\geq$	195 mm														
Horizontal anchorage spacing, $b_{w,e}/N_A$			500	$\geq$	370 mm														
Vertical anchorage edge distance, MIN $(A_{VED}, h-A_{VED}-N_A)$			500	$\geq$	195 mm														
Vertical anchorage spacing, A_{VS}			1000	$\geq$	370 mm														
Width / N/A of anchorage			Square ▼	100%	270 mm														
Depth / diameter of anchorage			Square ▼	100%	270 mm														
Total width of end block for each anchorage, $2y_{0,w}$					500 mm														
Total depth of end block for each anchorage, $2y_{0,d}$					1000 mm														
Total equivalent width of each anchorage, $2y_{p0,w}$					270 mm														
Total equivalent depth of each anchorage, $2y_{p0,d}$					270 mm														
Total width of end block for all anchorages, $\Sigma 2y_{0,w}$					500 mm														
Total depth of end block for all anchorages, $\Sigma 2y_{0,d}$					1000 mm														
Total equivalent width of all anchorages, $\Sigma 2y_{p0,w}$			1	$\times 2y_{p0,w}$	270 mm														
Total equivalent depth of all anchorages, $\Sigma 2y_{p0,d}$			1	$\times 2y_{p0,d}$	270 mm														
Maximum local compressive bearing stress, $[P_{0,free}/N_T]/[2y_{p0,w} \cdot 2y_{p0,d}]$				32.2	N/mm ²														
Maximum local compressive bearing stress utilisation, $[P_{0,free}/N_T]/[2y_{p0,w} \cdot 2y_{p0,d}]$				56%	OK														
Width ratios, $\{2y_{p0,w}/2y_{0,w}, \Sigma 2y_{p0,w}/\Sigma 2y_{0,w}\}$				0.54	0.54														
Depth ratios, $\{2y_{p0,d}/2y_{0,d}, \Sigma 2y_{p0,d}/\Sigma 2y_{0,d}\}$				0.27	0.27														
Jacking force at each anchorage, $P_{0,free}/N_T$				2346	kN														
Jacking force at all anchorages, $P_{0,free}$				2346	kN														
Bursting tensile force (width ratio), $F_{bst,w} = f(2y_{p0,w}/2y_{0,w}) \cdot (P_{0,free}/N_T)$				399	kN														
Bursting tensile force (depth ratio), $F_{bst,d} = f(2y_{p0,d}/2y_{0,d}) \cdot (P_{0,free}/N_T)$				540	kN														
Bursting tensile force (width ratio), $\Sigma F_{bst,w} = f(\Sigma 2y_{p0,w}/\Sigma 2y_{0,w}) \cdot P_{0,free}$				399	kN														
Bursting tensile force (depth ratio), $\Sigma F_{bst,d} = f(\Sigma 2y_{p0,d}/\Sigma 2y_{0,d}) \cdot P_{0,free}$				540	kN														
End block shear link diameter, $\phi_{link,e}$				16	mm														
End block number of links in a cross section, i.e. number of legs, $n_{leg,e}$				4															
End block area provided by closed links in a cross-section, $A_{sv,prov,e} = \pi \cdot \phi_{link,e}^2 / 4$				804	mm ²														
End block pitch of links, S_e				150	mm														
Allowable stress in end block shear links, $\sigma_e = 200 \text{ N/mm}^2$				200	N/mm ²														
Provide shear links $A_{sv,e}/S_e > [F_{bst,w}/(2y_{0,w}-0.2y_{0,w})]/\sigma_e$			4.43	mm ² /mm/	500 mm														
Provide shear links $A_{sv,e}/S_e > [F_{bst,d}/(2y_{0,d}-0.2y_{0,d})]/\sigma_e$			3.00	mm ² /mm/	1000 mm														
Provide shear links $A_{sv,e}/S_e > [\Sigma F_{bst,w}/(\Sigma 2y_{0,w}-\Sigma 0.2y_{0,w})]/\sigma_e$			4.43	mm ² /mm/	500 mm														
Provide shear links $A_{sv,e}/S_e > [\Sigma F_{bst,d}/(\Sigma 2y_{0,d}-\Sigma 0.2y_{0,d})]/\sigma_e$			3.00	mm ² /mm/	1000 mm														
Provide shear links $A_{sv,e}/S_e$ over distance of MAX $(2y_{0,w}, 2y_{0,d}, \Sigma 2y_{0,w}, \Sigma 2y_{0,d})$				1000	mm														
End block area provided by closed shear links in a cross-section, $A_{sv,prov,e}$				804	mm ²														
Tried $A_{sv,prov,e}/S_e$ value				5.36	mm ² /mm														
Design shear resistance at end block section utilisation				83%	OK														

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						Member/Location					
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Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026		Chd.
										BS8110	
Detailing Requirements Rectangular or Flanged Beam										BS8110	▼
All detailing requirements met ?								OK			
Cover to prestress tendon(s) ≥ MAX (MAX(D _{T,H} ,D _{T,V})/2, 25mm)								77	mm	OK	
Note cover to prestress tendon(s), MIN [h - x _{c,(SLS/ULS)} - e _{SAG} - D _{T,V} /2, x _{c,(SLS/ULS)} + e _{HOG} - D _{T,V} /2];						cl.4.12.3.1.4					
Min prestress tendon(s) clear spacing, S _T ≥ MAX (2D _{T,H} pre-T or D _{T,H} post-T, 5								N/A	mm	N/A	
Note S _T = (b _w - 2.cover - 2. ϕ _{link} - D _{T,H})/(N _T /n _{layers,PT} - 1) - D _{T,H} ;						cl.7.2 TR.43					
Max prestress tendon(s) clear spacing, S _T ≤ (8.h BD, 6.h un-BD, 1600mm)								N/A	mm	N/A	
Note S _T = (b _w - 2.cover - 2. ϕ _{link} - D _{T,H})/(N _T /n _{layers,PT} - 1) - D _{T,H} ;						cl.7.2 TR.43					
Min untensioned hogging steel reinforcement diameter, ϕ _t (≥6mm slab; ≥1								20	mm	OK	
Min untensioned hogging steel reinforcement pitch (>75mm+ϕ _t , >100mm+ϕ _t								95	mm	OK	
Note min untensioned hogging steel reinforcement pitch = (b _w - 2.cover - 2. ϕ _{link} - ϕ _t)/(n _t /n _{layers,tens} - 1);											
Max untensioned hogging steel reinforcement pitch (≤3.h, ≤500mm)								95	mm	OK	
Note max untensioned hogging steel reinforcement pitch (b _w - 2.cover - 2. ϕ _{link} - ϕ _t)/(n _t /n _{layers,tens} - 1);											
Min untensioned sagging steel reinforcement diameter, ϕ _t (≥6mm slab; ≥1								25	mm	OK	
Min untensioned sagging steel reinforcement pitch (>75mm+ϕ _t , >100mm+ϕ _t i								94	mm	OK	
Note min untensioned sagging steel reinforcement pitch = (b _w - 2.cover - 2. ϕ _{link} - ϕ _t)/(n _t /n _{layers,tens} - 1);											
Max untensioned sagging steel reinforcement pitch (≤3.h, ≤500mm)								94	mm	OK	
Note max untensioned sagging steel reinforcement pitch (b _w - 2.cover - 2. ϕ _{link} - ϕ _t)/(n _t /n _{layers,tens} - 1);											
% Min [SLS] untensioned reinforcement, A _{s,prov} /(b _w .h) @ Support Top								0.63	%	BS8110	▼
BD:- % Min [SLS] untensioned reinforcement (≥ 0.0000b _w .h)						Valid		0.00	%	6.10.6 TR.4	
Flat slab hogging:- % Min [SLS] untensioned reinforcement (≥ A ₁ /b _w .h)						Invalid				6.10.5 TR.4	
Un-BD:- % Min [SLS] untensioned reinforcement (≥ A ₁ /b _w .h)						Invalid				6.10.5 TR.4	
BS8110 beam, 1-way or 2-way slab class 3:- % Min [SLS] untensi						Valid		0.18	%	3.4.3 BS8	
ACI318 beam, 1-way or 2-way slab class T/C:- % Min [SLS] unten						Invalid				4.5.2.1 ACI	
ACI318 flat slab class U/T/C:- % Min [SLS] untensioned reinforcen						Invalid				6.2.3 ACI	
AS3600 class T/C:- % Min [SLS] untensioned reinforcement (≥ A ₁ /b _w .h)						Invalid				2, cl.9.4.2 A	
[SLS]	Tension zone, x = (-f _t . h) / (f _b - f _t) for support top							187	mm	6.10.5 TR.4	
	Tension zone, (h-x) = (-f _b . h) / (f _t - f _b) for span bottom							N/A	mm	6.10.5 TR.4	
	Tension force, F ₁ = -f _{t/b} . {x or (h-x)} . (b _w or b) / 2							255	kN	6.10.5 TR.4	
	Tension area, A ₁ = F ₁ / [function(f _y)]; ϕ _t = 20mm @ 288MPa							886	mm ²	6.10.5 TR.4	
Flat slab hogging:- % Min [SLS] untensioned reinforcement (≥ A ₁ /b _w .h)						Invalid		0.01	%		
Note tension area, A ₁ = 0.00075b _w .h x b _w /(2 x 1.5 x h + MIN(l _{h,h} ,l _{h,b}));						cl.6.10.6 TR.4					
Note concentrate rebar between 1.5 x slab thk either side of column width, extending ≥ 0.2L;						cl.6.10.6 TR.4					
Un-BD:- % Min [SLS] untensioned reinforcement (≥ A ₁ /b _w .h)						Invalid		0.18	%		
Note tension area, A ₁ = 0.0024-0.0032b _w .h G250; ≥ MAX (0.0013-0.0018, 0.0013-0.0018(f _{cu} /40						cl.3.1.7 TR.4					
% Min [SLS] untensioned reinforcement utilisation @ Support Top								28%		OK	
% Min [TLS] untensioned reinforcement, A _{s,prov} /(b _w .h) @ Support Bottom								0.49	%	BS8110	▼
BS8110 beam, 1-way or 2-way slab class 3:- % Min [TLS] untensi						Valid				3.5.2 BS8	
ACI318 beam, 1-way or 2-way slab class C:- % Min [TLS] untensi						Invalid		0.43	%	5.3.2.1 AC	
AS3600 class T/C:- % Min [TLS] untensioned reinforcement (≥ A ₁ /b _w .h)						Invalid				2, cl.9.4.2 A	
[TLS]	Tension zone, (h-x) = (-f _b . h) / (f _t - f _b) at support bottom							480	mm	6.10.5 TR.4	
	Tension zone, x = (-f _t . h) / (f _b - f _t) at span top							N/A	mm	6.10.5 TR.4	
	Tension force, F ₁ = -f _{b/t} . {(h-x) or x} . (b _w or b) / 2							614	kN	6.10.5 TR.4	
	Tension area, A ₁ = F ₁ / [function(f _y)]; ϕ _t = 25mm @ 288MPa							2136	mm ²	6.10.5 TR.4	
% Min [TLS] untensioned reinforcement utilisation @ Support Bottom								87%		OK	

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Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026
						Chd.			
						BS8110			
Deflection Criteria Rectangular or Flanged Beam						BS8110 ▼			
BS8110: The following deflection calculations assume an uncracked section for serviceability cl.4.3.6.2 BS8110									
classes 1 and 2 and (user defined %) cracked section for serviceability class 3;									
ACI318: The following deflection calculations assume an uncracked section for serviceability class U and (user defined %) cracked section for serviceability classes T and C; cl.24.2.3.8 ACI318									
cl.24.2.3.9 ACI318									
AS3600: The following deflection calculations assume an uncracked section for serviceability classes U and T and (user defined %) cracked section for serviceability class C; cl.8.5.2, cl.9.3.2 AS3600									
cl.6.2.5, cl.8.5.3.1									
All codes: If the flat slab (FTW-FS-DS) option is selected, the deflection calculations assume an uncracked section as the adopted stress limits ensure that a primarily uncracked behaviour is obtained;									
Note deflection, δ positive downwards; Note ω positive downwards;									
Elastic modulus, $E_{st} = E_{uncracked,28}$ or E_{ck}						0% Cracking		27.0 GPa	
Elastic modulus, $E_{lt} = E_{uncracked,28,cp}$ or $E_{ck,cp}$						0% Cracking		9.0 GPa	
Span, L						10.000 m			
TLS beam loading, $\omega_{TLS,E/E}$						35.0 kN/m			
DL+SDL beam loading, ω_{DL+SDL}						110.0 kN/m			
LL beam loading, ω_{LL}						50.0 kN/m			
SLS beam loading, $\omega_{SLS,E/E}$						160.0 kN/m			
						TLS SLS/ULS)			
Multiplier for rectangular or flanged $C_{1,1}$						Include if relevant ▼	0.8	0.8	Note Note Note Note
Multiplier for span more or less than 10m $C_{1,2}$						Include if relevant ▼	10/span		
Multiplier for flat slab $C_{1,3}$						Exclude	1.0		
Creep + live load deflection criteria						Brittle finishes L/500 ▼			
#3									
Onset of application of SDL and LL, %creep						Immediately with 0% creep ▼			cl.7.3
Creep modulus factor, C_{MF}						Storage loading, $CMF=1/[1+f=2.0]$ ▼			BS8110-2
Dead load, $DL = DL_h + DL_v + DL_b/t_w + DL_{point,h}/L/t_w + DL_{point,v}/L/t_w$						7.0 kPa			
Superimposed dead load, $SDL = SDL_h + SDL_v$						15.0 kPa			
Live load, $LL = LL_h + LL_v$						10.0 kPa			
Creep factor, $k_c = [(1-C_{MF}).(1-\%creep).DL+SDL] / [DL+SDL]$						0.89			
Note conservatively, creep factor, k_c calculated by assuming that both the elastic and creep components of the deflection due to the SDL contribute to the in-service deflection check, contrary to that which is assumed by MOSLEY, where only the creep component of the deflection due to the SDL is considered;									
Creep factor, $k_{c,PT} = (1-\%creep)$						1.00			
#3									
Inclusion of $\Sigma\delta_{limit,max}$						Include ▼		Note	
#9									
Detailing Requirements Rectangular or Flanged Beam (Continued)									
% Min tensioned and untensioned reinf., $(N_T.N_s.A_s + A_{s,prov})/(b_w.h)$						0.96 %			
% Min tensioned and untensioned reinf. ($\geq 0.0024-0.0032b_w h$ G250; $\geq \text{MAX}(0.0013-0.0018, \phi 1.3.1.7 \text{ TR.4})$)									
% Min tensioned and untensioned reinf. utilisation						19%		OK	
% Max tensioned and untensioned reinf., $(N_T.N_s.A_s + A_{s,prov})/(b_w.h)$						0.96 %			
% Max tensioned and untensioned reinf. ($\leq 0.04b_w h$)									
% Max tensioned and untensioned reinf. utilisation						24%		OK	
#3									
Min shear link diameter, $\phi_{link} (\geq 6\text{mm})$						10 mm			
Shear link pitch, S						100 mm			
Note require S ($\leq 0.75d_{max}$ ($\leq 0.50d_{max}$ if $V_d > 1.8 \phi V_c$), $\leq 4b_w$, $\leq 300\text{mm}$, $\geq \text{MAX}(100\text{mm}, 50+12.5n_{leg})$)									
$A_{sv,prov} / (b_w.S) (\geq 0.10\% \text{ G460; } \geq 0.17\% \text{ G250})$						0.63 %			
Note require an overall enclosing link; Note require additional restraining links for each alternate longitudinal bar									
Note lacer bars of 16mm are required at the sides of beams more than 750mm deep at 250mm pitch;									

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Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026
						Chd.			
						BS8110			
						BS8110 ▼			
Simply Supported						N/A			
Continuous (Infinitely, Encastre)						VALID			
Cantilever						N/A			
S3600									
AS3600									
						Duration	Short	Long	Long
						Term	Term	Term	Term
						Limit	TLS	SLS	In-
						State			Service
						E =	E _{st}	E _{lt}	E _{lt}
Elastic and Creep Deflections						$\omega =$	$\omega_{TLS,E/E}$	$\omega_{SLS,E/E}$	$k_C \cdot (\omega_{DL+S_{DL}}) + \omega_{LL}$
S/S.	$\delta_{EL} = 5\omega L^4 / (384EI_{TLS/(SLS/ULS)}) + f(DL_{point,h/v})$					N/A	N/A	N/A	mm
Cont.	$\delta_{EL} = \omega L^4 / (384EI_{TLS/(SLS/ULS)}) + f(DL_{pc})$					100%	0.5	6.5	6.0 mm
Cant.	$\delta_{EL} = \omega L^4 / (8EI_{TLS/(SLS/ULS)}) + f(DL_{point,h/v})$					N/A	N/A	N/A	mm
Prestress Deflection (Due to Drapes)						$\omega_{E/L} =$	$\omega_{TLS,E/L}$	$\omega_{SLS,E/L}$	$-(\omega_{SLS,E/L} - \omega_{E/L})$
Note for simplicity, the prestress deflection (due to drapes) calculation excludes the support peak tendon reverse curvature;									
S/S.	Total drapes, $e_d = \text{MAX}(e_{SAG}, e_{var}) - (e_{LHS} + e_R)$					N/A	N/A		mm
S/S.	TLS equivalent load, $\omega_{TLS,E/L} = -8P'e_d/L^2$					N/A			kN/m
S/S.	SLS equivalent load, $\omega_{SLS,E/L} = -8KP_0 \cdot e_d/L^2$						N/A		kN/m
Cont.	Total drapes, $e_d = \text{MAX}(e_{SAG}, e_{var}) - (e_{LHS} + e_R)$					720	720		mm
Cont.	TLS equivalent load, $\omega_{TLS,E/L} = -8P'e_d/L^2$					-121.9			kN/m
Cont.	SLS equivalent load, $\omega_{SLS,E/L} = -8KP_0 \cdot e_d/L^2$						-104.8		kN/m
Cant.	Total drapes, $e_d = \text{MAX}(e_{SAG}, e_{var}) - e_{LHS}$					N/A	N/A		mm
Cant.	TLS equivalent load, $\omega_{TLS,E/L} = -2P'e_d/L^2$					N/A			kN/m
Cant.	SLS equivalent load, $\omega_{SLS,E/L} = -2KP_0 \cdot e_d/L^2$						N/A		kN/m
S/S.	$\delta_{PT,D} = 5\omega_{E/L} L^4 / (384EI_{TLS/(SLS/ULS)})$					N/A	N/A	N/A	mm
Cont.	$\delta_{PT,D} = \omega_{E/L} L^4 / (384EI_{TLS/(SLS/ULS)})$					100%	-1.6	-4.3	-2.6 mm
Cant.	$\delta_{PT,D} = \omega_{E/L} L^4 / (8EI_{TLS/(SLS/ULS)})$					N/A	N/A	N/A	mm
Prestress Deflection (Due to End Eccentricity)						P'	KP_0	$k_C \cdot (KP_0 - P')$	
S/S.	$\delta_{PT,E} = -[P' \text{ or } KP_0] \cdot (e_{LHS} + e_{RHS}) / 2 \cdot L^2 / (8EI_{TLS/(SLS/ULS)})$					N/A	N/A	N/A	mm
Cont.	$\delta_{PT,E} = -[P' \text{ or } KP_0] \cdot (e_{LHS} + e_{RHS}) / 2 \cdot L^2 / (8EI_{TLS/(SLS/ULS)})$					100%	0.0	0.0	0.0 mm
Tendon termination at x=0 ?								Continues ▼	
Tendon termination at x=L ?								Continues ▼	
Cant.	$\delta_{PT,E} = [P' \text{ or } KP_0] \cdot e_{RHS} \cdot L^2 / (2EI_{TLS/(SLS/ULS)})$					N/A	N/A	N/A	mm
Note for simplicity, E/L effects due to any change of section not computed;						Note			
Total Deflection									
$\Sigma \delta = \delta_{EL} + \delta_{PT,D} + \delta_{PT,E}$						-1.2	2.2	3.4	mm
						Max	Max	Max	Max
						Defl'n	Upward	Downward	Creep+LL
$\Sigma \delta_{limit}$						-L/350	L/250	L/500	$C_{1,1} \cdot C_{1,2} \cdot C_{1,3}$
$\Sigma \delta_{limit,max}$						-20.0	-	20.0	mm
$\Sigma \delta_{limit}$						-20.0	32.0	16.0	mm
$\Sigma \delta / \Sigma \delta_{limit}$						6%	7%	21%	OK

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				Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab			Drg. Ref.			
Member Design - PC Beam and Slab				Made by	XX	Date 24/10/2026	
				Chd.			
				BS8110			
Bending at Design Section Rectangular or Flanged Beam (Tensioned Reinforcement)						Note	
ULS bending moment at design section, $M_{ULS} = M_{ULS,E/E} + M_{SLS,S/E}$				-1521	kNm		
Note by convention, a negative bending moment indicates hogging moment;							
Ultimate moment of resistance (steel), $M_{u,s} = F_{t,s,t} \cdot z_t = F_{t,s,t} \cdot (d_{ps} - 0.45x)$				1803	kNm	OK	
Ultimate moment of resistance (concrete), $M_{u,c} = F_{c,c} \cdot z_c = F_{c,c} \cdot (d_{ps} - 0.45x)$				1803	kNm	OK	
First Principles Approach (Bonded Tendons Only)	Eff. depth to tensioned reinf., d_{ps}		840		mm		
	Sagging Moment	Invalid	Hogging Moment	Valid			
	$d_{ps} = x_{c,(SLS/ULS)} + e_{SAG}$		$d_{ps} = h - x_{c,(SLS/ULS)} - e_{HOG}$				
	Trial depth of neutral axis, x (usually $0.5d_{ps}$, $0.4d_{ps}$ or $0.33d_{ps}$)				380	mm	Goal Seek
	Ratio, x/d_{ps}				0.45		OK
	Check compression block within flange, $0.9x \leq h_f$?				N/A		
	Total tensioned steel tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t} + \epsilon_{b,s,t}$				0.0101		
	Prestress strain, $\epsilon_{p,s,t} = [KP_0 / (N_T \cdot N_s \cdot A_s)] / E_p$				0.0058		
	Bending strain, $\epsilon_{b,s,t} = [(d_{ps} - x) / x] \cdot \epsilon_{cu}$				0.0042		
	Total tensioned steel tensile stress, $\sigma_{t,s,t}$				1605	N/mm ²	
	Ratio, $\sigma_{t,s,t} / 0.95f_{pk}$				0.91		
	Tensioned steel yielded ?				Partially Yielded		
	Figure 2.3 BS8110-1 • $\epsilon_{t,s,t} \leq 0.005$ [Not Yielded] $\Rightarrow \sigma_{t,s,t} = \epsilon_{t,s,t} E_p$ • $0.005 < \epsilon_{t,s,t} < 0.005 + \frac{f_{pk} / \gamma_m}{E_p}$ [Partially Yielded] $\Rightarrow \sigma_{t,s,t} = 0.8f_{pk} / \gamma_m + \left(\frac{f_{pk} / \gamma_m - 0.8f_{pk} / \gamma_m}{0.005 + \frac{f_{pk} / \gamma_m}{E_p} - 0.005} \right) (\epsilon_{t,s,t} - 0.005)$ • $\epsilon_{t,s,t} \geq 0.005 + \frac{f_{pk} / \gamma_m}{E_p}$ [Fully Yielded] $\Rightarrow \sigma_{t,s,t} = f_{pk} / \gamma_m$						
	Total tensioned steel tensile force, $F_{t,s,t} = \sigma_{t,s,t} \cdot N_T \cdot N_s \cdot A_s$				2697	kN	OK
	Total concrete compressive force, $F_{c,c}$				2697	kN	OK
Note $F_{c,c} = 0.45f_{cu} \cdot b_w \cdot (0.9x)$ for rect- section or T- or L- sections (with hogging)							
Note $F_{c,c} = \{0.45f_{cu} \cdot b \cdot (0.9x) \text{ if } 0.9x \leq h_f \text{ or } 0.45f_{cu} \cdot (b - b_w) \cdot h_f + 0.45f_{cu} \cdot b_w \cdot 0.9x \text{ if } 0.9x > h_f\}$ for T-							
Ultimate moment of resistance at design section, $\phi M_u = \pm \phi \text{AVERAGE}(M_{u,s}, M_{u,c})$				-1803	kNm		
Ultimate moment of resistance at design section utilisation				Converged	84%	OK	
Ultimate moment of resistance at design section, ϕM_u				-1544	kNm		
Codified Approach (Bonded and Unbonded Tendons)	$M_u = f_{pb} A_{ps} (d_{ps} - 0.45x)$ [Rectangular] or [Flanged - NA in Flange]				cl.4.3.7.3		
	$M_u = f_{pb} (A_{ps} - A_{pf}) (d_{ps} - 0.45x) + 0.45f_{cu} (b - b_w) h_f (d_{ps} - 0.45h_f)$ [Flanged - NA in Web]				cl.7.3.2 Krishna		
	Area of prestress tendon(s), $A_{ps} = N_T \cdot N_s \cdot A_s$				1680	mm ²	
	Equiv. area of prestress for flange, $A_{pf} = 0.45f_{cu} \cdot (b - b_w) \cdot (h_f / f_{pk})$				N/A	mm ²	
	Ratio, $[f_{pu} A_{ps}] / [f_{cu} b d]$				0.21		
	Note $[f_{pu} A_{ps}] / [f_{cu} b d] = [f_{pk} A_{ps}] / [f_{cu} b_w d_{ps}]$ [Rectangular];				cl.4.3.7.3		
	Note $[f_{pu} A_{ps}] / [f_{cu} b d] = [f_{pk} A_{ps}] / [f_{cu} b d_{ps}]$ [Flanged - NA in Flange];				cl.7.3.2 Krishna		
	Note $[f_{pu} A_{ps}] / [f_{cu} b d] = [f_{pk} (A_{ps} - A_{pf})] / [f_{cu} b_w d_{ps}]$ [Flanged - NA in Web];				cl.7.3.2 Krishna		
	Ratio, $f_{pe} / f_{pu} = KP_0 / [N_T \cdot N_s \cdot A_s] / f_{pk} \leq 0.60$				0.58	T.4.4	
					BD	Un-BD	
Ratio, $f_{pb} / 0.95f_{pu} = f_{pb} / 0.95f_{pk}$				0.79	N/A		
Ratio, $x/d = x/d_{ps}$				0.48	N/A		
				OK			
Un-	$f_{pb} = f_{pe} + \frac{7000}{L/d_{ps}} \cdot \left(1 - 1.7 \frac{f_{pu} A_{ps}}{f_{cu} b d} \right) \leq 0.7f_{pu} = 0.7f_{pk}$, $x = 2.47 \left[\frac{f_{pu} A_{ps}}{f_{cu} b d} \frac{f_{pb}}{f_{pu}} d \right] = 2.47 \left[\frac{f_{pu} A_{ps}}{f_{cu} b d} \frac{f_{pb}}{f_{pk}} d_{ps} \right]$						
Ultimate moment of resistance at design section utilisation, $M_{ULS} / \phi M_u$				98%		OK	

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				jXXX	33		
				Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab			Drg. Ref.			
Member Design - PC Beam and Slab				Made by	XX	Date 24/10/2026	
				Chd. BS8110			
Bending at Design Section Rectangular or Flanged Beam (Tensioned and Untensioned Reinf)							
ULS bending moment at design section, $M_{ULS} = M_{ULS,E/E} + M_{SLS,S/E}$				-1521	kNm		
Note by convention, a negative bending moment indicates hogging moment;							
Ultimate moment of resistance (steel), $M_{u,s}$				2424	kNm	OK	
Note $M_{u,s} = F_{t,s,t} \cdot z_t + F_{t,s,u} \cdot z_u = F_{t,s,t} \cdot (d_{ps} - 0.45x) + F_{t,s,u} \cdot (d_{rb} - 0.45x)$;							
First Principles Approach (Bonded Tendons Only)	Eff. depth to tensioned reinf., d_{ps}		840		mm		
	Sagging Moment	Invalid	Hogging Moment	Valid			
	$d_{ps} = x_{c,(SLS/ULS)} + e_{SAG}$		$d_{ps} = h - x_{c,(SLS/ULS)} - e_{HOG}$				
	Trial depth of neutral axis, x (usually $0.5d_{ps}$, $0.4d_{ps}$ or $0.33d_{ps}$)				553	mm	Goal Seek
	Ratio, x/d_{cen}				0.64		NOT OK
	Check compression block within flange, $0.9x \leq h_f$?				N/A		
	Total tensioned steel tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t} + \epsilon_{b,s,t}$				0.0076		
	Prestress strain, $\epsilon_{p,s,t} = [KP_0 / (N_T \cdot N_s \cdot A_s)] / E_p$				0.0058		
	Bending strain, $\epsilon_{b,s,t} = [(d_{ps} - x) / x] \cdot \epsilon_{cu}$				0.0018		
	Total tensioned steel tensile stress, $\sigma_{t,s,t}$				1515	N/mm ²	
	Ratio, $\sigma_{t,s,t} / 0.95f_{pk}$				0.86		
	Tensioned steel yielded ?				Partially Yielded		
	Figure 2.3 BS8110-1 $\epsilon_{t,s,t} \leq 0.005$ [Not Yielded] $\Rightarrow \sigma_{t,s,t} = \epsilon_{t,s,t} E_p$ $0.005 < \epsilon_{t,s,t} < 0.005 + \frac{f_{pk}}{E_p}$ [Partially Yielded] $\Rightarrow \sigma_{t,s,t} = 0.8f_{pk} / \gamma_m + \left(\frac{f_{pk}}{E_p} - 0.8f_{pk} / \gamma_m \right) (\epsilon_{t,s,t} - 0.005)$ $\epsilon_{t,s,t} \geq 0.005 + \frac{f_{pk}}{E_p}$ [Fully Yielded] $\Rightarrow \sigma_{t,s,t} = f_{pk} / \gamma_m$						
	Total tensioned steel tensile force, $F_{t,s,t} = \sigma_{t,s,t} \cdot N_T \cdot N_s \cdot A_s$				2546	kN	OK
	Total concrete compressive force, $F_{c,c}$				3922	kN	OK
Note $F_{c,c} = 0.45f_{cu} \cdot b_w \cdot (0.9x)$ for rect- section or T- or L- sections (with hogging);							
Note $F_{c,c} = \{0.45f_{cu} \cdot b \cdot (0.9x) \text{ if } 0.9x \leq h_f \text{ or } 0.45f_{cu} \cdot (b - b_w) \cdot h_f + 0.45f_{cu} \cdot b_w \cdot 0.9x \text{ if } 0.9x > h_f\}$ for T-							
Ultimate moment of resistance at design section, $\phi M_u = \pm \phi M_{u,s}$				-971	-2424	kNm	
Ultimate moment of resistance at design section utilisation				Converged	63%		
Ultimate moment of resistance at design section, ϕM_u				-971	-1953	kNm	
Codified Approach (Bonded and Unbonded Tendons)	$M_u = f_{pb} A_{ps} (d_{cen} - 0.45x)$ [Rectangular] or [Flanged - NA in Flange]				cl.4.3.7.3		
	$M_u = f_{pb} (A_{ps} - A_{pf}) (d_{cen} - 0.45x) + 0.45f_{cu} (b - b_w) h_f (d_{cen} - 0.45h_f)$ [Flanged - NA in Web]				cl.7.3.2 Krishna		
	Equiv. area of prestress tendon(s), $A_{ps} = N_T \cdot N_s \cdot A_s + A_{s,prov} \cdot f_y / f_{pk}$				2457	mm ²	cl.4.3.7.4
	Equiv. area of prestress for flange, $A_{pf} = 0.45f_{cu} \cdot (b - b_w) \cdot (h_f / f_{pk})$				N/A	mm ²	cl.7.3.2 Krishna
	Ratio, $[f_{pu} A_{ps}] / [f_{cu} b d]$				0.30		
	Note $[f_{pu} A_{ps}] / [f_{cu} b d] = [f_{pk} A_{ps}] / [f_{cu} b_w d_{cen}]$ [Rectangular];				cl.4.3.7.3		
	Note $[f_{pu} A_{ps}] / [f_{cu} b d] = [f_{pk} A_{ps}] / [f_{cu} b d_{cen}]$ [Flanged - NA in Flange];				cl.7.3.2 Krishna		
	Note $[f_{pu} A_{ps}] / [f_{cu} b d] = [f_{pk} (A_{ps} - A_{pf})] / [f_{cu} b_w d_{cen}]$ [Flanged - NA in Web];				cl.7.3.2 Krishna		
	Ratio, $f_{pe} / f_{pu} = KP_0 / [N_T \cdot N_s \cdot A_s + A_{s,prov} \cdot f_y / f_{pk}] / f_{pk} \leq 0.60$				0.40		T.4.4
					BD	Un-BD	
Ratio, $f_{pb} / 0.95f_{pu} = f_{pb} / 0.95f_{pk}$				0.70	N/A	T.4.4	
Ratio, $x/d = x/d_{cen}$				0.57	N/A	NOT OK	
Un-	$f_{pb} = f_{pe} + \frac{7000}{L/d_{cen}} \cdot \left(1 - 1.7 \frac{f_{pu} A_{ps}}{f_{cu} b d} \right) \leq 0.7f_{pu} = 0.7f_{pk}$, $x = 2.47 \left[\frac{f_{pu} A_{ps} f_{pb}}{f_{cu} b d f_{pu}} d \right] = 2.47 \left[\frac{f_{pu} A_{ps} f_{pb}}{f_{cu} b d f_{pk}} d_{cen} \right]$						
Ultimate moment of resistance at design section utilisation, $M_{ULS} / \phi M_u$				78%		OK	

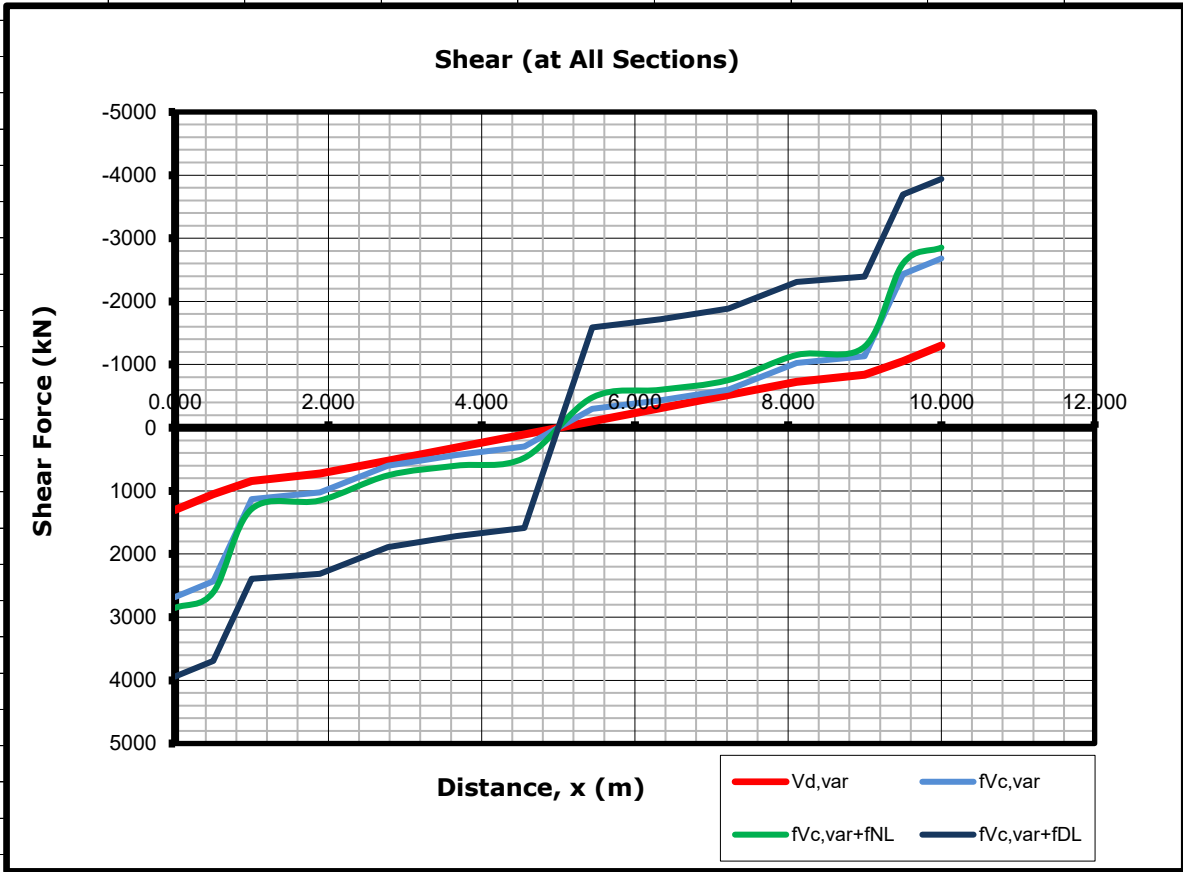
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						jXXX	35			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
										BS8110
Bending at All Sections Rectangular or Flanged Beam (Tensioned and Untensioned Reinfor										Note
M _{ULS,var} and e _{var} at All Sections										First Principles Approach (Bonded Tendons Only)
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m		
M _{ULS,var}	-1521	-965	-468	271	826	1196	1381	kNm		
e _{var}	-196	-160	-52	175	346	460	517	mm		
d _{ps,var}	840	804	696	532	702	816	873	mm		
x _{var}	553	552	546	150	150	150	150	mm	Goal Seek	
.9x _{var} ≤h _f ?	N/A	N/A	N/A	Yes	Yes	Yes	Yes			
ε _{p,s,t}	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058			
ε _{b,s,t,var}	0.0018	0.0016	0.0010	0.0089	0.0128	0.0155	0.0168			
ε _{t,s,t,var}	0.0076	0.0074	0.0068	0.0147	0.0187	0.0213	0.0226			
Tendon Yielded?	Partially Yielded	Partially Yielded	Partially Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded			
σ _{t,s,t,var}	1515	1507	1484	1771	1771	1771	1771	N/mm ²		
F _{t,s,t,var}	2546	2532	2492	2976	2976	2976	2976	kN		
F _{c,c,var}	3922	3909	3869	4051	4051	4051	4051	kN		
d _{rb}	917	917	917	937	937	937	937	mm		
ε _{b,s,u,var}	0.0023	0.0023	0.0024	0.0183	0.0183	0.0183	0.0183			
ε _{t,s,u,var}	0.0023	0.0023	0.0024	0.0183	0.0183	0.0183	0.0183			
Rebar Yielded?	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded			
σ _{t,s,u,var}	438	438	438	438	438	438	438	N/mm ²		
F _{t,s,u,var}	1376	1376	1376	1075	1075	1075	1075	kN		
φM _{u,var}	-2424	-2328	-2047	2315	2823	3162	3331	kNm		
Converge'n	Yes	Yes	Yes	Yes	Yes	Yes	Yes			
UT	63%	41%	23%	12%	29%	38%	41%	%		
Status	OK	OK	OK	OK	OK	OK	OK			
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m		
M _{ULS,var}	1381	1196	826	271	-468	-965	-1521	kNm		
e _{var}	517	460	346	175	-52	-160	-196	mm		
d _{ps,var}	873	816	702	532	696	804	840	mm		
x _{var}	150	150	150	150	546	552	553	mm	Goal Seek	
.9x _{var} ≤h _f ?	Yes	Yes	Yes	Yes	N/A	N/A	N/A			
ε _{p,s,t}	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058			
ε _{b,s,t,var}	0.0168	0.0155	0.0128	0.0089	0.0010	0.0016	0.0018			
ε _{t,s,t,var}	0.0226	0.0213	0.0187	0.0147	0.0068	0.0074	0.0076			
Tendon Yielded?	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Partially Yielded	Partially Yielded	Partially Yielded			
σ _{t,s,t,var}	1771	1771	1771	1771	1484	1507	1515	N/mm ²		
F _{t,s,t,var}	2976	2976	2976	2976	2492	2532	2546	kN		
F _{c,c,var}	4051	4051	4051	4051	3869	3909	3922	kN		
d _{rb}	937	937	937	937	917	917	917	mm		
ε _{b,s,u,var}	0.0183	0.0183	0.0183	0.0183	0.0024	0.0023	0.0023			
ε _{t,s,u,var}	0.0183	0.0183	0.0183	0.0183	0.0024	0.0023	0.0023			
Rebar Yielded?	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded	Fully Yielded			
σ _{t,s,u,var}	438	438	438	438	438	438	438	N/mm ²		
F _{t,s,u,var}	1075	1075	1075	1075	1376	1376	1376	kN		
φM _{u,var}	3331	3162	2823	2315	-2047	-2328	-2424	kNm		
Converge'n	Yes	Yes	Yes	Yes	Yes	Yes	Yes			
UT	41%	38%	29%	12%	23%	41%	63%	%		
Status	OK	OK	OK	OK	OK	OK	OK			
Note by convention, a negative bending moment indicates hogging moment; Note above M _{ULS,var} = M _{ULS,E/E,var}										
Ultimate moment of resistance utilisation, MAX (M _{ULS,var} /M _{u,var})							63%		OK	
Convergence of moment of resistance equations						Converged				

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				Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab			Drg. Ref.		
Member Design - PC Beam and Slab				Made by	XX	Date 24/10/2026 Chd.
				BS8110 [PC]		
Shear at Critical and (Shear) Design Section Rectangular Beam				Note		
Shear at Critical and (Shear) Design Section Add. Code Options [Appl. When BS8110 Chosen]						
BS8110 and TR.43-1 [PC] BS8110 [RC] EC2 and TR.43-2 [PC]				BS8110 TR.43-1 [PC]	▼	Note
Shear at Critical Design Section Rectangular Beam						
ULS shear force at critical section, $V_{ult} = ABS (V_{ULS,E/E} + V_{SLS,S/E})$				1301	kN	
Ult. shear stress at crit. sect., $v_{ult} = V_{ult}/b_v d_{cen,ult} (< 0.8f_{cu}^{0.5} \& \{5.0, 7.0\}N/mm^2)$				3.01	N/mm ²	3.4.5.2, cl.4.3.8.1
Ult. shear strength at crit. sect., $MIN\{0.8f_{cu}^{0.5} \& \{5.0, 7.0\}N/mm^2\}$				4.73	N/mm ²	3.4.5.2, cl.4.3.8.1
Breadth, $b_v = b_w - (2/3 BD, 1 \text{ un-BD}).N_T.MAX(D_{T,H}, D_{T,V})$				500	mm	cl.4.3.8.1
Exclude duct ▼						
ULS bending moment at critical section, $M_{ult} = M_{ULS,E/E,var}(x=0) + M_{SLS,S/E,var}(x=0)$				-1521	kNm	
Note by convention, a negative bending moment indicates hogging moment;						
Eff. depth to A_s at critical section, $d_{ps,ult}$				840	mm	N/A
Sagging Moment		Invalid		Hogging Moment		Valid
$d_{ps,ult} = x_{c,(SLS/ULS)} + e_{var}(x=0)$				$d_{ps,ult} = h - x_{c,(SLS/ULS)} - e_{var}(x=0)$		
Note $d_{ps,ult}$ calculated based on actual section, rectangular or flanged, $x_{c,(SLS/ULS)}$ property;						
Eff. depth to $A_{s,prov}$, d_{rb}				917	mm	N/A
Sagging Moment		Invalid		Hogging Moment		Valid
Sag $d_{rb} = h - cover - \phi_{link} - [\phi_t + (n_{layers,tens} - 1)(\phi_t + s_{r,tens})]/2$ [exterior untensioned reinforcement];						
Hog $d_{rb} = h - cover - MAX(\phi_{link}, cover_{add}) - [\phi_t + (n_{layers,tens} - 1)(\phi_t + s_{r,tens})]/2$ [exterior untensioned reinforcement];						
Eff. depth to centroid of A_s and $A_{s,prov}$ at critical section, $d_{cen,ult}$				864	mm	cl.4.3.8.1
Note $d_{cen,ult} = [N_T.N_s.A_s.d_{ps,ult} + A_{s,prov}.f_y/f_{pk}.d_{rb}]/[N_T.N_s.A_s + A_{s,prov}.f_y/f_{pk}]$; Note $d_{cen,ult} > 0.8h$ in ACI318;						
Eff. depth to max of A_s and $A_{s,prov}$ at critical section, $d_{max,ult}$				917	mm	3.8.1, cl.4.3.8.1
Note $d_{max,ult} = MAX(d_{ps,ult}, d_{rb})$; Note $d_{max,ult} > 0.8h$ in AS3600;						
Ultimate shear stress at critical section utilisation				64%		OK
Shear at (Shear) Design Section Rectangular Beam						
(Shear) design section distance, x_d				0%L	▼	0.000 m
Note that the (shear) design section location differs to that of the (bending) design section location;						
ULS shear force at (shear) design section, V_d				1301	kN	
Note $V_d = ABS (V_{ULS,E/E,var}(x=x_d) + V_{SLS,S/E,var}(x=x_d))$;						
Note no sign convention applicable as ABS function applied;						
ULS bending moment at (shear) design section, M_d				-1521	kNm	
Note $M_d = M_{ULS,E/E,var}(x=x_d) + M_{SLS,S/E,var}(x=x_d)$;						
Note by convention, a negative bending moment indicates hogging moment;						
Eff. depth to A_s at (shear) design section, $d_{ps,d}$				840	mm	N/A
Sagging Moment		Invalid		Hogging Moment		Valid
$d_{ps,d} = x_{c,(SLS/ULS)} + e_{var}(x=x_d)$				$d_{ps,d} = h - x_{c,(SLS/ULS)} - e_{var}(x=x_d)$		
Note $d_{ps,d}$ calculated based on actual section, rectangular or flanged, $x_{c,(SLS/ULS)}$ property;						
Eff. depth to $A_{s,prov}$, d_{rb}				917	mm	N/A
Sagging Moment		Invalid		Hogging Moment		Valid
Sag $d_{rb} = h - cover - \phi_{link} - [\phi_t + (n_{layers,tens} - 1)(\phi_t + s_{r,tens})]/2$ [exterior untensioned reinforcement];						
Hog $d_{rb} = h - cover - MAX(\phi_{link}, cover_{add}) - [\phi_t + (n_{layers,tens} - 1)(\phi_t + s_{r,tens})]/2$ [exterior untensioned reinforcement];						
Eff. depth to centroid of A_s and $A_{s,prov}$ at (shear) design section, $d_{cen,d}$				864	mm	cl.4.3.8.1
Note $d_{cen,d} = [N_T.N_s.A_s.d_{ps,d} + A_{s,prov}.f_y/f_{pk}.d_{rb}]/[N_T.N_s.A_s + A_{s,prov}.f_y/f_{pk}]$; Note $d_{cen,d} > 0.8h$ in ACI318;						
Eff. depth to max of A_s and $A_{s,prov}$ at (shear) design section, $d_{max,d}$				917	mm	3.8.1, cl.4.3.8.1
Note $d_{max,d} = MAX(d_{ps,d}, d_{rb})$; Note $d_{max,d} > 0.8h$ in AS3600;						
Design shear stress at (shear) design section, $v_d = V_d/b_v d_{cen,d}$				3.01	N/mm ²	cl.4.3.8.1

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Member Design - PC Beam and Slab				Made by	XX	Date	24/10/2026
				Chd.			
				BS8110 [PC			
				BS8110 [F ▼			
Uncracked design shear resistance, V_{co}				757	kN	cl.4.3.8.4	
$V_{co} = 0.67b_v h \sqrt{f_t^2 + 0.8f_{cp}f_t} + V_p$							
Uncracked design shear strength, $V_{co}/b_v h$				1.51	N/mm ²		
Vertical component of prestress force, $V_p = \gamma_p \cdot KP_0 \sin \beta$				131	kN	cl.4.3.8.4	
Maximum design principal tensile stress, $f_t = 0.24 \sqrt{f_{cu}}$, $f_{cu} \leq 80 \text{ N/mm}^2$				1.42	N/mm ²	cl.4.3.8.4	
Comp. stress at centroid, $f_{cp/pc}[\sigma_{cp}] = KP_0/A_{(SLS/ULS)}$				1.3	N/mm ²		
Note $f_{cp/pc}[\sigma_{cp}]$ calculated based on actual section, rectangular or flanged, $A_{(SLS/ULS)}$ property;							
4.3.8.1 Note for pre-tensioned members, where the design section occurs within the prestressed							
4.3.8.1 development length, the compressive stress at the centroidal axis due to prestress, $f_{cp/pc}[\sigma_{cp}]$							
should be calculated based on cl.4.3.8.4 BS8110 [cl.6.2.2(2) EC2] and cl.22.5.9 ACI318;							
Cracked design shear resistance, V_{cr}				256	670	kN	cl.4.3.8.5
Cracked design shear strength, $V_{cr}/b_v d_{cen,d}$				1.55	N/mm ²	cl.4.3.8.1	
$V_{cr} = (1 - 0.55 \frac{f_{pe}}{f_{pu}}) v_c b_v d + M_0 \frac{V}{M} \geq 0.1b_v d \sqrt{f_{cu}}$ Note V , M and d refer to V_{dr} M_d and $d_{cen,d}$ respectively;							
$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3} (400/d_{cen,d})^{1/4}$; $\rho_w < 3$; $f_{cu} < 80$; $(400/d_{cen,d})^{1/4} > (0.67 \text{ or}$							
Vertical component of prestress force, $V_p = \gamma_p \cdot KP_0 \sin \beta$					N/A	kN	
Component $(1 - 0.55 f_{pe}/f_{pu}) \cdot v_c b_v d_{cen,d}$					247	kN	cl.4.3.8.5
Component $M_0 \cdot V/M$					423	kN	cl.4.3.8.5
BS8110 and TR.43-1 [PC] BS8110 [RC] E				247	423	kN	
ACI318				N/A	N/A	kN	
AS3600				N/A	N/A	kN	
Ratio, $f_{pe}/f_{pu} = KP_0/[N_T \cdot N_s \cdot A_s + A_{s,prov} \cdot f_y/f_{pk}]/f_{pk} \leq 0.60$				0.40	cl.4.3.8.1 BS8.		
Note f_{pe}/f_{pu} refers to ratio of design effective prestress to ultimate tensile strength in reinforcement							
4.3.8.8 $N_T \cdot N_s \cdot A_s + A_{s,prov}$				4822	mm ²	cl.4.3.8.1	
$\rho_w = 100(N_T \cdot N_s \cdot A_s + A_{s,prov})/b_w d_{cen,d}$				1.12	%	cl.4.3.8.1	
Bending moment for zero tensile stress, $M_0 = 0.8f_{pt} Z_{b/t,(SLS/ULS)}$				494	kNm	cl.4.3.8.1	
Comp. stress at extreme tensile fibre due to prestress, f_{pt}				3.1	N/mm ²		
Sagging Moment		Invalid	Hogging Moment		Valid		
$f_{pt} = \frac{KP_0}{A_{(SLS/ULS)}} + \frac{KP_0 e_{var}(x = x_d)}{Z_{b,(SLS/ULS)}}$		$f_{pt} = \frac{KP_0}{A_{(SLS/ULS)}} - \frac{KP_0 e_{var}(x = x_d)}{Z_{t,(SLS/ULS)}}$					
Note f_{pt} calculated based on actual section, rectangular or flanged, $A_{(SLS/ULS)}$ and $Z_{b/t,(SLS/ULS)}$ proper							
Shear enhancement near support, $k_{enh} = 2d_{cen,d}/x_d$				4.00	cl.3.4.5.8, cl.4.3.8.3		
Design shear resistance, $V_c = \{V_{co} \text{ uncracked, MIN } (V_{co}, V_{cr}) \text{ cracked}\}$				670	kN	cl.4.3.8.3	
Uncracked section ($ M_d < M_{0/ct}$)				1521	<	494	kNm
Cracked section ($ M_d \geq M_{0/ct}$)				1521	≥	494	kNm
Minimum shear strength, $v_r = \text{MAX } (0.4, 0.4(f_{cu}/40)^{2/3})$, $f_{cu} \leq 80 \text{ N/mm}^2$				0.40	N/mm ²	cl.3.4.5.3 BS	
Check $V_d < 0.5k_{enh} \cdot V_c$ (beam) (minor elements) or $1.0k_{enh} \cdot V_c$ (slab) for				VALID	Beam	cl.4.3.8.6	
$k_{enh} \cdot V_c$				2680	kN	cl.4.3.8.5,	
Check 0.0 (beam) or $1.0k_{enh} \cdot V_c$ (slab) $< V_d < k_{enh} \cdot V_c + NL$ for nominal li				VALID	cl.4.3.8.7		
$A_{sv,nom}/S > v_r \cdot b_v / (0.95f_{yv})$, $f_{yv} \leq 460 \text{ N/mm}^2$ i.e. $A_{sv,nom}/S >$				0.46	mm ² /mm	cl.3.4.5.8,	
$k_{enh} \cdot V_c + NL = v_r \cdot b_v d_{cen,d} + k_{enh} \cdot V_c$				2853	kN	cl.3.4.5.8,	
Check $V_d > k_{enh} \cdot V_c + NL$ for design links				N/A	cl.4.3.8.8		
$A_{sv}/S > (V_d - k_{enh} \cdot V_c) / (0.95f_{yv} \cdot d_{max,d})$, $f_{yv} \leq 460 \text{ N/mm}^2$ i.e. $A_{sv}/S >$				0.46	mm ² /mm	cl.3.4.5.8,	
$k_{enh} \cdot V_c + DL = (A_{sv,prov}/S) \cdot (0.95f_{yv}) \cdot d_{max,d} + k_{enh} \cdot V_c$, $f_{yv} \leq 460 \text{ N/mm}^2$				3939	kN	cl.3.4.5.8,	
Area provided by all shear links in a cross-section, $A_{sv,prov}$				314	mm ²		
Tried $A_{sv,prov}/S$ value				3.14	mm ² /mm		
Design shear resistance at (shear) design section utilisation				72%	OK		

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						Member/Location				
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Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
									BS8110 [PC	
Shear at All Sections Rectangular Beam										BS8110 [F ▼
V _{d,var} and e _{var} at All Sections										
Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m		
V _{d,var}	1301	1053	841	728	520	312	104	kN		
M _{d,var}	-1521	-965	-468	271	826	1196	1381	kNm		
e _{var}	-196	-160	-52	175	346	460	517	mm		
d _{cen,d,var}	864	840	766	639	765	848	890	mm		
d _{max,d,var}	917	917	917	937	937	937	937	mm		
v _{d,var}	3.01	2.51	2.20	2.28	1.36	0.74	0.23	N/mm ²		
f _{t/ctd}	1.42	1.42	1.42	1.42	1.42	1.42	1.42	N/mm ²		
f _{cp/pc} [σ _{cp}]	1.3	1.3	1.3	1.3	1.3	1.3	1.3	N/mm ²		
V _{co/cw}	757	885	1053	1024	914	800	684	kN		
f _{pe} /f _{pu}	0.40	0.40	0.40	0.43	0.43	0.43	0.43			
ρ _{w,var}	1.12	1.15	1.26	1.29	1.08	0.97	0.93	%		
f _{pt}	3.1	2.8	1.8	4.2	7.0	8.8	9.8	N/mm ²		
M _{0/ct,var}	494	442	284	370	619	784	867	kNm		
k _{enh}	4.00	3.36	1.53	1.00	1.00	1.00	1.00			
V _{cr/ci/uc,var}	670	724	739	916	602	432	300	kN		
Cracked?	Yes	Yes	Yes	No	Yes	Yes	Yes			
k _{enh} •φV _{c,var}	2680	2433	1133	1024	602	432	300	kN		
φV _{c,var} +k _{enl}	2853	2601	1286	1152	755	602	478	kN		
φV _{c,var} +k _{enl}	3939	3692	2392	2310	1888	1718	1586	kN		
No Links	VALID	VALID	INVALID	INVALID	INVALID	INVALID	VALID			
Nom Links	VALID	VALID	VALID	VALID	VALID	VALID	VALID			
Des Links	N/A	N/A	N/A	N/A	N/A	N/A	N/A			
A _{sv} /S >	0.46	0.46	0.46	0.46	0.46	0.46	0.46	mm ² /mm		
UT	72%	70%	54%	50%	40%	35%	30%	%		
Status	OK	OK	OK	OK	OK	OK	OK			
Dist, x	5.444	6.333	7.222	8.111	9.000	9.500	10.000	m		
V _{d,var}	-104	-312	-520	-728	-841	-1053	-1301	kN		
M _{d,var}	1381	1196	826	271	-468	-965	-1521	kNm		
e _{var}	517	460	346	175	-52	-160	-196	mm		
d _{cen,d,var}	890	848	765	639	766	840	864	mm		
d _{max,d,var}	937	937	937	937	917	917	917	mm		
v _{d,var}	0.23	0.74	1.36	2.28	2.20	2.51	3.01	N/mm ²		
f _{t/ctd}	1.42	1.42	1.42	1.42	1.42	1.42	1.42	N/mm ²		
f _{cp/pc} [σ _{cp}]	1.3	1.3	1.3	1.3	1.3	1.3	1.3	N/mm ²		
V _{co/cw}	-684	-800	-914	-1024	-1053	-885	-757	kN		
f _{pe} /f _{pu}	0.43	0.43	0.43	0.43	0.40	0.40	0.40			
ρ _{w,var}	0.93	0.97	1.08	1.29	1.26	1.15	1.12	%		
f _{pt}	9.8	8.8	7.0	4.2	1.8	2.8	3.1	N/mm ²		
M _{0/ct,var}	867	784	619	370	284	442	494	kNm		
k _{enh}	1.00	1.00	1.00	1.00	1.53	3.36	4.00			
V _{cr/ci/uc,var}	-300	-432	-602	-916	-739	-724	-670	kN		
Cracked?	Yes	Yes	Yes	No	Yes	Yes	Yes			
k _{enh} •φV _{c,var}	-300	-432	-602	-1024	-1133	-2433	-2680	kN		
φV _{c,var} +k _{enl}	-478	-602	-755	-1152	-1286	-2601	-2853	kN		
φV _{c,var} +k _{enl}	-1586	-1718	-1888	-2310	-2392	-3692	-3939	kN		
No Links	VALID	INVALID	INVALID	INVALID	INVALID	VALID	VALID			
Nom Links	VALID	VALID	VALID	VALID	VALID	VALID	VALID			
Des Links	N/A	N/A	N/A	N/A	N/A	N/A	N/A			
A _{sv} /S >	0.46	0.46	0.46	0.46	0.46	0.46	0.46	mm ² /mm		
UT	30%	35%	40%	50%	54%	70%	72%	%		
Status	OK	OK	OK	OK	OK	OK	OK			
Note an arbitrary shear force sign convention is employed; Note above V _{d,var} = V _{ULS,E/E,var} + V _{SLS,S/E,var} ;										
Design shear resistance at (shear) design section utilisation						72%		OK		

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Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
						BS8110 [PC				
						BS8110 [F ▼				
Shear (at All Sections) 										
Note an arbitrary shear force sign convention is employed; Note above $V_{d,var} = V_{ULS,E/E,var} + V_{SLS,S/E,var}$;										

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Member Design - PC Beam and Slab					Made by	XX	Date	24/10/2026	Chd.
L								BS8110 [PC	
								BS8110 [F	▼
First Shear Perimeter			@1.5d _{cen,2}	N/A	to	@0.0d _{cen,2}	N/A	mm	cl.3.7.7.6
Eff. depth to A _s , d _{ps,2} = h-X _{c,(SLS/ULS)} -e _{var} (X=@shear perimeter)					N/A		N/A	mm	Goal Seek
Eff. depth to centroid of A _s and A _{s,prov,h} , d _{cen,2}							N/A	mm	cl.4.3.8.1
Eff. depth to max of A _s and A _{s,prov,h} , d _{max,2}							N/A	mm	3.8.1, cl.4.3.8.1
Shear force at first shear perimeter, V ₂ = (V _t -V _{reduced,2})					N/A		N/A	kN	
					Rectangular		Circular		
IC:	(l _{h,b} +3d _{cen,2}). (l _{h,h} +3d _{cen,2})		(l _{h,D} +3d _{cen,2}) ²		N/A		N/A	m ²	
EC:	(l _{h,b} +1.5d _{cen,2}). (l _{h,h} +3d _{cen,2})		(l _{h,D} +1.5d _{cen,2}). (l _{h,D} +1.5d _{cen,2})		N/A		N/A	m ²	
CC:	(l _{h,b} +1.5d _{cen,2}). (l _{h,h} +1.5d _{cen,2})		(l _{h,D} +1.5d _{cen,2}) ²		N/A		N/A	m ²	
Eff. shear force, V _{eff,2} = (1.15 int., 1.40 edge, 1.50 corner column) . V ₂							N/A	kN	cl.3.7.6
Column first perimeter, u ₂ ≤ {2L+2L ₁ +L ₂ +L ₃ +L ₄ +L ₅ +L ₆ +L ₇ +L ₈ +L ₉ +L ₁₀ +L ₁₁ +L ₁₂ +L ₁₃ +L ₁₄ +L ₁₅ +L ₁₆ +L ₁₇ +L ₁₈ +L ₁₉ +L ₂₀ +L ₂₁ +L ₂₂ +L ₂₃ +L ₂₄ +L ₂₅ +L ₂₆ +L ₂₇ +L ₂₈ +L ₂₉ +L ₃₀ +L ₃₁ +L ₃₂ +L ₃₃ +L ₃₄ +L ₃₅ +L ₃₆ +L ₃₇ +L ₃₈ +L ₃₉ +L ₄₀ +L ₄₁ +L ₄₂ +L ₄₃ +L ₄₄ +L ₄₅ +L ₄₆ +L ₄₇ +L ₄₈ +L ₄₉ +L ₅₀ +L ₅₁ +L ₅₂ +L ₅₃ +L ₅₄ +L ₅₅ +L ₅₆ +L ₅₇ +L ₅₈ +L ₅₉ +L ₆₀ +L ₆₁ +L ₆₂ +L ₆₃ +L ₆₄ +L ₆₅ +L ₆₆ +L ₆₇ +L ₆₈ +L ₆₉ +L ₇₀ +L ₇₁ +L ₇₂ +L ₇₃ +L ₇₄ +L ₇₅ +L ₇₆ +L ₇₇ +L ₇₈ +L ₇₉ +L ₈₀ +L ₈₁ +L ₈₂ +L ₈₃ +L ₈₄ +L ₈₅ +L ₈₆ +L ₈₇ +L ₈₈ +L ₈₉ +L ₉₀ +L ₉₁ +L ₉₂ +L ₉₃ +L ₉₄ +L ₉₅ +L ₉₆ +L ₉₇ +L ₉₈ +L ₉₉ +L ₁₀₀ +L ₁₀₁ +L ₁₀₂ +L ₁₀₃ +L ₁₀₄ +L ₁₀₅ +L ₁₀₆ +L ₁₀₇ +L ₁₀₈ +L ₁₀₉ +L ₁₁₀ +L ₁₁₁ +L ₁₁₂ +L ₁₁₃ +L ₁₁₄ +L ₁₁₅ +L ₁₁₆ +L ₁₁₇ +L ₁₁₈ +L ₁₁₉ +L ₁₂₀ +L ₁₂₁ +L ₁₂₂ +L ₁₂₃ +L ₁₂₄ +L ₁₂₅ +L ₁₂₆ +L ₁₂₇ +L ₁₂₈ +L ₁₂₉ +L ₁₃₀ +L ₁₃₁ +L ₁₃₂ +L ₁₃₃ +L ₁₃₄ +L ₁₃₅ +L ₁₃₆ +L ₁₃₇ +L ₁₃₈ +L ₁₃₉ +L ₁₄₀ +L ₁₄₁ +L ₁₄₂ +L ₁₄₃ +L ₁₄₄ +L ₁₄₅ +L ₁₄₆ +L ₁₄₇ +L ₁₄₈ +L ₁₄₉ +L ₁₅₀ +L ₁₅₁ +L ₁₅₂ +L ₁₅₃ +L ₁₅₄ +L ₁₅₅ +L ₁₅₆ +L ₁₅₇ +L ₁₅₈ +L ₁₅₉ +L ₁₆₀ +L ₁₆₁ +L ₁₆₂ +L ₁₆₃ +L ₁₆₄ +L ₁₆₅ +L ₁₆₆ +L ₁₆₇ +L ₁₆₈ +L ₁₆₉ +L ₁₇₀ +L ₁₇₁ +L ₁₇₂ +L ₁₇₃ +L ₁₇₄ +L ₁₇₅ +L ₁₇₆ +L ₁₇₇ +L ₁₇₈ +L ₁₇₉ +L ₁₈₀ +L ₁₈₁ +L ₁₈₂ +L ₁₈₃ +L ₁₈₄ +L ₁₈₅ +L ₁₈₆ +L ₁₈₇ +L ₁₈₈ +L ₁₈₉ +L ₁₉₀ +L ₁₉₁ +L ₁₉₂ +L ₁₉₃ +L ₁₉₄ +L ₁₉₅ +L ₁₉₆ +L ₁₉₇ +L ₁₉₈ +L ₁₉₉ +L ₂₀₀ +L ₂₀₁ +L ₂₀₂ +L ₂₀₃ +L ₂₀₄ +L ₂₀₅ +L ₂₀₆ +L ₂₀₇ +L ₂₀₈ +L ₂₀₉ +L ₂₁₀ +L ₂₁₁ +L ₂₁₂ +L ₂₁₃ +L ₂₁₄ +L ₂₁₅ +L ₂₁₆ +L ₂₁₇ +L ₂₁₈ +L ₂₁₉ +L ₂₂₀ +L ₂₂₁ +L ₂₂₂ +L ₂₂₃ +L ₂₂₄ +L ₂₂₅ +L ₂₂₆ +L ₂₂₇ +L ₂₂₈ +L ₂₂₉ +L ₂₃₀ +L ₂₃₁ +L ₂₃₂ +L ₂₃₃ +L ₂₃₄ +L ₂₃₅ +L ₂₃₆ +L ₂₃₇ +L ₂₃₈ +L ₂₃₉ +L ₂₄₀ +L ₂₄₁ +L ₂₄₂ +L ₂₄₃ +L ₂₄₄ +L ₂₄₅ +L ₂₄₆ +L ₂₄₇ +L ₂₄₈ +L ₂₄₉ +L ₂₅₀ +L ₂₅₁ +L ₂₅₂ +L ₂₅₃ +L ₂₅₄ +L ₂₅₅ +L ₂₅₆ +L ₂₅₇ +L ₂₅₈ +L ₂₅₉ +L ₂₆₀ +L ₂₆₁ +L ₂₆₂ +L ₂₆₃ +L ₂₆₄ +L ₂₆₅ +L ₂₆₆ +L ₂₆₇ +L ₂₆₈ +L ₂₆₉ +L ₂₇₀ +L ₂₇₁ +L ₂₇₂ +L ₂₇₃ +L ₂₇₄ +L ₂₇₅ +L ₂₇₆ +L ₂₇₇ +L ₂₇₈ +L ₂₇₉ +L ₂₈₀ +L ₂₈₁ +L ₂₈₂ +L ₂₈₃ +L ₂₈₄ +L ₂₈₅ +L ₂₈₆ +L ₂₈₇ +L ₂₈₈ +L ₂₈₉ +L ₂₉₀ +L ₂₉₁ +L ₂₉₂ +L ₂₉₃ +L ₂₉₄ +L ₂₉₅ +L ₂₉₆ +L ₂₉₇ +L ₂₉₈ +L ₂₉₉ +L ₃₀₀ +L ₃₀₁ +L ₃₀₂ +L ₃₀₃ +L ₃₀₄ +L ₃₀₅ +L ₃₀₆ +L ₃₀₇ +L ₃₀₈ +L ₃₀₉ +L ₃₁₀ +L ₃₁₁ +L ₃₁₂ +L ₃₁₃ +L ₃₁₄ +L ₃₁₅ +L ₃₁₆ +L ₃₁₇ +L ₃₁₈ +L ₃₁₉ +L ₃₂₀ +L ₃₂₁ +L ₃₂₂ +L ₃₂₃ +L ₃₂₄ +L ₃₂₅ +L ₃₂₆ +L ₃₂₇ +L ₃₂₈ +L ₃₂₉ +L ₃₃₀ +L ₃₃₁ +L ₃₃₂ +L ₃₃₃ +L ₃₃₄ +L ₃₃₅ +L ₃₃₆ +L ₃₃₇ +L ₃₃₈ +L ₃₃₉ +L ₃₄₀ +L ₃₄₁ +L ₃₄₂ +L ₃₄₃ +L ₃₄₄ +L ₃₄₅ +L ₃₄₆ +L ₃₄₇ +L ₃₄₈ +L ₃₄₉ +L ₃₅₀ +L ₃₅₁ +L ₃₅₂ +L ₃₅₃ +L ₃₅₄ +L ₃₅₅ +L ₃₅₆ +L ₃₅₇ +L ₃₅₈ +L ₃₅₉ +L ₃₆₀ +L ₃₆₁ +L ₃₆₂ +L ₃₆₃ +L ₃₆₄ +L ₃₆₅ +L ₃₆₆ +L ₃₆₇ +L ₃₆₈ +L ₃₆₉ +L ₃₇₀ +L ₃₇₁ +L ₃₇₂ +L ₃₇₃ +L ₃₇₄ +L ₃₇₅ +L ₃₇₆ +L ₃₇₇ +L ₃₇₈ +L ₃₇₉ +L ₃₈₀ +L ₃₈₁ +L ₃₈₂ +L ₃₈₃ +L ₃₈₄ +L ₃₈₅ +L ₃₈₆ +L ₃₈₇ +L ₃₈₈ +L ₃₈₉ +L ₃₉₀ +L ₃₉₁ +L ₃₉₂ +L ₃₉₃ +L ₃₉₄ +L ₃₉₅ +L ₃₉₆ +L ₃₉₇ +L ₃₉₈ +L ₃₉₉ +L ₄₀₀ +L ₄₀₁ +L ₄₀₂ +L ₄₀₃ +L ₄₀₄ +L ₄₀₅ +L ₄₀₆ +L ₄₀₇ +L ₄₀₈ +L ₄₀₉ +L ₄₁₀ +L ₄₁₁ +L ₄₁₂ +L ₄₁₃ +L ₄₁₄ +L ₄₁₅ +L ₄₁₆ +L ₄₁₇ +L ₄₁₈ +L ₄₁₉ +L ₄₂₀ +L ₄₂₁ +L ₄₂₂ +L ₄₂₃ +L ₄₂₄ +L ₄₂₅ +L ₄₂₆ +L ₄₂₇ +L ₄₂₈ +L ₄₂₉ +L ₄₃₀ +L ₄₃₁ +L ₄₃₂ +L ₄₃₃ +L ₄₃₄ +L ₄₃₅ +L ₄₃₆ +L ₄₃₇ +L ₄₃₈ +L ₄₃₉ +L ₄₄₀ +L ₄₄₁ +L ₄₄₂ +L ₄₄₃ +L ₄₄₄ +L ₄₄₅ +L ₄₄₆ +L ₄₄₇ +L ₄₄₈ +L ₄₄₉ +L ₄₅₀ +L ₄₅₁ +L ₄₅₂ +L ₄₅₃ +L ₄₅₄ +L ₄₅₅ +L ₄₅₆ +L ₄₅₇ +L ₄₅₈ +L ₄₅₉ +L ₄₆₀ +L ₄₆₁ +L ₄₆₂ +L ₄₆₃ +L ₄₆₄ +L ₄₆₅ +L ₄₆₆ +L ₄₆₇ +L ₄₆₈ +L ₄₆₉ +L ₄₇₀ +L ₄₇₁ +L ₄₇₂ +L ₄₇₃ +L ₄₇₄ +L ₄₇₅ +L ₄₇₆ +L ₄₇₇ +L ₄₇₈ +L ₄₇₉ +L ₄₈₀ +L ₄₈₁ +L ₄₈₂ +L ₄₈₃ +L ₄₈₄ +L ₄₈₅ +L ₄₈₆ +L ₄₈₇ +L ₄₈₈ +L ₄₈₉ +L ₄₉₀ +L ₄₉₁ +L ₄₉₂ +L ₄₉₃ +L ₄₉₄ +L ₄₉₅ +L ₄₉₆ +L ₄₉₇ +L ₄₉₈ +L ₄₉₉ +L ₅₀₀ +L ₅₀₁ +L ₅₀₂ +L ₅₀₃ +L ₅₀₄ +L ₅₀₅ +L ₅₀₆ +L ₅₀₇ +L ₅₀₈ +L ₅₀₉ +L ₅₁₀ +L ₅₁₁ +L ₅₁₂ +L ₅₁₃ +L ₅₁₄ +L ₅₁₅ +L ₅₁₆ +L ₅₁₇ +L ₅₁₈ +L ₅₁₉ +L ₅₂₀ +L ₅₂₁ +L ₅₂₂ +L ₅₂₃ +L ₅₂₄ +L ₅₂₅ +L ₅₂₆ +L ₅₂₇ +L ₅₂₈ +L ₅₂₉ +L ₅₃₀ +L ₅₃₁ +L ₅₃₂ +L ₅₃₃ +L ₅₃₄ +L ₅₃₅ +L ₅₃₆ +L ₅₃₇ +L ₅₃₈ +L ₅₃₉ +L ₅₄₀ +L ₅₄₁ +L ₅₄₂ +L ₅₄₃ +L ₅₄₄ +L ₅₄₅ +L ₅₄₆ +L ₅₄₇ +L ₅₄₈ +L ₅₄₉ +L ₅₅₀ +L ₅₅₁ +L ₅₅₂ +L ₅₅₃ +L ₅₅₄ +L ₅₅₅ +L ₅₅₆ +L ₅₅₇ +L ₅₅₈ +L ₅₅₉ +L ₅₆₀ +L ₅₆₁ +L ₅₆₂ +L ₅₆₃ +L ₅₆₄ +L ₅₆₅ +L ₅₆₆ +L ₅₆₇ +L ₅₆₈ +L ₅₆₉ +L ₅₇₀ +L ₅₇₁ +L ₅₇₂ +L ₅₇₃ +L ₅₇₄ +L ₅₇₅ +L ₅₇₆ +L ₅₇₇ +L ₅₇₈ +L ₅₇₉ +L ₅₈₀ +L ₅₈₁ +L ₅₈₂ +L ₅₈₃ +L ₅₈₄ +L ₅₈₅ +L ₅₈₆ +L ₅₈₇ +L ₅₈₈ +L ₅₈₉ +L ₅₉₀ +L ₅₉₁ +L ₅₉₂ +L ₅₉₃ +L ₅₉₄ +L ₅₉₅ +L ₅₉₆ +L ₅₉₇ +L ₅₉₈ +L ₅₉₉ +L ₆₀₀ +L ₆₀₁ +L ₆₀₂ +L ₆₀₃ +L ₆₀₄ +L ₆₀₅ +L ₆₀₆ +L ₆₀₇ +L ₆₀₈ +L ₆₀₉ +L ₆₁₀ +L ₆₁₁ +L ₆₁₂ +L ₆₁₃ +L ₆₁₄ +L ₆₁₅ +L ₆₁₆ +L ₆₁₇ +L ₆₁₈ +L ₆₁₉ +L ₆₂₀ +L ₆₂₁ +L ₆₂₂ +L ₆₂₃ +L ₆₂₄ +L ₆₂₅ +L ₆₂₆ +L ₆₂₇ +L ₆₂₈ +L ₆₂₉ +L ₆₃₀ +L ₆₃₁ +L ₆₃₂ +L ₆₃₃ +L ₆₃₄ +L ₆₃₅ +L ₆₃₆ +L ₆₃₇ +L ₆₃₈ +L ₆₃₉ +L ₆₄₀ +L ₆₄₁ +L ₆₄₂ +L ₆₄₃ +L ₆₄₄ +L ₆₄₅ +L ₆₄₆ +L ₆₄₇ +L ₆₄₈ +L ₆₄₉ +L ₆₅₀ +L ₆₅₁ +L ₆₅₂ +L ₆₅₃ +L ₆₅₄ +L ₆₅₅ +L ₆₅₆ +L ₆₅₇ +L ₆₅₈ +L ₆₅₉ +L ₆₆₀ +L ₆₆₁ +L ₆₆₂ +L ₆₆₃ +L ₆₆₄ +L ₆₆₅ +L ₆₆₆ +L ₆₆₇ +L ₆₆₈ +L ₆₆₉ +L ₆₇₀ +L ₆₇₁ +L ₆₇₂ +L ₆₇₃ +L ₆₇₄ +L ₆₇₅ +L ₆₇₆ +L ₆₇₇ +L ₆₇₈ +L ₆₇₉ +L ₆₈₀ +L ₆₈₁ +L ₆₈₂ +L ₆₈₃ +L ₆₈₄ +L ₆₈₅ +L ₆₈₆ +L ₆₈₇ +L ₆₈₈ +L ₆₈₉ +L ₆₉₀ +L ₆₉₁ +L ₆₉₂ +L ₆₉₃ +L ₆₉₄ +L ₆₉₅ +L ₆₉₆ +L ₆₉₇ +L ₆₉₈ +L ₆₉₉ +L ₇₀₀ +L ₇₀₁ +L ₇₀₂ +L ₇₀₃ +L ₇₀₄ +L ₇₀₅ +L ₇₀₆ +L ₇₀₇ +L ₇₀₈ +L ₇₀₉ +L ₇₁₀ +L ₇₁₁ +L ₇₁₂ +L ₇₁₃ +L ₇₁₄ +L ₇₁₅ +L ₇₁₆ +L ₇₁₇ +L ₇₁₈ +L ₇₁₉ +L ₇₂₀ +L ₇₂₁ +L ₇₂₂ +L ₇₂₃ +L ₇₂₄ +L ₇₂₅ +L ₇₂₆ +L ₇₂₇ +L ₇₂₈ +L ₇₂₉ +L ₇₃₀ +L ₇₃₁ +L ₇₃₂ +L ₇₃₃ +L ₇₃₄ +L ₇₃₅ +L ₇₃₆ +L ₇₃₇ +L ₇₃₈ +L ₇₃₉ +L ₇₄₀ +L ₇₄₁ +L ₇₄₂ +L ₇₄₃ +L ₇₄₄ +L ₇₄₅ +L ₇₄₆ +L ₇₄₇ +L ₇₄₈ +L ₇₄₉ +L ₇₅₀ +L ₇₅₁ +L ₇₅₂ +L ₇₅₃ +L ₇₅₄ +L ₇₅₅ +L ₇₅₆ +L ₇₅₇ +L ₇₅₈ +L ₇₅₉ +L ₇₆₀ +L ₇₆₁ +L ₇₆₂ +L ₇₆₃ +L ₇₆₄ +L ₇₆₅ +L ₇₆₆ +L ₇₆₇ +L ₇₆₈ +L ₇₆₉ +L ₇₇₀ +L ₇₇₁ +L ₇₇₂ +L ₇₇₃ +L ₇₇₄ +L ₇₇₅ +L ₇₇₆ +L ₇₇₇ +L ₇₇₈ +L ₇₇₉ +L ₇₈₀ +L ₇₈₁ +L ₇₈₂ +L ₇₈₃ +L ₇₈₄ +L ₇₈₅ +L ₇₈₆ +L ₇₈₇ +L ₇₈₈ +L ₇₈₉ +L ₇₉₀ +L ₇₉₁ +L ₇₉₂ +L ₇₉₃ +L ₇₉₄ +L ₇₉₅ +L ₇₉₆ +L ₇₉₇ +L ₇₉₈ +L ₇₉₉ +L ₈₀₀ +L ₈₀₁ +L ₈₀₂ +L ₈₀₃ +L ₈₀₄ +L ₈₀₅ +L ₈₀₆ +L ₈₀₇ +L ₈₀₈ +L ₈₀₉ +L ₈₁₀ +L ₈₁₁ +L ₈₁₂ +L ₈₁₃ +L ₈₁₄ +L ₈₁₅ +L ₈₁₆ +L ₈₁₇ +L ₈₁₈ +L ₈₁₉ +L ₈₂₀ +L ₈₂₁ +L ₈₂₂ +L ₈₂₃ +L ₈₂₄ +L ₈₂₅ +L ₈₂₆ +L ₈₂₇ +L ₈₂₈ +L ₈₂₉ +L ₈₃₀ +L ₈₃₁ +L ₈₃₂ +L ₈₃₃ +L ₈₃₄ +L ₈₃₅ +L ₈₃₆ +L ₈₃₇ +L ₈₃₈ +L ₈₃₉ +L ₈₄₀ +L ₈₄₁ +L ₈₄₂ +L ₈₄₃ +L ₈₄₄ +L ₈₄₅ +L ₈₄₆ +L ₈₄₇ +L ₈₄₈ +L ₈₄₉ +L ₈₅₀ +L ₈₅₁ +L ₈₅₂ +L ₈₅₃ +L ₈₅₄ +L ₈₅₅ +L ₈₅₆ +L ₈₅₇ +L ₈₅₈ +L ₈₅₉ +L ₈₆₀ +L ₈₆₁ +L ₈₆₂ +L ₈₆₃ +L ₈₆₄ +L ₈₆₅ +L ₈₆₆ +L ₈₆₇ +L ₈₆₈ +L ₈₆₉ +L ₈₇₀ +L ₈₇₁ +L ₈₇₂ +L ₈₇₃ +L ₈₇₄ +L ₈₇₅ +L ₈₇₆ +L ₈₇₇ +L ₈₇₈ +L ₈₇₉ +L ₈₈₀ +L ₈₈₁ +L ₈₈₂ +L ₈₈₃ +L ₈₈₄ +L ₈₈₅ +L ₈₈₆ +L ₈₈₇ +L ₈₈₈ +L ₈₈₉ +L ₈₉₀ +L ₈₉₁ +L ₈₉₂ +L ₈₉₃ +L ₈₉₄ +L ₈₉₅ +L ₈₉₆ +L ₈₉₇ +L ₈₉₈ +L ₈₉₉ +L ₉₀₀ +L ₉₀₁ +L ₉₀₂ +L ₉₀₃ +L ₉₀₄ +L ₉₀₅ +L ₉₀₆ +L ₉₀₇ +L ₉₀₈ +L ₉₀₉ +L ₉₁₀ +L ₉₁₁ +L ₉₁₂ +L ₉₁₃ +L ₉₁₄ +L ₉₁₅ +L ₉₁₆ +L ₉₁₇ +L ₉₁₈ +L ₉₁₉ +L ₉₂₀ +L ₉₂₁ +L ₉₂₂ +L ₉₂₃ +L ₉₂₄ +L ₉₂₅ +L ₉₂₆ +L ₉₂₇ +L ₉₂₈ +L ₉₂₉ +L ₉₃₀ +L ₉₃₁ +L ₉₃₂ +L ₉₃₃ +L ₉₃₄ +L ₉₃₅ +L ₉₃₆ +L ₉₃₇ +L ₉₃₈ +L ₉₃₉ +L ₉₄₀ +L ₉₄₁ +L ₉₄₂ +L ₉₄₃ +L ₉₄₄ +L ₉₄₅ +L ₉₄₆ +L ₉₄₇ +L ₉₄₈ +L ₉₄₉ +L ₉₅₀ +L ₉₅₁ +L ₉₅₂ +L ₉₅₃ +L ₉₅₄ +L ₉₅₅ +L ₉₅₆ +L ₉₅₇ +L ₉₅₈ +L ₉₅₉ +L ₉₆₀ +L ₉₆₁ +L ₉₆₂ +L ₉₆₃ +L ₉₆₄ +L ₉₆₅ +L ₉₆₆ +L ₉₆₇ +L ₉₆₈ +L ₉₆₉ +L ₉₇₀ +L ₉₇₁ +L ₉₇₂ +L ₉₇₃ +L ₉₇₄ +L ₉₇₅ +L ₉₇₆ +L ₉₇₇ +L ₉₇₈ +L ₉₇₉ +L ₉₈₀ +L ₉₈₁ +L ₉₈₂ +L ₉₈₃ +L ₉₈₄ +L ₉₈₅ +L ₉₈₆ +L ₉₈₇ +L ₉₈₈ +L ₉₈₉ +L ₉₉₀ +L ₉₉₁ +L ₉₉₂ +L ₉₉₃ +L ₉₉₄ +L ₉₉₅ +L ₉₉₆ +L ₉₉₇ +L ₉₉₈ +L ₉₉₉ +L ₁₀₀₀ +L ₁₀₀₁ +L ₁₀₀₂ +L ₁₀₀₃ +L ₁₀₀₄ +L ₁₀₀₅ +L ₁₀₀₆ +L ₁₀₀₇ +L ₁₀₀₈ +L ₁₀₀₉ +L ₁₀₁₀ +L ₁₀₁₁ +L ₁₀₁₂ +L ₁₀₁₃ +L ₁₀₁₄ +L ₁₀₁₅ +L ₁₀₁₆ +L ₁₀₁₇ +L ₁₀₁₈ +L ₁₀₁₉ +L ₁₀₂₀ +L ₁₀₂₁ +L ₁₀₂₂ +L ₁₀₂₃ +L ₁₀₂₄ +L ₁₀₂₅ +L ₁₀₂₆ +L ₁₀₂₇ +L ₁₀₂₈ +L ₁₀₂₉ +L ₁₀₃₀ +L ₁₀₃₁ +L ₁₀₃₂ +L ₁₀₃₃ +L ₁₀₃₄ +L ₁₀₃₅ +L ₁₀₃₆ +L ₁₀₃₇ +L ₁₀₃₈ +L ₁₀₃₉ +L ₁₀₄₀ +L ₁₀₄₁ +L ₁₀₄₂ +L ₁₀₄₃ +L ₁₀₄₄ +L ₁₀₄₅ +L ₁₀₄₆ +L ₁₀₄₇ +L ₁₀₄₈ +L ₁₀₄₉ +L ₁₀₅₀ +L ₁₀₅₁ +L ₁₀₅₂ +L ₁₀₅₃ +L ₁₀₅₄ +L ₁₀₅₅ +L ₁₀₅₆ +L ₁₀₅₇ +L ₁₀₅₈ +L ₁₀₅₉ +L ₁₀₆₀ +L ₁₀₆₁ +L ₁₀₆₂ +L ₁₀₆₃ +L ₁₀₆₄ +L ₁₀₆₅ +L ₁₀₆₆ +L ₁₀₆₇ +L ₁₀₆₈ +L ₁₀₆₉ +L ₁₀₇₀ +L ₁₀₇₁ +L ₁₀₇₂ +L ₁₀₇₃ +L ₁₀₇₄ +L ₁₀₇₅ +L ₁₀₇₆ +L ₁₀₇₇ +L ₁₀₇₈ +L ₁₀₇₉ +L ₁₀₈₀ +L ₁₀₈₁ +L ₁₀₈₂ +L ₁₀₈₃ +L ₁₀₈₄ +L ₁₀₈₅ +L ₁₀₈₆ +L ₁₀₈₇ +L ₁₀₈₈ +L ₁₀₈₉ +L ₁₀₉₀ +L ₁₀₉₁ +L ₁₀₉₂ +L ₁₀₉₃ +L ₁₀₉₄ +L ₁₀₉₅ +L ₁₀₉₆ +L ₁₀₉₇ +L ₁₀₉₈ +L ₁₀₉₉ +L ₁₁₀₀ +L ₁₁₀₁ +L ₁₁₀₂ +L ₁₁₀₃ +L ₁₁₀₄ +L ₁₁₀₅ +L ₁₁₀₆ +L ₁₁₀₇ +L ₁₁₀₈ +L ₁₁₀₉ +L ₁₁₁₀ +L ₁₁₁₁									

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	43			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
L									BS8110 [PC	
									BS8110 [F	▼
Second Shear Perimeter			@2.25d _{cen,3}	N/A	to	@0.75d _{cen,3}	N/A	mm	cl.3.7.7.6	
Eff. depth to A _s , d _{ps,3} = h-x _c (SLS/ULS)-e _{var} (x=@shear perimeter)						N/A	N/A	mm	Goal Seek	
Eff. depth to centroid of A _s and A _{s,prov,h} , d _{cen,3}							N/A	mm	cl.4.3.8.1	
Eff. depth to max of A _s and A _{s,prov,h} , d _{max,3}							N/A	mm	3.8.1, cl.4.3	
Shear force at second shear perimeter, V ₃ = (V _t -V _{reduced,3})						N/A	N/A	kN		
						Rectangular	Circular			
IC:	(l _{h,b} + 4.5d _{cen,3}).		(l _{h,h} + 4.5d _{cen,3}).		(l _{h,D} + 4.5d _{cen,3}) ²	N/A	N/A	m ²		
EC:	(l _{h,b} + 2.25d _{cen,3}).		(l _{h,h} + 4.5d _{cen,3}).		(l _{h,D} + 2.25d _{cen,3}).(l _{h,h} + 2.25d _{cen,3})	N/A	N/A	m ²		
CC:	(l _{h,b} + 2.25d _{cen,3}).		(l _{h,h} + 2.25d _{cen,3}).		(l _{h,D} + 2.25d _{cen,3}) ²	N/A	N/A	m ²		
Eff. shear force, V _{eff,3} = (1.15 int., 1.40 edge, 1.50 corner column) . V ₃							N/A	kN	cl.3.7.6	
Column second perimeter, u ₃ ≤ {2L+2L ₁ +L ₂ +L ₃ +L ₄ +L ₅ +L ₆ +L ₇ +L ₈ +L ₉ +L ₁₀ +L ₁₁ +L ₁₂ +L ₁₃ +L ₁₄ +L ₁₅ +L ₁₆ +L ₁₇ +L ₁₈ +L ₁₉ +L ₂₀ +L ₂₁ +L ₂₂ +L ₂₃ +L ₂₄ +L ₂₅ +L ₂₆ +L ₂₇ +L ₂₈ +L ₂₉ +L ₃₀ +L ₃₁ +L ₃₂ +L ₃₃ +L ₃₄ +L ₃₅ +L ₃₆ +L ₃₇ +L ₃₈ +L ₃₉ +L ₄₀ +L ₄₁ +L ₄₂ +L ₄₃ +L ₄₄ +L ₄₅ +L ₄₆ +L ₄₇ +L ₄₈ +L ₄₉ +L ₅₀ +L ₅₁ +L ₅₂ +L ₅₃ +L ₅₄ +L ₅₅ +L ₅₆ +L ₅₇ +L ₅₈ +L ₅₉ +L ₆₀ +L ₆₁ +L ₆₂ +L ₆₃ +L ₆₄ +L ₆₅ +L ₆₆ +L ₆₇ +L ₆₈ +L ₆₉ +L ₇₀ +L ₇₁ +L ₇₂ +L ₇₃ +L ₇₄ +L ₇₅ +L ₇₆ +L ₇₇ +L ₇₈ +L ₇₉ +L ₈₀ +L ₈₁ +L ₈₂ +L ₈₃ +L ₈₄ +L ₈₅ +L ₈₆ +L ₈₇ +L ₈₈ +L ₈₉ +L ₉₀ +L ₉₁ +L ₉₂ +L ₉₃ +L ₉₄ +L ₉₅ +L ₉₆ +L ₉₇ +L ₉₈ +L ₉₉ +L ₁₀₀ +L ₁₀₁ +L ₁₀₂ +L ₁₀₃ +L ₁₀₄ +L ₁₀₅ +L ₁₀₆ +L ₁₀₇ +L ₁₀₈ +L ₁₀₉ +L ₁₁₀ +L ₁₁₁ +L ₁₁₂ +L ₁₁₃ +L ₁₁₄ +L ₁₁₅ +L ₁₁₆ +L ₁₁₇ +L ₁₁₈ +L ₁₁₉ +L ₁₂₀ +L ₁₂₁ +L ₁₂₂ +L ₁₂₃ +L ₁₂₄ +L ₁₂₅ +L ₁₂₆ +L ₁₂₇ +L ₁₂₈ +L ₁₂₉ +L ₁₃₀ +L ₁₃₁ +L ₁₃₂ +L ₁₃₃ +L ₁₃₄ +L ₁₃₅ +L ₁₃₆ +L ₁₃₇ +L ₁₃₈ +L ₁₃₉ +L ₁₄₀ +L ₁₄₁ +L ₁₄₂ +L ₁₄₃ +L ₁₄₄ +L ₁₄₅ +L ₁₄₆ +L ₁₄₇ +L ₁₄₈ +L ₁₄₉ +L ₁₅₀ +L ₁₅₁ +L ₁₅₂ +L ₁₅₃ +L ₁₅₄ +L ₁₅₅ +L ₁₅₆ +L ₁₅₇ +L ₁₅₈ +L ₁₅₉ +L ₁₆₀ +L ₁₆₁ +L ₁₆₂ +L ₁₆₃ +L ₁₆₄ +L ₁₆₅ +L ₁₆₆ +L ₁₆₇ +L ₁₆₈ +L ₁₆₉ +L ₁₇₀ +L ₁₇₁ +L ₁₇₂ +L ₁₇₃ +L ₁₇₄ +L ₁₇₅ +L ₁₇₆ +L ₁₇₇ +L ₁₇₈ +L ₁₇₉ +L ₁₈₀ +L ₁₈₁ +L ₁₈₂ +L ₁₈₃ +L ₁₈₄ +L ₁₈₅ +L ₁₈₆ +L ₁₈₇ +L ₁₈₈ +L ₁₈₉ +L ₁₉₀ +L ₁₉₁ +L ₁₉₂ +L ₁₉₃ +L ₁₉₄ +L ₁₉₅ +L ₁₉₆ +L ₁₉₇ +L ₁₉₈ +L ₁₉₉ +L ₂₀₀ +L ₂₀₁ +L ₂₀₂ +L ₂₀₃ +L ₂₀₄ +L ₂₀₅ +L ₂₀₆ +L ₂₀₇ +L ₂₀₈ +L ₂₀₉ +L ₂₁₀ +L ₂₁₁ +L ₂₁₂ +L ₂₁₃ +L ₂₁₄ +L ₂₁₅ +L ₂₁₆ +L ₂₁₇ +L ₂₁₈ +L ₂₁₉ +L ₂₂₀ +L ₂₂₁ +L ₂₂₂ +L ₂₂₃ +L ₂₂₄ +L ₂₂₅ +L ₂₂₆ +L ₂₂₇ +L ₂₂₈ +L ₂₂₉ +L ₂₃₀ +L ₂₃₁ +L ₂₃₂ +L ₂₃₃ +L ₂₃₄ +L ₂₃₅ +L ₂₃₆ +L ₂₃₇ +L ₂₃₈ +L ₂₃₉ +L ₂₄₀ +L ₂₄₁ +L ₂₄₂ +L ₂₄₃ +L ₂₄₄ +L ₂₄₅ +L ₂₄₆ +L ₂₄₇ +L ₂₄₈ +L ₂₄₉ +L ₂₅₀ +L ₂₅₁ +L ₂₅₂ +L ₂₅₃ +L ₂₅₄ +L ₂₅₅ +L ₂₅₆ +L ₂₅₇ +L ₂₅₈ +L ₂₅₉ +L ₂₆₀ +L ₂₆₁ +L ₂₆₂ +L ₂₆₃ +L ₂₆₄ +L ₂₆₅ +L ₂₆₆ +L ₂₆₇ +L ₂₆₈ +L ₂₆₉ +L ₂₇₀ +L ₂₇₁ +L ₂₇₂ +L ₂₇₃ +L ₂₇₄ +L ₂₇₅ +L ₂₇₆ +L ₂₇₇ +L ₂₇₈ +L ₂₇₉ +L ₂₈₀ +L ₂₈₁ +L ₂₈₂ +L ₂₈₃ +L ₂₈₄ +L ₂₈₅ +L ₂₈₆ +L ₂₈₇ +L ₂₈₈ +L ₂₈₉ +L ₂₉₀ +L ₂₉₁ +L ₂₉₂ +L ₂₉₃ +L ₂₉₄ +L ₂₉₅ +L ₂₉₆ +L ₂₉₇ +L ₂₉₈ +L ₂₉₉ +L ₃₀₀ +L ₃₀₁ +L ₃₀₂ +L ₃₀₃ +L ₃₀₄ +L ₃₀₅ +L ₃₀₆ +L ₃₀₇ +L ₃₀₈ +L ₃₀₉ +L ₃₁₀ +L ₃₁₁ +L ₃₁₂ +L ₃₁₃ +L ₃₁₄ +L ₃₁₅ +L ₃₁₆ +L ₃₁₇ +L ₃₁₈ +L ₃₁₉ +L ₃₂₀ +L ₃₂₁ +L ₃₂₂ +L ₃₂₃ +L ₃₂₄ +L ₃₂₅ +L ₃₂₆ +L ₃₂₇ +L ₃₂₈ +L ₃₂₉ +L ₃₃₀ +L ₃₃₁ +L ₃₃₂ +L ₃₃₃ +L ₃₃₄ +L ₃₃₅ +L ₃₃₆ +L ₃₃₇ +L ₃₃₈ +L ₃₃₉ +L ₃₄₀ +L ₃₄₁ +L ₃₄₂ +L ₃₄₃ +L ₃₄₄ +L ₃₄₅ +L ₃₄₆ +L ₃₄₇ +L ₃₄₈ +L ₃₄₉ +L ₃₅₀ +L ₃₅₁ +L ₃₅₂ +L ₃₅₃ +L ₃₅₄ +L ₃₅₅ +L ₃₅₆ +L ₃₅₇ +L ₃₅₈ +L ₃₅₉ +L ₃₆₀ +L ₃₆₁ +L ₃₆₂ +L ₃₆₃ +L ₃₆₄ +L ₃₆₅ +L ₃₆₆ +L ₃₆₇ +L ₃₆₈ +L ₃₆₉ +L ₃₇₀ +L ₃₇₁ +L ₃₇₂ +L ₃₇₃ +L ₃₇₄ +L ₃₇₅ +L ₃₇₆ +L ₃₇₇ +L ₃₇₈ +L ₃₇₉ +L ₃₈₀ +L ₃₈₁ +L ₃₈₂ +L ₃₈₃ +L ₃₈₄ +L ₃₈₅ +L ₃₈₆ +L ₃₈₇ +L ₃₈₈ +L ₃₈₉ +L ₃₉₀ +L ₃₉₁ +L ₃₉₂ +L ₃₉₃ +L ₃₉₄ +L ₃₉₅ +L ₃₉₆ +L ₃₉₇ +L ₃₉₈ +L ₃₉₉ +L ₄₀₀ +L ₄₀₁ +L ₄₀₂ +L ₄₀₃ +L ₄₀₄ +L ₄₀₅ +L ₄₀₆ +L ₄₀₇ +L ₄₀₈ +L ₄₀₉ +L ₄₁₀ +L ₄₁₁ +L ₄₁₂ +L ₄₁₃ +L ₄₁₄ +L ₄₁₅ +L ₄₁₆ +L ₄₁₇ +L ₄₁₈ +L ₄₁₉ +L ₄₂₀ +L ₄₂₁ +L ₄₂₂ +L ₄₂₃ +L ₄₂₄ +L ₄₂₅ +L ₄₂₆ +L ₄₂₇ +L ₄₂₈ +L ₄₂₉ +L ₄₃₀ +L ₄₃₁ +L ₄₃₂ +L ₄₃₃ +L ₄₃₄ +L ₄₃₅ +L ₄₃₆ +L ₄₃₇ +L ₄₃₈ +L ₄₃₉ +L ₄₄₀ +L ₄₄₁ +L ₄₄₂ +L ₄₄₃ +L ₄₄₄ +L ₄₄₅ +L ₄₄₆ +L ₄₄₇ +L ₄₄₈ +L ₄₄₉ +L ₄₅₀ +L ₄₅₁ +L ₄₅₂ +L ₄₅₃ +L ₄₅₄ +L ₄₅₅ +L ₄₅₆ +L ₄₅₇ +L ₄₅₈ +L ₄₅₉ +L ₄₆₀ +L ₄₆₁ +L ₄₆₂ +L ₄₆₃ +L ₄₆₄ +L ₄₆₅ +L ₄₆₆ +L ₄₆₇ +L ₄₆₈ +L ₄₆₉ +L ₄₇₀ +L ₄₇₁ +L ₄₇₂ +L ₄₇₃ +L ₄₇₄ +L ₄₇₅ +L ₄₇₆ +L ₄₇₇ +L ₄₇₈ +L ₄₇₉ +L ₄₈₀ +L ₄₈₁ +L ₄₈₂ +L ₄₈₃ +L ₄₈₄ +L ₄₈₅ +L ₄₈₆ +L ₄₈₇ +L ₄₈₈ +L ₄₈₉ +L ₄₉₀ +L ₄₉₁ +L ₄₉₂ +L ₄₉₃ +L ₄₉₄ +L ₄₉₅ +L ₄₉₆ +L ₄₉₇ +L ₄₉₈ +L ₄₉₉ +L ₅₀₀ +L ₅₀₁ +L ₅₀₂ +L ₅₀₃ +L ₅₀₄ +L ₅₀₅ +L ₅₀₆ +L ₅₀₇ +L ₅₀₈ +L ₅₀₉ +L ₅₁₀ +L ₅₁₁ +L ₅₁₂ +L ₅₁₃ +L ₅₁₄ +L ₅₁₅ +L ₅₁₆ +L ₅₁₇ +L ₅₁₈ +L ₅₁₉ +L ₅₂₀ +L ₅₂₁ +L ₅₂₂ +L ₅₂₃ +L ₅₂₄ +L ₅₂₅ +L ₅₂₆ +L ₅₂₇ +L ₅₂₈ +L ₅₂₉ +L ₅₃₀ +L ₅₃₁ +L ₅₃₂ +L ₅₃₃ +L ₅₃₄ +L ₅₃₅ +L ₅₃₆ +L ₅₃₇ +L ₅₃₈ +L ₅₃₉ +L ₅₄₀ +L ₅₄₁ +L ₅₄₂ +L ₅₄₃ +L ₅₄₄ +L ₅₄₅ +L ₅₄₆ +L ₅₄₇ +L ₅₄₈ +L ₅₄₉ +L ₅₅₀ +L ₅₅₁ +L ₅₅₂ +L ₅₅₃ +L ₅₅₄ +L ₅₅₅ +L ₅₅₆ +L ₅₅₇ +L ₅₅₈ +L ₅₅₉ +L ₅₆₀ +L ₅₆₁ +L ₅₆₂ +L ₅₆₃ +L ₅₆₄ +L ₅₆₅ +L ₅₆₆ +L ₅₆₇ +L ₅₆₈ +L ₅₆₉ +L ₅₇₀ +L ₅₇₁ +L ₅₇₂ +L ₅₇₃ +L ₅₇₄ +L ₅₇₅ +L ₅₇₆ +L ₅₇₇ +L ₅₇₈ +L ₅₇₉ +L ₅₈₀ +L ₅₈₁ +L ₅₈₂ +L ₅₈₃ +L ₅₈₄ +L ₅₈₅ +L ₅₈₆ +L ₅₈₇ +L ₅₈₈ +L ₅₈₉ +L ₅₉₀ +L ₅₉₁ +L ₅₉₂ +L ₅₉₃ +L ₅₉₄ +L ₅₉₅ +L ₅₉₆ +L ₅₉₇ +L ₅₉₈ +L ₅₉₉ +L ₆₀₀ +L ₆₀₁ +L ₆₀₂ +L ₆₀₃ +L ₆₀₄ +L ₆₀₅ +L ₆₀₆ +L ₆₀₇ +L ₆₀₈ +L ₆₀₉ +L ₆₁₀ +L ₆₁₁ +L ₆₁₂ +L ₆₁₃ +L ₆₁₄ +L ₆₁₅ +L ₆₁₆ +L ₆₁₇ +L ₆₁₈ +L ₆₁₉ +L ₆₂₀ +L ₆₂₁ +L ₆₂₂ +L ₆₂₃ +L ₆₂₄ +L ₆₂₅ +L ₆₂₆ +L ₆₂₇ +L ₆₂₈ +L ₆₂₉ +L ₆₃₀ +L ₆₃₁ +L ₆₃₂ +L ₆₃₃ +L ₆₃₄ +L ₆₃₅ +L ₆₃₆ +L ₆₃₇ +L ₆₃₈ +L ₆₃₉ +L ₆₄₀ +L ₆₄₁ +L ₆₄₂ +L ₆₄₃ +L ₆₄₄ +L ₆₄₅ +L ₆₄₆ +L ₆₄₇ +L ₆₄₈ +L ₆₄₉ +L ₆₅₀ +L ₆₅₁ +L ₆₅₂ +L ₆₅₃ +L ₆₅₄ +L ₆₅₅ +L ₆₅₆ +L ₆₅₇ +L ₆₅₈ +L ₆₅₉ +L ₆₆₀ +L ₆₆₁ +L ₆₆₂ +L ₆₆₃ +L ₆₆₄ +L ₆₆₅ +L ₆₆₆ +L ₆₆₇ +L ₆₆₈ +L ₆₆₉ +L ₆₇₀ +L ₆₇₁ +L ₆₇₂ +L ₆₇₃ +L ₆₇₄ +L ₆₇₅ +L ₆₇₆ +L ₆₇₇ +L ₆₇₈ +L ₆₇₉ +L ₆₈₀ +L ₆₈₁ +L ₆₈₂ +L ₆₈₃ +L ₆₈₄ +L ₆₈₅ +L ₆₈₆ +L ₆₈₇ +L ₆₈₈ +L ₆₈₉ +L ₆₉₀ +L ₆₉₁ +L ₆₉₂ +L ₆₉₃ +L ₆₉₄ +L ₆₉₅ +L ₆₉₆ +L ₆₉₇ +L ₆₉₈ +L ₆₉₉ +L ₇₀₀ +L ₇₀₁ +L ₇₀₂ +L ₇₀₃ +L ₇₀₄ +L ₇₀₅ +L ₇₀₆ +L ₇₀₇ +L ₇₀₈ +L ₇₀₉ +L ₇₁₀ +L ₇₁₁ +L ₇₁₂ +L ₇₁₃ +L ₇₁₄ +L ₇₁₅ +L ₇₁₆ +L ₇₁₇ +L ₇₁₈ +L ₇₁₉ +L ₇₂₀ +L ₇₂₁ +L ₇₂₂ +L ₇₂₃ +L ₇₂₄ +L ₇₂₅ +L ₇₂₆ +L ₇₂₇ +L ₇₂₈ +L ₇₂₉ +L ₇₃₀ +L ₇₃₁ +L ₇₃₂ +L ₇₃₃ +L ₇₃₄ +L ₇₃₅ +L ₇₃₆ +L ₇₃₇ +L ₇₃₈ +L ₇₃₉ +L ₇₄₀ +L ₇₄₁ +L ₇₄₂ +L ₇₄₃ +L ₇₄₄ +L ₇₄₅ +L ₇₄₆ +L ₇₄₇ +L ₇₄₈ +L ₇₄₉ +L ₇₅₀ +L ₇₅₁ +L ₇₅₂ +L ₇₅₃ +L ₇₅₄ +L ₇₅₅ +L ₇₅₆ +L ₇₅₇ +L ₇₅₈ +L ₇₅₉ +L ₇₆₀ +L ₇₆₁ +L ₇₆₂ +L ₇₆₃ +L ₇₆₄ +L ₇₆₅ +L ₇₆₆ +L ₇₆₇ +L ₇₆₈ +L ₇₆₉ +L ₇₇₀ +L ₇₇₁ +L ₇₇₂ +L ₇₇₃ +L ₇₇₄ +L ₇₇₅ +L ₇₇₆ +L ₇₇₇ +L ₇₇₈ +L ₇₇₉ +L ₇₈₀ +L ₇₈₁ +L ₇₈₂ +L ₇₈₃ +L ₇₈₄ +L ₇₈₅ +L ₇₈₆ +L ₇₈₇ +L ₇₈₈ +L ₇₈₉ +L ₇₉₀ +L ₇₉₁ +L ₇₉₂ +L ₇₉₃ +L ₇₉₄ +L ₇₉₅ +L ₇₉₆ +L ₇₉₇ +L ₇₉₈ +L ₇₉₉ +L ₈₀₀ +L ₈₀₁ +L ₈₀₂ +L ₈₀₃ +L ₈₀₄ +L ₈₀₅ +L ₈₀₆ +L ₈₀₇ +L ₈₀₈ +L ₈₀₉ +L ₈₁₀ +L ₈₁₁ +L ₈₁₂ +L ₈₁₃ +L ₈₁₄ +L ₈₁₅ +L ₈₁₆ +L ₈₁₇ +L ₈₁₈ +L ₈₁₉ +L ₈₂₀ +L ₈₂₁ +L ₈₂₂ +L ₈₂₃ +L ₈₂₄ +L ₈₂₅ +L ₈₂₆ +L _{82</}										

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	44			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
L									BS8110 [PC	
									BS8110 [F	▼
Third Shear Perimeter			@3.0d _{cen,4}	N/A	to	@1.5d _{cen,4}	N/A	mm	cl.3.7.7.6	
Eff. depth to A _s , d _{ps,4} = h-X _c (SLS/ULS)-e _{var} (x=@shear perimeter)						N/A	N/A	mm	Goal Seek	
Eff. depth to centroid of A _s and A _{s,prov,h} , d _{cen,4}							N/A	mm	cl.4.3.8.1	
Eff. depth to max of A _s and A _{s,prov,h} , d _{max,4}							N/A	mm	3.8.1, cl.4.3	
Shear force at third shear perimeter, V ₄ = (V _t -V _{reduced,4})						N/A	N/A	kN		
						Rectangular	Circular			
IC:	(l _{h,b} +6d _{cen,4}). (l _{h,h} +6d _{cen,4})		(l _{h,D} +6d _{cen,4}) ²		N/A	N/A	m ²			
EC:	(l _{h,b} +3d _{cen,4}). (l _{h,h} +6d _{cen,4}) or (l _{h,D} +3d _{cen,4}). (l _{h,D} +6d _{cen,4})				N/A	N/A	m ²			
CC:	(l _{h,b} +3d _{cen,4}). (l _{h,h} +3d _{cen,4})		(l _{h,D} +3d _{cen,4}) ²		N/A	N/A	m ²			
Eff. shear force, V _{eff,4} = (1.15 int., 1.40 edge, 1.50 corner column) . V ₄							N/A	kN	cl.3.7.6	
Column third perimeter, u ₄ ≤ {2L+2L ₁ +L ₂ +L ₃ +L ₄ +L ₅ +L ₆ +L ₇ +L ₈ +L ₉ +L ₁₀ +L ₁₁ +L ₁₂ +L ₁₃ +L ₁₄ +L ₁₅ +L ₁₆ +L ₁₇ +L ₁₈ +L ₁₉ +L ₂₀ +L ₂₁ +L ₂₂ +L ₂₃ +L ₂₄ +L ₂₅ +L ₂₆ +L ₂₇ +L ₂₈ +L ₂₉ +L ₃₀ +L ₃₁ +L ₃₂ +L ₃₃ +L ₃₄ +L ₃₅ +L ₃₆ +L ₃₇ +L ₃₈ +L ₃₉ +L ₄₀ +L ₄₁ +L ₄₂ +L ₄₃ +L ₄₄ +L ₄₅ +L ₄₆ +L ₄₇ +L ₄₈ +L ₄₉ +L ₅₀ +L ₅₁ +L ₅₂ +L ₅₃ +L ₅₄ +L ₅₅ +L ₅₆ +L ₅₇ +L ₅₈ +L ₅₉ +L ₆₀ +L ₆₁ +L ₆₂ +L ₆₃ +L ₆₄ +L ₆₅ +L ₆₆ +L ₆₇ +L ₆₈ +L ₆₉ +L ₇₀ +L ₇₁ +L ₇₂ +L ₇₃ +L ₇₄ +L ₇₅ +L ₇₆ +L ₇₇ +L ₇₈ +L ₇₉ +L ₈₀ +L ₈₁ +L ₈₂ +L ₈₃ +L ₈₄ +L ₈₅ +L ₈₆ +L ₈₇ +L ₈₈ +L ₈₉ +L ₉₀ +L ₉₁ +L ₉₂ +L ₉₃ +L ₉₄ +L ₉₅ +L ₉₆ +L ₉₇ +L ₉₈ +L ₉₉ +L ₁₀₀ +L ₁₀₁ +L ₁₀₂ +L ₁₀₃ +L ₁₀₄ +L ₁₀₅ +L ₁₀₆ +L ₁₀₇ +L ₁₀₈ +L ₁₀₉ +L ₁₁₀ +L ₁₁₁ +L ₁₁₂ +L ₁₁₃ +L ₁₁₄ +L ₁₁₅ +L ₁₁₆ +L ₁₁₇ +L ₁₁₈ +L ₁₁₉ +L ₁₂₀ +L ₁₂₁ +L ₁₂₂ +L ₁₂₃ +L ₁₂₄ +L ₁₂₅ +L ₁₂₆ +L ₁₂₇ +L ₁₂₈ +L ₁₂₉ +L ₁₃₀ +L ₁₃₁ +L ₁₃₂ +L ₁₃₃ +L ₁₃₄ +L ₁₃₅ +L ₁₃₆ +L ₁₃₇ +L ₁₃₈ +L ₁₃₉ +L ₁₄₀ +L ₁₄₁ +L ₁₄₂ +L ₁₄₃ +L ₁₄₄ +L ₁₄₅ +L ₁₄₆ +L ₁₄₇ +L ₁₄₈ +L ₁₄₉ +L ₁₅₀ +L ₁₅₁ +L ₁₅₂ +L ₁₅₃ +L ₁₅₄ +L ₁₅₅ +L ₁₅₆ +L ₁₅₇ +L ₁₅₈ +L ₁₅₉ +L ₁₆₀ +L ₁₆₁ +L ₁₆₂ +L ₁₆₃ +L ₁₆₄ +L ₁₆₅ +L ₁₆₆ +L ₁₆₇ +L ₁₆₈ +L ₁₆₉ +L ₁₇₀ +L ₁₇₁ +L ₁₇₂ +L ₁₇₃ +L ₁₇₄ +L ₁₇₅ +L ₁₇₆ +L ₁₇₇ +L ₁₇₈ +L ₁₇₉ +L ₁₈₀ +L ₁₈₁ +L ₁₈₂ +L ₁₈₃ +L ₁₈₄ +L ₁₈₅ +L ₁₈₆ +L ₁₈₇ +L ₁₈₈ +L ₁₈₉ +L ₁₉₀ +L ₁₉₁ +L ₁₉₂ +L ₁₉₃ +L ₁₉₄ +L ₁₉₅ +L ₁₉₆ +L ₁₉₇ +L ₁₉₈ +L ₁₉₉ +L ₂₀₀ +L ₂₀₁ +L ₂₀₂ +L ₂₀₃ +L ₂₀₄ +L ₂₀₅ +L ₂₀₆ +L ₂₀₇ +L ₂₀₈ +L ₂₀₉ +L ₂₁₀ +L ₂₁₁ +L ₂₁₂ +L ₂₁₃ +L ₂₁₄ +L ₂₁₅ +L ₂₁₆ +L ₂₁₇ +L ₂₁₈ +L ₂₁₉ +L ₂₂₀ +L ₂₂₁ +L ₂₂₂ +L ₂₂₃ +L ₂₂₄ +L ₂₂₅ +L ₂₂₆ +L ₂₂₇ +L ₂₂₈ +L ₂₂₉ +L ₂₃₀ +L ₂₃₁ +L ₂₃₂ +L ₂₃₃ +L ₂₃₄ +L ₂₃₅ +L ₂₃₆ +L ₂₃₇ +L ₂₃₈ +L ₂₃₉ +L ₂₄₀ +L ₂₄₁ +L ₂₄₂ +L ₂₄₃ +L ₂₄₄ +L ₂₄₅ +L ₂₄₆ +L ₂₄₇ +L ₂₄₈ +L ₂₄₉ +L ₂₅₀ +L ₂₅₁ +L ₂₅₂ +L ₂₅₃ +L ₂₅₄ +L ₂₅₅ +L ₂₅₆ +L ₂₅₇ +L ₂₅₈ +L ₂₅₉ +L ₂₆₀ +L ₂₆₁ +L ₂₆₂ +L ₂₆₃ +L ₂₆₄ +L ₂₆₅ +L ₂₆₆ +L ₂₆₇ +L ₂₆₈ +L ₂₆₉ +L ₂₇₀ +L ₂₇₁ +L ₂₇₂ +L ₂₇₃ +L ₂₇₄ +L ₂₇₅ +L ₂₇₆ +L ₂₇₇ +L ₂₇₈ +L ₂₇₉ +L ₂₈₀ +L ₂₈₁ +L ₂₈₂ +L ₂₈₃ +L ₂₈₄ +L ₂₈₅ +L ₂₈₆ +L ₂₈₇ +L ₂₈₈ +L ₂₈₉ +L ₂₉₀ +L ₂₉₁ +L ₂₉₂ +L ₂₉₃ +L ₂₉₄ +L ₂₉₅ +L ₂₉₆ +L ₂₉₇ +L ₂₉₈ +L ₂₉₉ +L ₃₀₀ +L ₃₀₁ +L ₃₀₂ +L ₃₀₃ +L ₃₀₄ +L ₃₀₅ +L ₃₀₆ +L ₃₀₇ +L ₃₀₈ +L ₃₀₉ +L ₃₁₀ +L ₃₁₁ +L ₃₁₂ +L ₃₁₃ +L ₃₁₄ +L ₃₁₅ +L ₃₁₆ +L ₃₁₇ +L ₃₁₈ +L ₃₁₉ +L ₃₂₀ +L ₃₂₁ +L ₃₂₂ +L ₃₂₃ +L ₃₂₄ +L ₃₂₅ +L ₃₂₆ +L ₃₂₇ +L ₃₂₈ +L ₃₂₉ +L ₃₃₀ +L ₃₃₁ +L ₃₃₂ +L ₃₃₃ +L ₃₃₄ +L ₃₃₅ +L ₃₃₆ +L ₃₃₇ +L ₃₃₈ +L ₃₃₉ +L ₃₄₀ +L ₃₄₁ +L ₃₄₂ +L ₃₄₃ +L ₃₄₄ +L ₃₄₅ +L ₃₄₆ +L ₃₄₇ +L ₃₄₈ +L ₃₄₉ +L ₃₅₀ +L ₃₅₁ +L ₃₅₂ +L ₃₅₃ +L ₃₅₄ +L ₃₅₅ +L ₃₅₆ +L ₃₅₇ +L ₃₅₈ +L ₃₅₉ +L ₃₆₀ +L ₃₆₁ +L ₃₆₂ +L ₃₆₃ +L ₃₆₄ +L ₃₆₅ +L ₃₆₆ +L ₃₆₇ +L ₃₆₈ +L ₃₆₉ +L ₃₇₀ +L ₃₇₁ +L ₃₇₂ +L ₃₇₃ +L ₃₇₄ +L ₃₇₅ +L ₃₇₆ +L ₃₇₇ +L ₃₇₈ +L ₃₇₉ +L ₃₈₀ +L ₃₈₁ +L ₃₈₂ +L ₃₈₃ +L ₃₈₄ +L ₃₈₅ +L ₃₈₆ +L ₃₈₇ +L ₃₈₈ +L ₃₈₉ +L ₃₉₀ +L ₃₉₁ +L ₃₉₂ +L ₃₉₃ +L ₃₉₄ +L ₃₉₅ +L ₃₉₆ +L ₃₉₇ +L ₃₉₈ +L ₃₉₉ +L ₄₀₀ +L ₄₀₁ +L ₄₀₂ +L ₄₀₃ +L ₄₀₄ +L ₄₀₅ +L ₄₀₆ +L ₄₀₇ +L ₄₀₈ +L ₄₀₉ +L ₄₁₀ +L ₄₁₁ +L ₄₁₂ +L ₄₁₃ +L ₄₁₄ +L ₄₁₅ +L ₄₁₆ +L ₄₁₇ +L ₄₁₈ +L ₄₁₉ +L ₄₂₀ +L ₄₂₁ +L ₄₂₂ +L ₄₂₃ +L ₄₂₄ +L ₄₂₅ +L ₄₂₆ +L ₄₂₇ +L ₄₂₈ +L ₄₂₉ +L ₄₃₀ +L ₄₃₁ +L ₄₃₂ +L ₄₃₃ +L ₄₃₄ +L ₄₃₅ +L ₄₃₆ +L ₄₃₇ +L ₄₃₈ +L ₄₃₉ +L ₄₄₀ +L ₄₄₁ +L ₄₄₂ +L ₄₄₃ +L ₄₄₄ +L ₄₄₅ +L ₄₄₆ +L ₄₄₇ +L ₄₄₈ +L ₄₄₉ +L ₄₅₀ +L ₄₅₁ +L ₄₅₂ +L ₄₅₃ +L ₄₅₄ +L ₄₅₅ +L ₄₅₆ +L ₄₅₇ +L ₄₅₈ +L ₄₅₉ +L ₄₆₀ +L ₄₆₁ +L ₄₆₂ +L ₄₆₃ +L ₄₆₄ +L ₄₆₅ +L ₄₆₆ +L ₄₆₇ +L ₄₆₈ +L ₄₆₉ +L ₄₇₀ +L ₄₇₁ +L ₄₇₂ +L ₄₇₃ +L ₄₇₄ +L ₄₇₅ +L ₄₇₆ +L ₄₇₇ +L ₄₇₈ +L ₄₇₉ +L ₄₈₀ +L ₄₈₁ +L ₄₈₂ +L ₄₈₃ +L ₄₈₄ +L ₄₈₅ +L ₄₈₆ +L ₄₈₇ +L ₄₈₈ +L ₄₈₉ +L ₄₉₀ +L ₄₉₁ +L ₄₉₂ +L ₄₉₃ +L ₄₉₄ +L ₄₉₅ +L ₄₉₆ +L ₄₉₇ +L ₄₉₈ +L ₄₉₉ +L ₅₀₀ +L ₅₀₁ +L ₅₀₂ +L ₅₀₃ +L ₅₀₄ +L ₅₀₅ +L ₅₀₆ +L ₅₀₇ +L ₅₀₈ +L ₅₀₉ +L ₅₁₀ +L ₅₁₁ +L ₅₁₂ +L ₅₁₃ +L ₅₁₄ +L ₅₁₅ +L ₅₁₆ +L ₅₁₇ +L ₅₁₈ +L ₅₁₉ +L ₅₂₀ +L ₅₂₁ +L ₅₂₂ +L ₅₂₃ +L ₅₂₄ +L ₅₂₅ +L ₅₂₆ +L ₅₂₇ +L ₅₂₈ +L ₅₂₉ +L ₅₃₀ +L ₅₃₁ +L ₅₃₂ +L ₅₃₃ +L ₅₃₄ +L ₅₃₅ +L ₅₃₆ +L ₅₃₇ +L ₅₃₈ +L ₅₃₉ +L ₅₄₀ +L ₅₄₁ +L ₅₄₂ +L ₅₄₃ +L ₅₄₄ +L ₅₄₅ +L ₅₄₆ +L ₅₄₇ +L ₅₄₈ +L ₅₄₉ +L ₅₅₀ +L ₅₅₁ +L ₅₅₂ +L ₅₅₃ +L ₅₅₄ +L ₅₅₅ +L ₅₅₆ +L ₅₅₇ +L ₅₅₈ +L ₅₅₉ +L ₅₆₀ +L ₅₆₁ +L ₅₆₂ +L ₅₆₃ +L ₅₆₄ +L ₅₆₅ +L ₅₆₆ +L ₅₆₇ +L ₅₆₈ +L ₅₆₉ +L ₅₇₀ +L ₅₇₁ +L ₅₇₂ +L ₅₇₃ +L ₅₇₄ +L ₅₇₅ +L ₅₇₆ +L ₅₇₇ +L ₅₇₈ +L ₅₇₉ +L ₅₈₀ +L ₅₈₁ +L ₅₈₂ +L ₅₈₃ +L ₅₈₄ +L ₅₈₅ +L ₅₈₆ +L ₅₈₇ +L ₅₈₈ +L ₅₈₉ +L ₅₉₀ +L ₅₉₁ +L ₅₉₂ +L ₅₉₃ +L ₅₉₄ +L ₅₉₅ +L ₅₉₆ +L ₅₉₇ +L ₅₉₈ +L ₅₉₉ +L ₆₀₀ +L ₆₀₁ +L ₆₀₂ +L ₆₀₃ +L ₆₀₄ +L ₆₀₅ +L ₆₀₆ +L ₆₀₇ +L ₆₀₈ +L ₆₀₉ +L ₆₁₀ +L ₆₁₁ +L ₆₁₂ +L ₆₁₃ +L ₆₁₄ +L ₆₁₅ +L ₆₁₆ +L ₆₁₇ +L ₆₁₈ +L ₆₁₉ +L ₆₂₀ +L ₆₂₁ +L ₆₂₂ +L ₆₂₃ +L ₆₂₄ +L ₆₂₅ +L ₆₂₆ +L ₆₂₇ +L ₆₂₈ +L ₆₂₉ +L ₆₃₀ +L ₆₃₁ +L ₆₃₂ +L ₆₃₃ +L ₆₃₄ +L ₆₃₅ +L ₆₃₆ +L ₆₃₇ +L ₆₃₈ +L ₆₃₉ +L ₆₄₀ +L ₆₄₁ +L ₆₄₂ +L ₆₄₃ +L ₆₄₄ +L ₆₄₅ +L ₆₄₆ +L ₆₄₇ +L ₆₄₈ +L ₆₄₉ +L ₆₅₀ +L ₆₅₁ +L ₆₅₂ +L ₆₅₃ +L ₆₅₄ +L ₆₅₅ +L ₆₅₆ +L ₆₅₇ +L ₆₅₈ +L ₆₅₉ +L ₆₆₀ +L ₆₆₁ +L ₆₆₂ +L ₆₆₃ +L ₆₆₄ +L ₆₆₅ +L ₆₆₆ +L ₆₆₇ +L ₆₆₈ +L ₆₆₉ +L ₆₇₀ +L ₆₇₁ +L ₆₇₂ +L ₆₇₃ +L ₆₇₄ +L ₆₇₅ +L ₆₇₆ +L ₆₇₇ +L ₆₇₈ +L ₆₇₉ +L ₆₈₀ +L ₆₈₁ +L ₆₈₂ +L ₆₈₃ +L ₆₈₄ +L ₆₈₅ +L ₆₈₆ +L ₆₈₇ +L ₆₈₈ +L ₆₈₉ +L ₆₉₀ +L ₆₉₁ +L ₆₉₂ +L ₆₉₃ +L ₆₉₄ +L ₆₉₅ +L ₆₉₆ +L ₆₉₇ +L ₆₉₈ +L ₆₉₉ +L ₇₀₀ +L ₇₀₁ +L ₇₀₂ +L ₇₀₃ +L ₇₀₄ +L ₇₀₅ +L ₇₀₆ +L ₇₀₇ +L ₇₀₈ +L ₇₀₉ +L ₇₁₀ +L ₇₁₁ +L ₇₁₂ +L ₇₁₃ +L ₇₁₄ +L ₇₁₅ +L ₇₁₆ +L ₇₁₇ +L ₇₁₈ +L ₇₁₉ +L ₇₂₀ +L ₇₂₁ +L ₇₂₂ +L ₇₂₃ +L ₇₂₄ +L ₇₂₅ +L ₇₂₆ +L ₇₂₇ +L ₇₂₈ +L ₇₂₉ +L ₇₃₀ +L ₇₃₁ +L ₇₃₂ +L ₇₃₃ +L ₇₃₄ +L ₇₃₅ +L ₇₃₆ +L ₇₃₇ +L ₇₃₈ +L ₇₃₉ +L ₇₄₀ +L ₇₄₁ +L ₇₄₂ +L ₇₄₃ +L ₇₄₄ +L ₇₄₅ +L ₇₄₆ +L ₇₄₇ +L ₇₄₈ +L ₇₄₉ +L ₇₅₀ +L ₇₅₁ +L ₇₅₂ +L ₇₅₃ +L ₇₅₄ +L ₇₅₅ +L ₇₅₆ +L ₇₅₇ +L ₇₅₈ +L ₇₅₉ +L ₇₆₀ +L ₇₆₁ +L ₇₆₂ +L ₇₆₃ +L ₇₆₄ +L ₇₆₅ +L ₇₆₆ +L ₇₆₇ +L ₇₆₈ +L ₇₆₉ +L ₇₇₀ +L ₇₇₁ +L ₇₇₂ +L ₇₇₃ +L ₇₇₄ +L ₇₇₅ +L ₇₇₆ +L ₇₇₇ +L ₇₇₈ +L ₇₇₉ +L ₇₈₀ +L ₇₈₁ +L ₇₈₂ +L ₇₈₃ +L ₇₈₄ +L ₇₈₅ +L ₇₈₆ +L ₇₈₇ +L ₇₈₈ +L ₇₈₉ +L ₇₉₀ +L ₇₉₁ +L ₇₉₂ +L ₇₉₃ +L ₇₉₄ +L ₇₉₅ +L ₇₉₆ +L ₇₉₇ +L ₇₉₈ +L ₇₉₉ +L ₈₀₀ +L ₈₀₁ +L ₈₀₂ +L ₈₀₃ +L ₈₀₄ +L ₈₀₅ +L ₈₀₆ +L ₈₀₇ +L ₈₀₈ +L ₈₀₉ +L ₈₁₀ +L ₈₁₁ +L ₈₁₂ +L ₈₁₃ +L ₈₁₄ +L ₈₁₅ +L ₈₁₆ +L ₈₁₇ +L ₈₁₈ +L ₈₁₉ +L ₈₂₀ +L ₈₂₁ +L ₈₂₂ +L ₈₂₃ +L ₈₂₄ +L ₈₂₅ +L ₈₂₆ +L ₈₂₇ +L ₈₂₈ +L ₈₂₉ +L ₈₃₀ +L ₈₃₁ +L ₈₃₂ +L ₈₃₃ +L ₈₃₄ +L ₈₃₅ +L ₈₃₆ +L ₈₃₇ +L ₈₃₈ +L ₈₃₉ +L ₈₄₀ +L ₈₄₁ +L ₈₄₂ +L ₈₄₃ +L ₈₄₄ +L ₈₄₅ +L ₈₄₆ +L ₈										

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	45			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
						BS8110 [PC				
						BS8110 [F ▼				
Fourth Shear Perimeter		@3.75d _{cen}	N/A	to	@2.25d _{cen}	N/A	mm	cl.3.7.7.6		
Eff. depth to A _s , d _{ps,5} = h-X _c (SLS/ULS)-e _{var} (x=@shear perimeter)						N/A	N/A	mm	Goal Seek	
Eff. depth to centroid of A _s and A _{s,prov,h} , d _{cen,5}							N/A	mm	cl.4.3.8.1	
Eff. depth to max of A _s and A _{s,prov,h} , d _{max,5}							N/A	mm	3.8.1, cl.4.3	
Shear force at fourth shear perimeter, V ₅ = (V _t -V _{reduced,5})						N/A	N/A	kN		
						Rectangular	Circular			
IC:	(l _{h,b} + 7.5d _{cen,5}). (l _{h,h} + 7.5d _{cen,5})		(l _{h,D} + 7.5d _{cen,5}) ²		N/A	N/A	m ²			
EC:	(l _{h,b} + 3.75d _{cen,5}). (l _{h,h} + 7.5d _{cen,5})		(l _{h,D} + 3.75d _{cen,5}). (l _{h,h} + 3.75d _{cen,5})		N/A	N/A	m ²			
CC:	(l _{h,b} + 3.75d _{cen,5}). (l _{h,h} + 3.75d _{cen,5})		(l _{h,D} + 3.75d _{cen,5}) ²		N/A	N/A	m ²			
Eff. shear force, V _{eff,5} = (1.15 int., 1.40 edge, 1.50 corner column) . V ₅						N/A	kN	cl.3.7.6		
Column fourth perimeter, u ₅ ≤ {2L+2L ₁ +L ₂ +L ₃ +L ₄ +L ₅ +L ₆ +L ₇ +L ₈ +L ₉ +L ₁₀ +L ₁₁ +L ₁₂ +L ₁₃ +L ₁₄ +L ₁₅ +L ₁₆ +L ₁₇ +L ₁₈ +L ₁₉ +L ₂₀ +L ₂₁ +L ₂₂ +L ₂₃ +L ₂₄ +L ₂₅ +L ₂₆ +L ₂₇ +L ₂₈ +L ₂₉ +L ₃₀ +L ₃₁ +L ₃₂ +L ₃₃ +L ₃₄ +L ₃₅ +L ₃₆ +L ₃₇ +L ₃₈ +L ₃₉ +L ₄₀ +L ₄₁ +L ₄₂ +L ₄₃ +L ₄₄ +L ₄₅ +L ₄₆ +L ₄₇ +L ₄₈ +L ₄₉ +L ₅₀ +L ₅₁ +L ₅₂ +L ₅₃ +L ₅₄ +L ₅₅ +L ₅₆ +L ₅₇ +L ₅₈ +L ₅₉ +L ₆₀ +L ₆₁ +L ₆₂ +L ₆₃ +L ₆₄ +L ₆₅ +L ₆₆ +L ₆₇ +L ₆₈ +L ₆₉ +L ₇₀ +L ₇₁ +L ₇₂ +L ₇₃ +L ₇₄ +L ₇₅ +L ₇₆ +L ₇₇ +L ₇₈ +L ₇₉ +L ₈₀ +L ₈₁ +L ₈₂ +L ₈₃ +L ₈₄ +L ₈₅ +L ₈₆ +L ₈₇ +L ₈₈ +L ₈₉ +L ₉₀ +L ₉₁ +L ₉₂ +L ₉₃ +L ₉₄ +L ₉₅ +L ₉₆ +L ₉₇ +L ₉₈ +L ₉₉ +L ₁₀₀ +L ₁₀₁ +L ₁₀₂ +L ₁₀₃ +L ₁₀₄ +L ₁₀₅ +L ₁₀₆ +L ₁₀₇ +L ₁₀₈ +L ₁₀₉ +L ₁₁₀ +L ₁₁₁ +L ₁₁₂ +L ₁₁₃ +L ₁₁₄ +L ₁₁₅ +L ₁₁₆ +L ₁₁₇ +L ₁₁₈ +L ₁₁₉ +L ₁₂₀ +L ₁₂₁ +L ₁₂₂ +L ₁₂₃ +L ₁₂₄ +L ₁₂₅ +L ₁₂₆ +L ₁₂₇ +L ₁₂₈ +L ₁₂₉ +L ₁₃₀ +L ₁₃₁ +L ₁₃₂ +L ₁₃₃ +L ₁₃₄ +L ₁₃₅ +L ₁₃₆ +L ₁₃₇ +L ₁₃₈ +L ₁₃₉ +L ₁₄₀ +L ₁₄₁ +L ₁₄₂ +L ₁₄₃ +L ₁₄₄ +L ₁₄₅ +L ₁₄₆ +L ₁₄₇ +L ₁₄₈ +L ₁₄₉ +L ₁₅₀ +L ₁₅₁ +L ₁₅₂ +L ₁₅₃ +L ₁₅₄ +L ₁₅₅ +L ₁₅₆ +L ₁₅₇ +L ₁₅₈ +L ₁₅₉ +L ₁₆₀ +L ₁₆₁ +L ₁₆₂ +L ₁₆₃ +L ₁₆₄ +L ₁₆₅ +L ₁₆₆ +L ₁₆₇ +L ₁₆₈ +L ₁₆₉ +L ₁₇₀ +L ₁₇₁ +L ₁₇₂ +L ₁₇₃ +L ₁₇₄ +L ₁₇₅ +L ₁₇₆ +L ₁₇₇ +L ₁₇₈ +L ₁₇₉ +L ₁₈₀ +L ₁₈₁ +L ₁₈₂ +L ₁₈₃ +L ₁₈₄ +L ₁₈₅ +L ₁₈₆ +L ₁₈₇ +L ₁₈₈ +L ₁₈₉ +L ₁₉₀ +L ₁₉₁ +L ₁₉₂ +L ₁₉₃ +L ₁₉₄ +L ₁₉₅ +L ₁₉₆ +L ₁₉₇ +L ₁₉₈ +L ₁₉₉ +L ₂₀₀ +L ₂₀₁ +L ₂₀₂ +L ₂₀₃ +L ₂₀₄ +L ₂₀₅ +L ₂₀₆ +L ₂₀₇ +L ₂₀₈ +L ₂₀₉ +L ₂₁₀ +L ₂₁₁ +L ₂₁₂ +L ₂₁₃ +L ₂₁₄ +L ₂₁₅ +L ₂₁₆ +L ₂₁₇ +L ₂₁₈ +L ₂₁₉ +L ₂₂₀ +L ₂₂₁ +L ₂₂₂ +L ₂₂₃ +L ₂₂₄ +L ₂₂₅ +L ₂₂₆ +L ₂₂₇ +L ₂₂₈ +L ₂₂₉ +L ₂₃₀ +L ₂₃₁ +L ₂₃₂ +L ₂₃₃ +L ₂₃₄ +L ₂₃₅ +L ₂₃₆ +L ₂₃₇ +L ₂₃₈ +L ₂₃₉ +L ₂₄₀ +L ₂₄₁ +L ₂₄₂ +L ₂₄₃ +L ₂₄₄ +L ₂₄₅ +L ₂₄₆ +L ₂₄₇ +L ₂₄₈ +L ₂₄₉ +L ₂₅₀ +L ₂₅₁ +L ₂₅₂ +L ₂₅₃ +L ₂₅₄ +L ₂₅₅ +L ₂₅₆ +L ₂₅₇ +L ₂₅₈ +L ₂₅₉ +L ₂₆₀ +L ₂₆₁ +L ₂₆₂ +L ₂₆₃ +L ₂₆₄ +L ₂₆₅ +L ₂₆₆ +L ₂₆₇ +L ₂₆₈ +L ₂₆₉ +L ₂₇₀ +L ₂₇₁ +L ₂₇₂ +L ₂₇₃ +L ₂₇₄ +L ₂₇₅ +L ₂₇₆ +L ₂₇₇ +L ₂₇₈ +L ₂₇₉ +L ₂₈₀ +L ₂₈₁ +L ₂₈₂ +L ₂₈₃ +L ₂₈₄ +L ₂₈₅ +L ₂₈₆ +L ₂₈₇ +L ₂₈₈ +L ₂₈₉ +L ₂₉₀ +L ₂₉₁ +L ₂₉₂ +L ₂₉₃ +L ₂₉₄ +L ₂₉₅ +L ₂₉₆ +L ₂₉₇ +L ₂₉₈ +L ₂₉₉ +L ₃₀₀ +L ₃₀₁ +L ₃₀₂ +L ₃₀₃ +L ₃₀₄ +L ₃₀₅ +L ₃₀₆ +L ₃₀₇ +L ₃₀₈ +L ₃₀₉ +L ₃₁₀ +L ₃₁₁ +L ₃₁₂ +L ₃₁₃ +L ₃₁₄ +L ₃₁₅ +L ₃₁₆ +L ₃₁₇ +L ₃₁₈ +L ₃₁₉ +L ₃₂₀ +L ₃₂₁ +L ₃₂₂ +L ₃₂₃ +L ₃₂₄ +L ₃₂₅ +L ₃₂₆ +L ₃₂₇ +L ₃₂₈ +L ₃₂₉ +L ₃₃₀ +L ₃₃₁ +L ₃₃₂ +L ₃₃₃ +L ₃₃₄ +L ₃₃₅ +L ₃₃₆ +L ₃₃₇ +L ₃₃₈ +L ₃₃₉ +L ₃₄₀ +L ₃₄₁ +L ₃₄₂ +L ₃₄₃ +L ₃₄₄ +L ₃₄₅ +L ₃₄₆ +L ₃₄₇ +L ₃₄₈ +L ₃₄₉ +L ₃₅₀ +L ₃₅₁ +L ₃₅₂ +L ₃₅₃ +L ₃₅₄ +L ₃₅₅ +L ₃₅₆ +L ₃₅₇ +L ₃₅₈ +L ₃₅₉ +L ₃₆₀ +L ₃₆₁ +L ₃₆₂ +L ₃₆₃ +L ₃₆₄ +L ₃₆₅ +L ₃₆₆ +L ₃₆₇ +L ₃₆₈ +L ₃₆₉ +L ₃₇₀ +L ₃₇₁ +L ₃₇₂ +L ₃₇₃ +L ₃₇₄ +L ₃₇₅ +L ₃₇₆ +L ₃₇₇ +L ₃₇₈ +L ₃₇₉ +L ₃₈₀ +L ₃₈₁ +L ₃₈₂ +L ₃₈₃ +L ₃₈₄ +L ₃₈₅ +L ₃₈₆ +L ₃₈₇ +L ₃₈₈ +L ₃₈₉ +L ₃₉₀ +L ₃₉₁ +L ₃₉₂ +L ₃₉₃ +L ₃₉₄ +L ₃₉₅ +L ₃₉₆ +L ₃₉₇ +L ₃₉₈ +L ₃₉₉ +L ₄₀₀ +L ₄₀₁ +L ₄₀₂ +L ₄₀₃ +L ₄₀₄ +L ₄₀₅ +L ₄₀₆ +L ₄₀₇ +L ₄₀₈ +L ₄₀₉ +L ₄₁₀ +L ₄₁₁ +L ₄₁₂ +L ₄₁₃ +L ₄₁₄ +L ₄₁₅ +L ₄₁₆ +L ₄₁₇ +L ₄₁₈ +L ₄₁₉ +L ₄₂₀ +L ₄₂₁ +L ₄₂₂ +L ₄₂₃ +L ₄₂₄ +L ₄₂₅ +L ₄₂₆ +L ₄₂₇ +L ₄₂₈ +L ₄₂₉ +L ₄₃₀ +L ₄₃₁ +L ₄₃₂ +L ₄₃₃ +L ₄₃₄ +L ₄₃₅ +L ₄₃₆ +L ₄₃₇ +L ₄₃₈ +L ₄₃₉ +L ₄₄₀ +L ₄₄₁ +L ₄₄₂ +L ₄₄₃ +L ₄₄₄ +L ₄₄₅ +L ₄₄₆ +L ₄₄₇ +L ₄₄₈ +L ₄₄₉ +L ₄₅₀ +L ₄₅₁ +L ₄₅₂ +L ₄₅₃ +L ₄₅₄ +L ₄₅₅ +L ₄₅₆ +L ₄₅₇ +L ₄₅₈ +L ₄₅₉ +L ₄₆₀ +L ₄₆₁ +L ₄₆₂ +L ₄₆₃ +L ₄₆₄ +L ₄₆₅ +L ₄₆₆ +L ₄₆₇ +L ₄₆₈ +L ₄₆₉ +L ₄₇₀ +L ₄₇₁ +L ₄₇₂ +L ₄₇₃ +L ₄₇₄ +L ₄₇₅ +L ₄₇₆ +L ₄₇₇ +L ₄₇₈ +L ₄₇₉ +L ₄₈₀ +L ₄₈₁ +L ₄₈₂ +L ₄₈₃ +L ₄₈₄ +L ₄₈₅ +L ₄₈₆ +L ₄₈₇ +L ₄₈₈ +L ₄₈₉ +L ₄₉₀ +L ₄₉₁ +L ₄₉₂ +L ₄₉₃ +L ₄₉₄ +L ₄₉₅ +L ₄₉₆ +L ₄₉₇ +L ₄₉₈ +L ₄₉₉ +L ₅₀₀ +L ₅₀₁ +L ₅₀₂ +L ₅₀₃ +L ₅₀₄ +L ₅₀₅ +L ₅₀₆ +L ₅₀₇ +L ₅₀₈ +L ₅₀₉ +L ₅₁₀ +L ₅₁₁ +L ₅₁₂ +L ₅₁₃ +L ₅₁₄ +L ₅₁₅ +L ₅₁₆ +L ₅₁₇ +L ₅₁₈ +L ₅₁₉ +L ₅₂₀ +L ₅₂₁ +L ₅₂₂ +L ₅₂₃ +L ₅₂₄ +L ₅₂₅ +L ₅₂₆ +L ₅₂₇ +L ₅₂₈ +L ₅₂₉ +L ₅₃₀ +L ₅₃₁ +L ₅₃₂ +L ₅₃₃ +L ₅₃₄ +L ₅₃₅ +L ₅₃₆ +L ₅₃₇ +L ₅₃₈ +L ₅₃₉ +L ₅₄₀ +L ₅₄₁ +L ₅₄₂ +L ₅₄₃ +L ₅₄₄ +L ₅₄₅ +L ₅₄₆ +L ₅₄₇ +L ₅₄₈ +L ₅₄₉ +L ₅₅₀ +L ₅₅₁ +L ₅₅₂ +L ₅₅₃ +L ₅₅₄ +L ₅₅₅ +L ₅₅₆ +L ₅₅₇ +L ₅₅₈ +L ₅₅₉ +L ₅₆₀ +L ₅₆₁ +L ₅₆₂ +L ₅₆₃ +L ₅₆₄ +L ₅₆₅ +L ₅₆₆ +L ₅₆₇ +L ₅₆₈ +L ₅₆₉ +L ₅₇₀ +L ₅₇₁ +L ₅₇₂ +L ₅₇₃ +L ₅₇₄ +L ₅₇₅ +L ₅₇₆ +L ₅₇₇ +L ₅₇₈ +L ₅₇₉ +L ₅₈₀ +L ₅₈₁ +L ₅₈₂ +L ₅₈₃ +L ₅₈₄ +L ₅₈₅ +L ₅₈₆ +L ₅₈₇ +L ₅₈₈ +L ₅₈₉ +L ₅₉₀ +L ₅₉₁ +L ₅₉₂ +L ₅₉₃ +L ₅₉₄ +L ₅₉₅ +L ₅₉₆ +L ₅₉₇ +L ₅₉₈ +L ₅₉₉ +L ₆₀₀ +L ₆₀₁ +L ₆₀₂ +L ₆₀₃ +L ₆₀₄ +L ₆₀₅ +L ₆₀₆ +L ₆₀₇ +L ₆₀₈ +L ₆₀₉ +L ₆₁₀ +L ₆₁₁ +L ₆₁₂ +L ₆₁₃ +L ₆₁₄ +L ₆₁₅ +L ₆₁₆ +L ₆₁₇ +L ₆₁₈ +L ₆₁₉ +L ₆₂₀ +L ₆₂₁ +L ₆₂₂ +L ₆₂₃ +L ₆₂₄ +L ₆₂₅ +L ₆₂₆ +L ₆₂₇ +L ₆₂₈ +L ₆₂₉ +L ₆₃₀ +L ₆₃₁ +L ₆₃₂ +L ₆₃₃ +L ₆₃₄ +L ₆₃₅ +L ₆₃₆ +L ₆₃₇ +L ₆₃₈ +L ₆₃₉ +L ₆₄₀ +L ₆₄₁ +L ₆₄₂ +L ₆₄₃ +L ₆₄₄ +L ₆₄₅ +L ₆₄₆ +L ₆₄₇ +L ₆₄₈ +L ₆₄₉ +L ₆₅₀ +L ₆₅₁ +L ₆₅₂ +L ₆₅₃ +L ₆₅₄ +L ₆₅₅ +L ₆₅₆ +L ₆₅₇ +L ₆₅₈ +L ₆₅₉ +L ₆₆₀ +L ₆₆₁ +L ₆₆₂ +L ₆₆₃ +L ₆₆₄ +L ₆₆₅ +L ₆₆₆ +L ₆₆₇ +L ₆₆₈ +L ₆₆₉ +L ₆₇₀ +L ₆₇₁ +L ₆₇₂ +L ₆₇₃ +L ₆₇₄ +L ₆₇₅ +L ₆₇₆ +L ₆₇₇ +L ₆₇₈ +L ₆₇₉ +L ₆₈₀ +L ₆₈₁ +L ₆₈₂ +L ₆₈₃ +L ₆₈₄ +L ₆₈₅ +L ₆₈₆ +L ₆₈₇ +L ₆₈₈ +L ₆₈₉ +L ₆₉₀ +L ₆₉₁ +L ₆₉₂ +L ₆₉₃ +L ₆₉₄ +L ₆₉₅ +L ₆₉₆ +L ₆₉₇ +L ₆₉₈ +L ₆₉₉ +L ₇₀₀ +L ₇₀₁ +L ₇₀₂ +L ₇₀₃ +L ₇₀₄ +L ₇₀₅ +L ₇₀₆ +L ₇₀₇ +L ₇₀₈ +L ₇₀₉ +L ₇₁₀ +L ₇₁₁ +L ₇₁₂ +L ₇₁₃ +L ₇₁₄ +L ₇₁₅ +L ₇₁₆ +L ₇₁₇ +L ₇₁₈ +L ₇₁₉ +L ₇₂₀ +L ₇₂₁ +L ₇₂₂ +L ₇₂₃ +L ₇₂₄ +L ₇₂₅ +L ₇₂₆ +L ₇₂₇ +L ₇₂₈ +L ₇₂₉ +L ₇₃₀ +L ₇₃₁ +L ₇₃₂ +L ₇₃₃ +L ₇₃₄ +L ₇₃₅ +L ₇₃₆ +L ₇₃₇ +L ₇₃₈ +L ₇₃₉ +L ₇₄₀ +L ₇₄₁ +L ₇₄₂ +L ₇₄₃ +L ₇₄₄ +L ₇₄₅ +L ₇₄₆ +L ₇₄₇ +L ₇₄₈ +L ₇₄₉ +L ₇₅₀ +L ₇₅₁ +L ₇₅₂ +L ₇₅₃ +L ₇₅₄ +L ₇₅₅ +L ₇₅₆ +L ₇₅₇ +L ₇₅₈ +L ₇₅₉ +L ₇₆₀ +L ₇₆₁ +L ₇₆₂ +L ₇₆₃ +L ₇₆₄ +L ₇₆₅ +L ₇₆₆ +L ₇₆₇ +L ₇₆₈ +L ₇₆₉ +L ₇₇₀ +L ₇₇₁ +L ₇₇₂ +L ₇₇₃ +L ₇₇₄ +L ₇₇₅ +L ₇₇₆ +L ₇₇₇ +L ₇₇₈ +L ₇₇₉ +L ₇₈₀ +L ₇₈₁ +L ₇₈₂ +L ₇₈₃ +L ₇₈₄ +L ₇₈₅ +L ₇₈₆ +L ₇₈₇ +L ₇₈₈ +L ₇₈₉ +L ₇₉₀ +L ₇₉₁ +L ₇₉₂ +L ₇₉₃ +L ₇₉₄ +L ₇₉₅ +L ₇₉₆ +L ₇₉₇ +L ₇₉₈ +L ₇₉₉ +L ₈₀₀ +L ₈₀₁ +L ₈₀₂ +L ₈₀₃ +L ₈₀₄ +L ₈₀₅ +L ₈₀₆ +L ₈₀₇ +L ₈₀₈ +L ₈₀₉ +L ₈₁₀ +L ₈₁₁ +L ₈₁₂ +L ₈₁₃ +L ₈₁₄ +L ₈₁₅ +L ₈₁₆ +L ₈₁₇ +L ₈₁₈ +L ₈₁₉ +L ₈₂₀ +L ₈₂₁ +L ₈₂₂ +L ₈₂₃ +L ₈₂₄ +L ₈₂₅ +L ₈₂₆ +L ₈₂₇ +L ₈₂₈ +L ₈₂₉ +L ₈₃₀ +L ₈₃₁ +L ₈₃₂ +L ₈₃₃ +L ₈₃₄ +L ₈₃₅ +L ₈₃₆ +L ₈₃₇ +L ₈₃₈ +L ₈₃₉ +L ₈₄₀ +L ₈₄₁ +L ₈₄₂ +L ₈₄₃ +L ₈₄₄ +L ₈₄₅ +L ₈₄₆ +L ₈₄₇ +L ₈₄₈ +L ₈₄₉ +L ₈₅₀ +L ₈₅₁ +L ₈₅₂ +L ₈₅₃ +L ₈₅₄ +L ₈₅₅ +L<										

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers			Job No. jXXX	Sheet No. 46	Rev.
					Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab				Drg. Ref.		
Member Design - PC Beam and Slab					Made by	XX	Date 24/10/2026 Chd.
							EC2
Longitudinal Shear Between Web and Flange Rectangular or Flanged Beam (EC2)							
Note that this check is performed for both rectangular and flanged section designs, although theoretically only applicable in the latter case;							
Longitudinal shear stress, $K_S \cdot v_{Ed}$					2.09	N/mm ²	
Longitudinal shear stress, $v_{Ed} = \Delta F_d / (h_f \cdot \Delta x)$					1.57	N/mm ²	cl.6.2.4
8.8	Change of normal force in flange half over Δx , $\Delta F_d = K_B \cdot (M_{ULS,E/E+1})$					872	kN
Note conservatively factor, $K_B = 0.5(b_{eff} - b_w)/b_{eff}$ employed even if neutral axis within web;							
Lever arm, z					0.643	m	BC2
Neutral axis, x		$x =$	$(d - z)/0.45$, for $f_{cu} \leq 60$ N/mm ²		Note d here refers to d_{cen} ;	cl.3.4.4.4	
			$(d - z)/0.40$, for $60 < f_{cu} \leq 75$ N/mm ²			cl.3.4.4.4	
			$(d - z)/0.36$, for $75 < f_{cu} \leq 105$ N/mm ²			cl.3.4.4.4	
Thickness of the flange at the junctions, h_f					200	mm	
Length under consideration, Δx					2778	mm	
Note the maximum value that may be assumed for Δx is half the distance between the section where the moment is 0 and the section where the moment is maximum. However, since ΔF_d is also calculated over Δx based on a variation of moment of $\sim M_{ULS}/2 - 0$ 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 $M_{ULS} - 0$ and factored by K_S .							
Shear stress distribution factor, K_S					1.33		
For UDLs, K_S may be taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;							
Effective width, $b_{eff} = \text{MIN}(b_w + \text{function (span, section, structure)})$					1901	mm	
Note for rectangular sections, b_{eff} equivalent to that of T-sections assumed;							
Width (rectangular) or web width (flanged), b_w					500	mm	
Longitudinal shear stress limit to prevent crushing, $v f_{cd} \sin \theta_f \cos \theta_f$					4.31	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
13	Strength reduction factor for concrete cracked in shear, v					0.533	
13	$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$						cl.6.2.2
Longitudinal shear stress limit to prevent crushing utilisation, $(K_S \cdot v_{Ed}) / (v f_{cd} \sin \theta_f)$					48%		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	
13	$f_{ctd} = \alpha_{ct} f_{ctk,0.05} / \gamma_c$ 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, $(K_S \cdot v_{Ed}) / (v f_{ctd} \sin \theta_f)$					404%		NOT OK
Required design transverse reinforcement per unit length, $A_{sf}/s_f >$					603	mm ² /m	
$(A_{sf} f_{yd}/s_f) \geq v_{Ed} \cdot h_f / \cot \theta_f$							
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;							
Design yield strength of reinforcement, $f_{yd} = f_y / \gamma_s$, $\gamma_s = 1.15$					400	N/mm ²	cl.2.4.2.4
Thickness of the flange at the junctions, h_f					200	mm	
Angle, θ_f					30.0	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, $(A_{sf}/s_f)/A_e$					77%		OK

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Member Design - PC Beam and Slab			Made by	XX	Date 24/10/2026 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, $V_1 = K_S \cdot \Delta F_d / \Delta x$			418 kN/m																														
Change of normal force in flange half over Δx , $\Delta F_d = K_B \cdot (M_{ULS,E/E} + $			872 kN																														
Note conservatively factor, $K_B = 0.5(b_{eff} - b_w)/b_{eff}$ employed even if neutral axis within web;																																	
Lever arm, z			0.643 m		BC2																												
Neutral axis, x	$x =$	$(d - z)/0.45$, for $f_{cu} \leq 60 \text{ N/mm}^2$	Note d here refers to d_{cen} ;		cl.3.4.4.4																												
		$(d - z)/0.40$, for $60 < f_{cu} \leq 75 \text{ N/mm}^2$			cl.3.4.4.4																												
		$(d - z)/0.36$, for $75 < f_{cu} \leq 105 \text{ N/mm}^2$			cl.3.4.4.4																												
Thickness of the flange at the junctions, h_f			200 mm																														
Length under consideration, Δx			2778 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, $b_{eff} = \text{MIN}(b_w + \text{function (span, section, structure)})$			1901 mm																														
Note for rectangular sections, b_{eff} equivalent to that of T-sections assumed;																																	
Width (rectangular) or web width (flanged), b_w			500 mm																														
Longitudinal shear force limit per unit length, $V_{1,limit}$			503 kN/m																														
V_1 should not exceed the lesser of the following: a) $k_1 f_{cu} L_s$ b) $v_1 L_s + 0.7 A_e f_y$			(a)	1050 kN/m	cl.7.4.2.3																												
			(b)	503 kN/m	cl.7.4.2.3																												
Table 31 — Ultimate longitudinal shear stress, v_1, and values of k_1 for composite members <table border="1"> <thead> <tr> <th rowspan="2">Type of shear plane</th> <th colspan="4">Longitudinal shear stress for concrete grade</th> <th rowspan="2">k_1</th> </tr> <tr> <th>20 N/mm²</th> <th>25 N/mm²</th> <th>30 N/mm²</th> <th>40 or more N/mm²</th> </tr> </thead> <tbody> <tr> <td>Monolithic construction</td> <td>0.90</td> <td>0.90</td> <td>1.25</td> <td>1.25</td> <td>0.15</td> </tr> <tr> <td>Surface type 1</td> <td>0.50</td> <td>0.63</td> <td>0.75</td> <td>0.80</td> <td>0.15</td> </tr> <tr> <td>Surface type 2</td> <td>0.30</td> <td>0.38</td> <td>0.45</td> <td>0.50</td> <td>0.09</td> </tr> </tbody> </table> NOTE For construction with lightweight aggregate concrete, the values given in this table should be reduced by 25 %.						Type of shear plane	Longitudinal shear stress for concrete grade				k_1	20 N/mm ²	25 N/mm ²	30 N/mm ²	40 or more 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
Type of shear plane	Longitudinal shear stress for concrete grade				k_1																												
	20 N/mm ²	25 N/mm ²	30 N/mm ²	40 or more 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																												
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			460 N/mm ²																														
Longitudinal shear force limit per unit length utilisation, $V_1/V_{1,limit}$			83%		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, $0.15\%L$			38%		OK																												

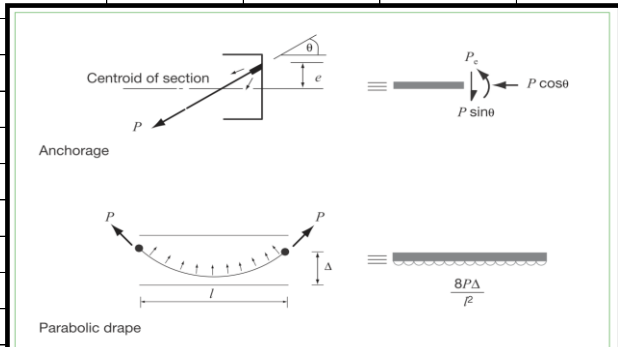
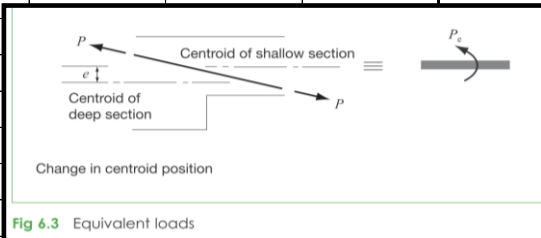
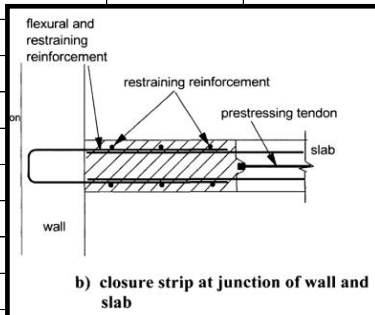
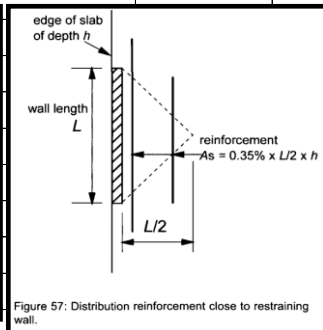
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Job Title	Member Design - Prestressed Concrete Beam and Slab			Drg. Ref.		
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				EC2		
Longitudinal Shear Within Web Rectangular or Flanged Beam (EC2)						
Longitudinal shear stress,		$V_{Edi} = \beta V_{Ed} / (z b_i)$			2.02	N/mm ²
Ratio, $\beta = 1.0$					1.0	
Transverse shear force, $V_{Ed} = \text{ABS} (V_{ULS,E/E} + V_{SLS,S/E}) / 2$					651	kN
Lever arm, z					0.643	m
Neutral axis, x	$x =$	$(d - z)/0.45, \text{ for } f_{cu} \leq 60 \text{ N/mm}^2$		Note d here refers to d_{cenr}		
		$(d - z)/0.40, \text{ for } 60 < f_{cu} \leq 75 \text{ N/mm}^2$				
		$(d - z)/0.36, \text{ for } 75 < f_{cu} \leq 105 \text{ N/mm}^2$				
Width of the interface, $b_i = b_w$					500	mm
Longitudinal shear stress limit, V_{Rdi}					2.28	N/mm ²
$V_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) \leq 0,5 v f_{cd}$						
Note $c.f_{ctd} = 0.00$ if σ_n is negative (tension);						
Roughness coefficient, c		Rough		▼	0.400	
Roughness coefficient, μ		Rough		▼	0.7	
Very smooth: a surface cast against steel, plastic or specially prepared wooden moulds: $c = 0,025$ to $0,10$ and $\mu = 0,5$ Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: $c = 0,20$ and $\mu = 0,6$ Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: $c = 0,40$ and $\mu = 0,7$ (ACI) Indented: a surface with indentations complying with Figure 6.9: $c = 0,50$ and $\mu = 0,9$						
Design tensile strength, f_{ctd}					1.29	N/mm ²
$f_{ctd} = \alpha_{ct} f_{ctk,0.05} / \gamma_C$		with $\alpha_{ct}=1.0, \gamma_C=1.5$				
$f_{ctk,0.05} = 0,7 \times f_{ctm}$					1.94	N/mm ²
$f_{ctm} = 0,30 \times f_{ck}^{(2/3)} \leq C50/60$		$f_{ctm} = 2,12 \cdot \ln(1 + (f_{cm}/10)) > C50/60$			2.77	N/mm ²
$f_{cm} = f_{ck} + 8 \text{ (MPa)}$					36	N/mm ²
Characteristic cylinder strength of concrete, f_{ck}					28	N/mm ²
Characteristic cube strength of concrete, f_{cu}					35	N/mm ²
Normal stress across longitudinal shear interface, $\sigma_n = 0$					0.00	N/mm ²
Reinforcement ratio, $\rho = A_s / A_i$					0.006	
Area of reinforcement, $A_s = A_{sv,prov}/S$					3142	mm ² /m
Note that the area of reinforcement crossing the shear interface may include ordinary shear reinforcement with adequate anchorage at both sides of the interface;						
Area of the joint, $A_i = 1000.b_i$					500000	mm ² /m
Design yield strength of reinforcement, $f_{yd} = f_{yv} / \gamma_S$		$\gamma_S=1.15$			400	N/mm ²
Angle of reinforcement, $\alpha = 90.0^\circ$					90.0	degrees
Design compressive strength, f_{cd}					19	N/mm ²
$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C$		with $\alpha_{cc}=1.0, \gamma_C=1.5$				
Strength reduction factor for concrete cracked in shear, v					0.533	
$v = 0,6 \left[1 - \frac{f_{ck}}{250} \right]$						
Longitudinal shear stress limit utilisation, V_{Edi}/V_{Rdi}					89%	OK

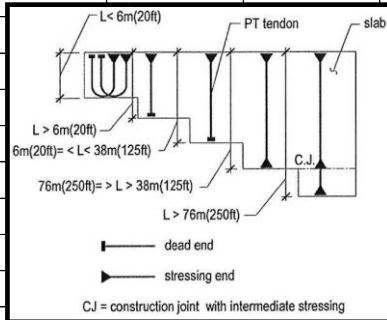
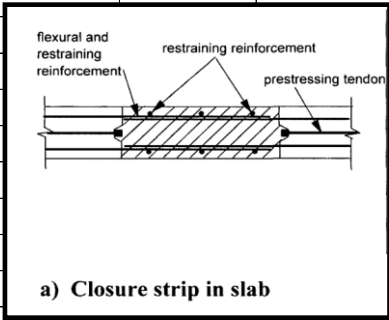
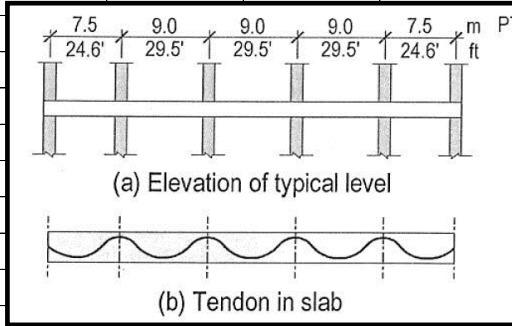
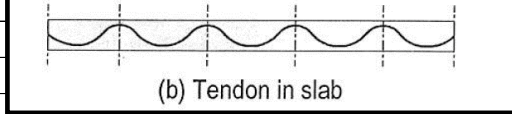
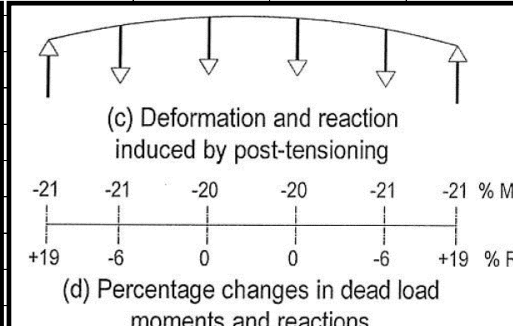
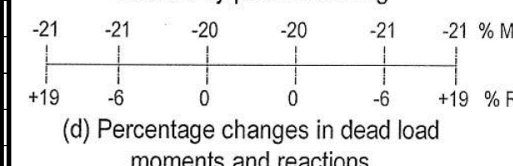
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					Chd. BS8110																																		
Longitudinal Shear Within Web Rectangular or Flanged Beam (BS8110)																																							
Longitudinal shear stress, $v_h = K_s \cdot \Delta F_c / (b_w \cdot \Delta x)$				2.27	N/mm ² cl.5.4.7.2																																		
Change of total compression force over Δx , $\Delta F_c = (M_{ULS,E/E} + M_{SLS,S/E}$				2367	kN cl.5.4.7.1																																		
Lever arm, z				0.643	m																																		
Neutral axis, x	$x =$	$(d - z)/0.45$, for $f_{cu} \leq 60$ N/mm ²	Note d here refers to d_{cen}		cl.3.4.4.4																																		
		$(d - z)/0.40$, for $60 < f_{cu} \leq 75$ N/mm ²			cl.3.4.4.4																																		
		$(d - z)/0.36$, for $75 < f_{cu} \leq 105$ N/mm ²			cl.3.4.4.4																																		
Length under consideration, Δx				2778	mm																																		
Note Δx is the beam length between the point of maximum design moment and the point of zero moment;					cl.5.4.7.2																																		
Shear stress distribution factor, K_s				1.33																																			
The average design shear stress should then be distributed in proportion to the vertical design shear force diagram to give the horizontal shear stress at any point along the length of the member. For UDLs, K_s maybe taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;					cl.5.4.7.2																																		
Width (rectangular) or web width (flanged), b_w				500	mm																																		
Longitudinal shear stress limit for no nominal / design vertical reinforcement,				2.35	N/mm ²																																		
Surface type		Washed to remove laitance etc			T.5.5																																		
<table border="1"> <caption>Table 5.5 — Design ultimate horizontal shear stresses at interface</caption> <thead> <tr> <th rowspan="2">Precast unit</th> <th rowspan="2">Surface type</th> <th colspan="3">Grade of in-situ concrete</th> </tr> <tr> <th>25 N/mm²</th> <th>30 N/mm²</th> <th>40 and over N/mm²</th> </tr> </thead> <tbody> <tr> <td rowspan="3">Without links</td> <td>As-cast or as-extruded</td> <td>0.4</td> <td>0.55</td> <td>0.65</td> </tr> <tr> <td>Brushed, screeded or rough-tamped</td> <td>0.6</td> <td>0.65</td> <td>0.75</td> </tr> <tr> <td>Washed to remove laitance or treated with retarder and cleaned</td> <td>0.7</td> <td>0.75</td> <td>0.80</td> </tr> <tr> <td rowspan="3">With nominal links projecting into in-situ concrete</td> <td>As-cast or as-extruded</td> <td>1.2</td> <td>1.8</td> <td>2.0</td> </tr> <tr> <td>Brushed, screeded or rough-tamped</td> <td>1.8</td> <td>2.0</td> <td>2.2</td> </tr> <tr> <td>Washed to remove laitance or treated with retarder and cleaned</td> <td>2.1</td> <td>2.2</td> <td>2.5</td> </tr> </tbody> </table> NOTE 1 The description "as-cast" covers those cases where the concrete is placed and vibrated leaving a rough finish. The surface is rougher than would be required for finishes to be applied directly without a further finishing screed but not as rough as would be obtained if tamping, brushing or other artificial roughening had taken place. NOTE 2 The description "as-extruded" covers those cases in which an open-textured surface is produced direct from an extruding machine. NOTE 3 The description "brushed, screeded or rough-tamped" covers those cases where some form of deliberate surface roughening has taken place but not to the extent of exposing the aggregate. NOTE 4 For structural assessment purposes, it may be assumed that the appropriate value of V_m included in the table is 1.5.						Precast unit	Surface type	Grade of in-situ concrete			25 N/mm ²	30 N/mm ²	40 and over N/mm ²	Without links	As-cast or as-extruded	0.4	0.55	0.65	Brushed, screeded or rough-tamped	0.6	0.65	0.75	Washed to remove laitance or treated with retarder and cleaned	0.7	0.75	0.80	With nominal links projecting into in-situ concrete	As-cast or as-extruded	1.2	1.8	2.0	Brushed, screeded or rough-tamped	1.8	2.0	2.2	Washed to remove laitance or treated with retarder and cleaned	2.1	2.2	2.5
Precast unit	Surface type	Grade of in-situ concrete																																					
		25 N/mm ²	30 N/mm ²	40 and over N/mm ²																																			
Without links	As-cast or as-extruded	0.4	0.55	0.65																																			
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	Washed to remove laitance or treated with retarder and cleaned	2.1	2.2	2.5																																			
Longitudinal shear stress limit for no nominal / design vertical reinforcement				96%	OK																																		
Required nominal vertical reinforcement per unit length, $0.15\%b_w$				750	mm ² /m cl.5.4.7.3																																		
Provided vertical reinforcement per unit length, A_e				3142	mm ² /m																																		
Note $A_e = A_{sv,prov} / S$;																																							
Required nominal vertical reinforcement per unit length utilisation, $0.15\%b_w/A_e$				24%	OK																																		
Note UT set to 0% if longitudinal shear stress limit for no nominal vertical reinforcement UT <= 100%;																																							
Required design vertical reinforcement per unit length, A_h				2593	mm ² /m																																		
$A_h = \frac{1\ 000 b v_h}{0.95 f_y}$					cl.5.4.7.4																																		
Required design vertical reinforcement per unit length utilisation, A_h/A_e				0%	OK																																		
Note UT set to 0% if longitudinal shear stress limit for no design vertical reinforcement UT <= 100%;																																							

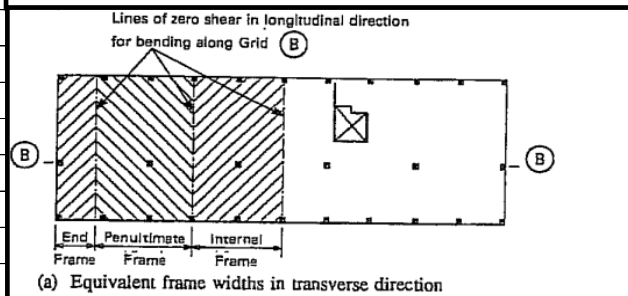
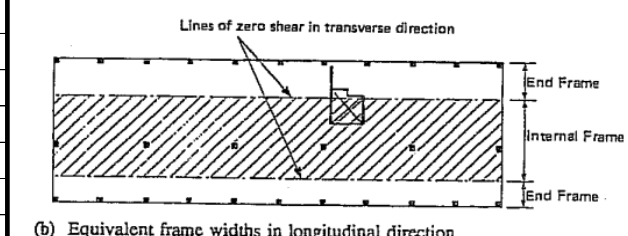
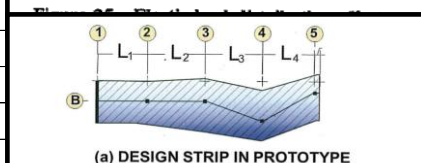
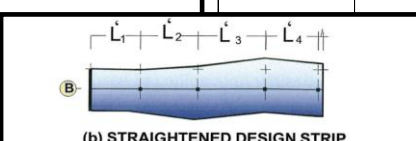
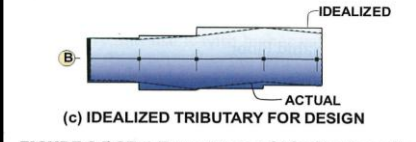
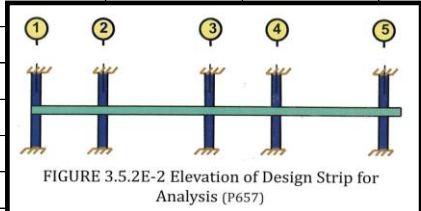
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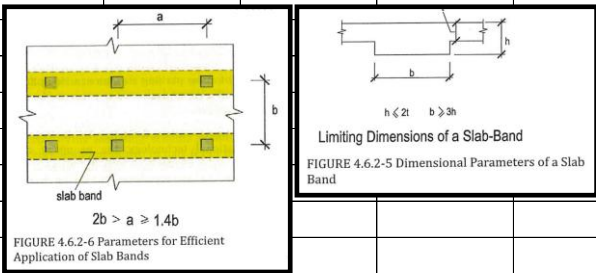
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Investigation into Significant Design Parameters																																																																																													
TLS / SLS Stress Capacity increases as: –																																																																																													
(a)	prestress force, KP_0	increases																																																																																											
(b)	tendon eccentricity, e	increases																																																																																											
(c)	section modulus, $Z_{b/t, TLS/(SLS/ULS)}$	increases																																																																																											
TLS / SLS Deflection Capacity increases as: –																																																																																													
(a)	serviceability class 1 or 2	adopted	instead of serviceability class 3																																																																																										
(b)	section second moment of area, $I_{TLS/(SLS/ULS)}$	increases																																																																																											
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(d)	prestress tendon(s) eccentricity, e	increases																																																																																											
ULS Moment Capacity increases as: –																																																																																													
	$M_u = f_{pb} A_{ps} (d - d_n)$																																																																																												
(a)	ratio of design effective prestress to ultimate tensile strength in reinforcement, f_{pe}/f_{pu} in design tensile stress in tendons, f_{pb} increases and ratio of tensile capacity to concrete capacity, $[f_{pu} A_{ps}]/[f_{cu} b d]$ decreases (T.4.4 BS8110-1)																																																																																												
	$\frac{f_{pe}}{f_{pu}} = \frac{KP_0 / (N_T \cdot N_s \cdot A_s + A_{s,prov} \cdot f_y / f_{pk})}{f_{pk}} \leq 0.60$																																																																																												
	<table border="1"> <caption>Table 4.4 — Conditions at the ultimate limit state for rectangular beams with pre-tensioned tendons or post-tensioned tendons having effective bond</caption> <thead> <tr> <th rowspan="3">$\frac{f_{pu} A_{ps}}{f_{cu} b d}$</th> <th colspan="3">Design stress in tendons as a proportion of the design strength, $f_{pb}/0.95f_{pu}$</th> <th colspan="3">Ratio of depth of neutral axis to that of the centroid of the tendons in the tension zone, x/d</th> </tr> <tr> <th colspan="3">f_{pe}/f_{pu}</th> <th colspan="3">f_{pe}/f_{pu}</th> </tr> <tr> <th>0.6</th> <th>0.5</th> <th>0.4</th> <th>0.6</th> <th>0.5</th> <th>0.4</th> </tr> </thead> <tbody> <tr><td>0.05</td><td>1.00</td><td>1.00</td><td>1.00</td><td>0.12</td><td>0.12</td><td>0.12</td></tr> <tr><td>0.10</td><td>1.00</td><td>1.00</td><td>1.00</td><td>0.23</td><td>0.23</td><td>0.23</td></tr> <tr><td>0.15</td><td>0.95</td><td>0.92</td><td>0.89</td><td>0.33</td><td>0.32</td><td>0.31</td></tr> <tr><td>0.20</td><td>0.87</td><td>0.84</td><td>0.82</td><td>0.41</td><td>0.40</td><td>0.38</td></tr> <tr><td>0.25</td><td>0.82</td><td>0.79</td><td>0.76</td><td>0.48</td><td>0.46</td><td>0.45</td></tr> <tr><td>0.30</td><td>0.78</td><td>0.75</td><td>0.72</td><td>0.55</td><td>0.53</td><td>0.51</td></tr> <tr><td>0.35</td><td>0.75</td><td>0.72</td><td>0.70</td><td>0.62</td><td>0.59</td><td>0.57</td></tr> <tr><td>0.40</td><td>0.73</td><td>0.70</td><td>0.66</td><td>0.69</td><td>0.66</td><td>0.62</td></tr> <tr><td>0.45</td><td>0.71</td><td>0.68</td><td>0.62</td><td>0.75</td><td>0.72</td><td>0.66</td></tr> <tr><td>0.50</td><td>0.70</td><td>0.65</td><td>0.59</td><td>0.82</td><td>0.76</td><td>0.69</td></tr> </tbody> </table>				$\frac{f_{pu} A_{ps}}{f_{cu} b d}$	Design stress in tendons as a proportion of the design strength, $f_{pb}/0.95f_{pu}$			Ratio of depth of neutral axis to that of the centroid of the tendons in the tension zone, x/d			f_{pe}/f_{pu}			f_{pe}/f_{pu}			0.6	0.5	0.4	0.6	0.5	0.4	0.05	1.00	1.00	1.00	0.12	0.12	0.12	0.10	1.00	1.00	1.00	0.23	0.23	0.23	0.15	0.95	0.92	0.89	0.33	0.32	0.31	0.20	0.87	0.84	0.82	0.41	0.40	0.38	0.25	0.82	0.79	0.76	0.48	0.46	0.45	0.30	0.78	0.75	0.72	0.55	0.53	0.51	0.35	0.75	0.72	0.70	0.62	0.59	0.57	0.40	0.73	0.70	0.66	0.69	0.66	0.62	0.45	0.71	0.68	0.62	0.75	0.72	0.66	0.50	0.70	0.65	0.59	0.82	0.76	0.69
$\frac{f_{pu} A_{ps}}{f_{cu} b d}$	Design stress in tendons as a proportion of the design strength, $f_{pb}/0.95f_{pu}$			Ratio of depth of neutral axis to that of the centroid of the tendons in the tension zone, x/d																																																																																									
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(b)	prestress tendon(s) area, A_{ps}	increases																																																																																											
(c)	section eff. depth, d_{ps}	increases																																																																																											
ULS Shear Capacity increases as: –																																																																																													
	$V_{co} = 0.67 b_v h \sqrt{f_t^2 + 0.8 f_{cp} f_t}$																																																																																												
	$V_{cr} = (1 - 0.55 \frac{f_{pe}}{f_{pu}}) v_c b_v d + M_o \frac{V}{M}$																																																																																												
(a)	section width, b_w in $b_v = b_w - (2/3 BD, 1 \text{ un-BD})$	$N_T \cdot D_T$	increases																																																																																										
(b)	section depth, h		increases																																																																																										
(c)	concrete grade, f_{cu} in $f_t = 0.24 \sqrt{f_{cu}}$		increases																																																																																										
(d)	prestress force, KP_0 in $f_{cp} = KP_0 / A_{(SLS/ULS)}$		increases																																																																																										
(e)	ratio of design effective prestress to ultimate tensile strength in reinforcement, f_{pe}/f_{pu} decreases																																																																																												
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(f)	% of tensile area, $\rho_w = 100(N_T \cdot N_s \cdot A_s + A_{s,prov})/b_w d$ and/or concrete grade, f_{cu} in v_c increase																																																																																												
(g)	prestress force, KP_0 and/or tendon eccentricity, $e_{var}(x=x_d)$ in f_{pt} and/or section modulus, $Z_{b/t, (SLS/ULS)}$ in $M_o = 0.8 f_{pt} Z_{b/t, (SLS/ULS)}$ increase																																																																																												
	$f_{pt} = \frac{KP_0}{A_{(SLS/ULS)}} \pm \frac{KP_0 e_{var}(x=x_d)}{Z_{b/t, (SLS/ULS)}}$																																																																																												
(h)	ratio of applied shear force to bending moment, V/M increases																																																																																												

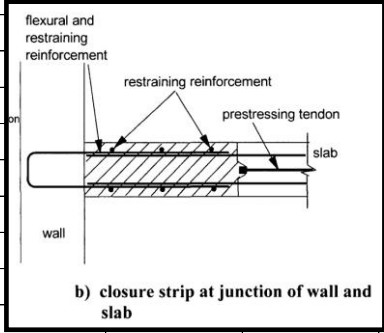
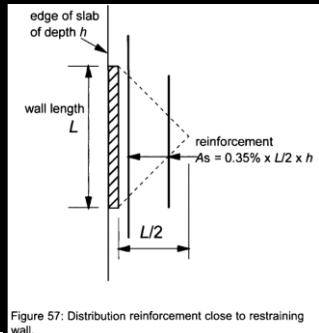
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	53			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
										BS8110
Concepts in Prestressed Concrete										
1						The prestress tendon(s) provide a suspension system within the member with the vertical component (which exists due to the eccentricity, e) of the tendon force carrying part of the dead and live loading and the horizontal component reducing the tensile stresses in the concrete.				cl.1.0 TR.43
2						Since prestressing is an internal force and not an external action, unlike the latter, prestress force cannot buckle a member - as long as the prestress is bonded . This occurs because as the compression in the member tries to buckle, the equal and opposite tension in the cable prevents it from doing so. As such a slender member can never buckle under prestress alone. Furthermore, a curved prestressed member cannot buckle either - it simply has an axial P/A .				IStructE Bourne Prestressing Feb-13
3						In continuous beams, secondary moments (parasitic moments) vary linearly as sagging moments between supports. Thus when combined with the external effects moments, it can be used to reduce the overall hogging moments and increase the sagging moments, effectively equalising the hogging and sagging moments. Secondary effects also include constant axial and shear forces throughout the span.				IStructE Bourne Prestressing Feb-13 cl.6.9 TR.43
4						Equivalent loads will automatically generate primary and secondary effects when applied to the structure. SLS calculations do not require any separation of the primary and secondary effects, and analysis using the equivalent loads is straightforward. However, at ULS the two effects must be separated because the secondary effects are treated as applied loads. The primary prestressing effects are taken into account by including the tendon force in the calculation of the ultimate section capacity. The primary prestressing forces and moments must therefore be subtracted from the equivalent load analysis to give the secondary effects.				cl.6.9 TR.43
						  Fig 6.3 Equivalent loads				
5						Favourable arrangements of restraining walls should be adopted to minimise the restraint force that reduces the prestress in the member, failing which pour strips should be employed.				IEM Mar-15
						  Figure 57: Distribution reinforcement close to restraining wall.				
6						Long-span insitu beams on bearings need to be designed to cater for the transfer of prestress force and displacement into the bearings .				IEM Mar-15
7						Prestressing of ground slabs and beams needs to be carefully evaluated as the restraining effect of the ground, pile caps or even piles need to be considered.				IEM Mar-15

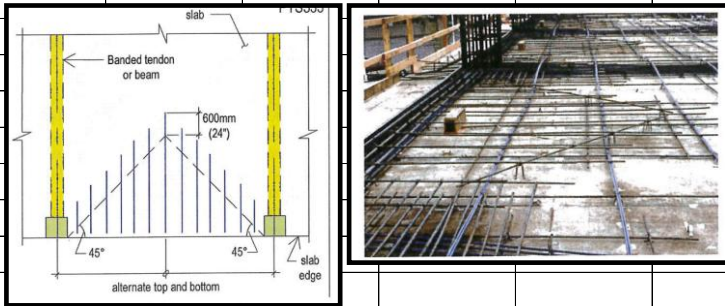
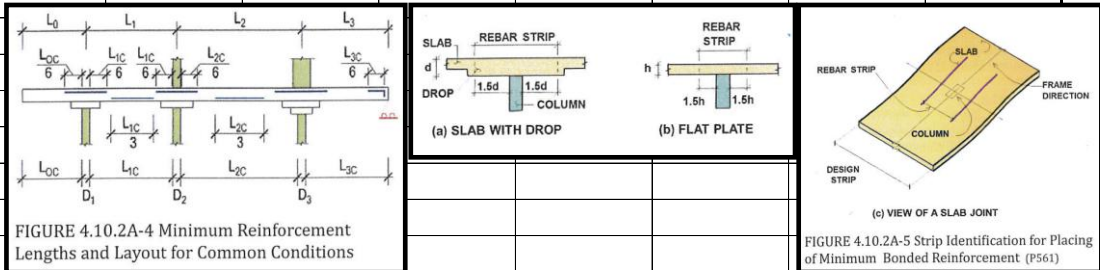
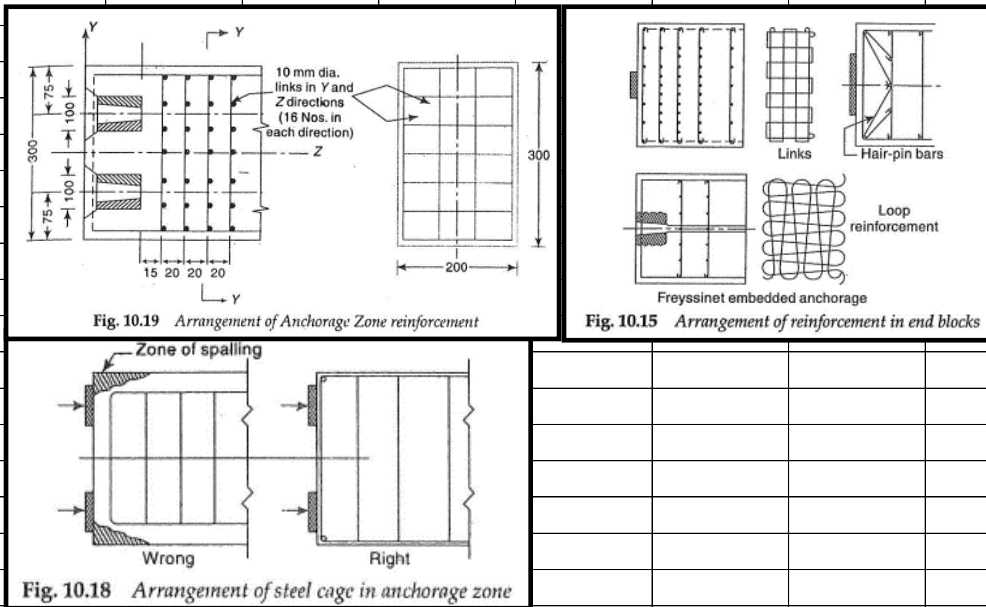
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.		
						jXXX	54			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
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8						Beams curved on plan are susceptible to torsion from prestressing as the tendon in the beam will apply an eccentric radial force about the beam's centroid, giving rise to torsional moments.				IEM Mar-15
9						Accurate measurement of the tendon elongation during stressing and its comparison with predictions are crucial in determining if stressing has been carried out properly. Any discrepancy could be attributed to faulty jacks, tendon breakages, leakage of grout into ducting, overstressing or understressing.				IEM Mar-15
10						The extent of pours is usually dictated by the limit to the length of tendons. With bonded tendons, friction losses usually restrict the length of single end stressed tendons to 25m, and double end stressed to 50m. The lower friction values for unbonded tendons extend these values to 35m and 70m respectively. Either intermediate anchorages are introduced to allow continuous stressing across the construction joint or alternatively infill strips are used.				cl.7.7.1 TR.4
						 				
11						For uniformly loaded and regular concrete frames, the impact of post-tensioning results in an increase in the axial force at the end supports, reduction of axial forces at the penultimate supports and design insignificant impact on the axial forces of the remainder supports. Post-tensioning reduces the design moments for the "strength condition" at the top of member supports. Post-tensioning in a floor results in redistribution of axial forces on walls and columns. However the sum of the axial forces for any given floor remains unchanged.				Aalami, 201
						    Impact of Post-Tensioning on Typical Level of a High Rise Building				
12						Recovery of the loss of precompression due to restraints occurs with typical floors. At the first suspended floor, the restraint of the supports and foundation absorb a fraction of the precompression intended for the floor being stressed. When the subsequent floor is post-tensioned, the restraint of its supports is somewhat less than that experienced by the floor below it. Again, a fraction of the precompression of the new floor is diverted to the structure below it. This results in partial recovery of the loss of prestressing in the first suspended floor. The pattern will continue with the initial loss of precompression to the level being recovered when the level above is stressed. Eventually, the precompression lost to the penultimate floor from the uppermost floor is not recovered, since there is no floor above it to be stressed.				Aalami, 201

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						BS8110				
Concepts in Prestressed Concrete in Flat Slabs										
1 Flat slab criteria include: -						cl.2.4.1 TR.4				
(a) precompression should be applied in two orthogonal directions										
(b) aspect ratio of any panel should not be greater than 2.0										
(c) the ratio of stiffness of the slab in two orthogonal directions should not exceed 10.0										
2 The concept of design strips is employed when analysing flat slabs using the equivalent frame method or the FE analysis method .						cl.6.6 TR.43				
3	It is usual to divide the structure into sub-frame elements in each direction. Each frame usually comprises one line of columns together with beam/slab elements of one bay width. The frames chosen for analysis should cover all the element types of the complete structure.  (a) Equivalent frame widths in transverse direction  (b) Equivalent frame widths in longitudinal direction  (a) DESIGN STRIP IN PROTOTYPE  (b) STRAIGHTENED DESIGN STRIP  (c) IDEALIZED TRIBUTARY FOR DESIGN FIGURE 3.5.2E-1 Extraction and Idealization of a Design Strip (P656)  FIGURE 3.5.2E-2 Elevation of Design Strip for Analysis (P657)									
3 Flat slabs should be reinforced to resist the moment from the full load in each orthogonal direction , and not by considering a reduced load when analysing the slab in any one direction using the equivalent frame method (as opposed to the FE analysis method), i.e.: -						cl.2.4 TR.43				
Effect		Floor Type		Hogging Moment		Sagging Moment				
BM-Interior	One way spanning slab	0.063n.L ²		n.L ² /16		0.063n.L ²		n.L ² /16		kNm/m
BM-Interior	Two way spanning slab	0.031n.L ²		n.L ² /32		0.024n.L ²		n.L ² /42		kNm/m
BM-Interior	Flat slab	0.063n.L ²		n.L ² /16		0.063n.L ²		n.L ² /16		kNm/m
Note n is the ULS slab loading (kPa). The coefficients above assume an interior span and include a 20% moment redistribution. The coefficients for two way spanning and flat slab assume a square panel.										
Conversely, the FE analysis method (as opposed to the equivalent frame method) inherently incorporates the biaxial behaviour of the floor system when determining the actions in the floor.						Aalami, 201				

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Member Design - PC Beam and Slab						Made by	XX	Date 24/10/2026 Chd.
								BS8110
4 Flat slab with drop panel dimensional requirements: -								
- width of drop panel $\geq$ shorter span / 3								cl.3.7.1.5 BS8110, T
- depth of drop panel (excluding slab) $\geq$ 3/4 x slab thickness								T.1 TR.43
5 Flat slab with slab band (economic for aspect ratio 1.4-2.0) dimensional requirements: -								
- width of slab band $\geq$ span / 5								T.1 TR.43
- width of slab band $\geq$ 3 x slab thickness								Aalami, 201
- width of slab band $\approx$ 0.4 x design strip width to maximise tendon drape								SELF
- depth of slab band (excluding slab) $\geq$ 3/4 x slab thickness								SELF
- depth of slab band (excluding slab) $\leq$ slab thickness								Aalami, 201
								
6 Flat slab with slab band (or insitu beam for that matter) which exhibits a T- or L- section should be represented by a constant second moment of area, <i>I</i> throughout its span irrespective of whether the section is hogging or sagging. This is unlike an RC flanged section which reverts to a rectangular section when the section is hogging. Further to this, in commercial 2D FE software (unlike 1D software), when the section is represented by a T or L- section , the design strip width should be limited (simplistically to the column strip width) in lieu of the full tributary width in order to model the effect of the reduced <i>I</i> (and <i>Z</i>) corresponding to the T- or L- section effective flange width .								
7 Flat slab deflection criteria : -								
(a) maximum downward SLS deflection due to SLS load combination case G+Q+PT with $E=E_{lt}=E_{ck,cp}$ which is based upon the summation of: -								
the loading, $\omega_{SLS,E/E}$ (+ve) with elastic modulus of the slab, $E_{lt}=E_{ck,cp}$								
the loading, $\omega_{SLS,E/L}$ (-ve) with elastic modulus of the slab, $E_{lt}=E_{ck,cp}$								
with respect to [span/250].C₁								
(b) incremental downward creep+LL deflection due to the summation of the load cases: -								
$(1 - 1/(1 + \phi)).(1 - \%creep).DL = 0.30DL$, $\phi = 1.0$, $\%creep = 40\%$ with $E=E_{lt}=E_{ck,cp}$								
or $(1 - 1/(1 + \phi)).(1 - \%creep).DL = 0.36DL$, $\phi = 1.5$, $\%creep = 40\%$ with $E=E_{lt}=E_{ck,cp}$								
+ 1.0SDL with $E=E_{lt}=E_{ck,cp}$								
+ 1.0Q with $E=E_{lt}=E_{ck,cp}$								
+ $(1 - 1/(1 + \phi)/K_{LT}.K_{ST}).(1 - \%creep).PT = 0.26PT$, $\phi = 1.0$, $\%creep = 40\%$, $K_{LT ST} \approx 0.8 0.9$ with E								
or $(1 - 1/(1 + \phi)/K_{LT}.K_{ST}).(1 - \%creep).PT = 0.33PT$, $\phi = 1.5$, $\%creep = 40\%$, $K_{LT ST} \approx 0.8 0.9$ with E								
which is based upon the summation of: -								
the loading, $k_C.(\omega_{DL+SDL}) + \omega_{LL}$ (+ve) with elastic modulus of the slab, $E_{lt}=E_{ck,cp}$								
the loading, $\omega_{SLS,E/L}$ (-ve) with elastic modulus of the slab, $E_{lt}=E_{ck,cp}$								
the loading, $-\omega_{TLS,E/L}$ (+ve) with elastic modulus of the slab, $E_{st}=E_{ck}$								
with respect to MIN {[span/500].C₁, 20mm} noting that the creep term also includes a total (elastic, creep, shrinkage) axial shortening component of the (one) storey in question : -								
column ULS stress %						50%	f_{cu}	
column SLS stress % = column ULS stress % / k_G						36%	f_{cu}	
column SLS stress, σ_{SLS} = column SLS stress % . f_{cu}						14.3	N/mm ²	
storey height, h_s						3000	mm	
column elastic modulus, $E_{col} = E_{uncracked,28,cp}$						9.3	GPa	
total (elastic, creep, shrinkage) axial shortening, $\delta_{ES,st} = \sigma_{SLS}$ [of the (one) storey in question]						4.6	mm	

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						jXXX	59			
						Member/Location				
Job Title	Member Design - Prestressed Concrete Beam and Slab					Drg. Ref.				
Member Design - PC Beam and Slab						Made by	XX	Date	24/10/2026	Chd.
						BS8110				
Additional Detailing Requirements										
1 The provision of minimum longitudinal steel (untensioned reinforcement) for unbonded tendon construction. cl.6.10.6 TR.										
2 The provision of flexural longitudinal (in the case of slip/jump formed walls where tendons do not protrude into the wall, unlike floor-by-floor construction) and restraining transverse steel (untensioned reinforcement) near restraining walls accounting for the effects of: -										
(a) (elastic, creep and shrinkage) restraint to axial precompression (inducing tension in the top and bottom longitudinal reinforcement), reduced with the introduction of a pour strip										
(b) bending moment due to SLS load combination case(s) 1.0G+1.0Q+ PT [and performing an SLS RC (stress based) longitudinal reinforcement design based on cl.6.10.5 TR.43] and bending moment due to ULS load combination case(s) $k_G \cdot G + k_Q \cdot Q + \text{HYP}$ [and performing a ULS RC longitudinal reinforcement design] (inducing tension in the top longitudinal reinforcement), reduced with the introduction of a pour strip and/or the allowance of transverse cracking with the assumption of a pinned wall support, noting that the SLS/ULS load combination case(s) should consider both methods of frame analysis, i.e. w.o./w.										
differential (elastic, creep, shrinkage) axial shortening of adjacent supports: -										
wall ULS stress %						40%	f_{cu}			
wall SLS stress % = wall ULS stress % / k_G						29%	f_{cu}			
wall SLS stress, σ_{SLS} = wall SLS stress % . f_{cu}						11.4	N/mm ²			
column ULS stress %						50%	f_{cu}			
column SLS stress % = column ULS stress % / k_G						36%	f_{cu}			
column SLS stress, σ_{SLS} = column SLS stress % . f_{cu}						14.3	N/mm ²			
number of storeys below, N_s						50	storeys			
typical storey height, h_s						3000	mm			
wall/column elastic modulus, $E_{wall/col} = E_{uncracked,28,cp}$						9.3	GPa			
differential axial shortening, $\Delta\delta_{ES,st} = \Delta\sigma_{SLS} \cdot N_s \cdot h_s / E_{wall/col}$						45.9	mm			
[of adjacent supports at the storey in question]										
(c) shear force due to ULS load combination case(s) $k_G \cdot G + k_Q \cdot Q + \text{HYP}$ [and performing a ULS RC shear reinforcement design] (with the area of the top longitudinal reinforcement contributing to the ULS RC shear capacity), noting that the ULS load combination case(s) should consider both methods of frame analysis, i.e. w.o./w. differential (elastic, creep, shrinkage) axial shortening of adjacent supports										
 b) closure strip at junction of wall and slab  Figure 57: Distribution reinforcement close to restraining wall. Note that the elastic modulus of the wall and column supports for the differential axial shortening assessment should be $E_{uncracked,28,cp}$										
3 The provision of flexural longitudinal (in the case of slip/jump formed walls where tendons do not protrude into the wall, unlike floor-by-floor construction) steel (untensioned reinforcement) near restraining walls accounting for the effects of: -										
(a) shear force due to ULS load combination case(s) $k_G \cdot G + k_Q \cdot Q + \text{HYP}$ [and performing a ULS RC dowel shear action reinforcement design] (with the area of the bottom longitudinal reinforcement contributing to the ULS RC dowel shear action capacity), noting that the ULS load combination case(s) should consider both methods of frame analysis, i.e. w.o./w. differential (elastic, creep, shrinkage) axial shortening of adjacent supports										

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4 The provision of longitudinal and transverse steel (untensioned reinforcement) between tendon anchorages at flat slab edges as follows: -				
(a) parallel to the edge , untensioned and/or tensioned reinforcement to resist the ULS bending moment for a continuous slab spanning l_a , which is the centre to centre distance between (groups of) anchorages, evenly distributed across a width of $0.7l_a$				
(b) perpendicular to the edge , untensioned reinforcement greater than $0.13\%bh$ and $1/4 \times$ parallel reinforcement, evenly distributed between the anchorages and extending $\text{MAX}(l_a, 0.7l_a + \text{anchorage})$				
				
5 The provision of minimum longitudinal steel (untensioned reinforcement) at column positions for all flat slabs of at least 0.075% of the gross concrete cross-sectional area, concentrated between lines that are 1.5 times the slab depth either side of the width of the column and extending 0.2L into the span, L.				
				
6 End block detailing as follows: -				
				

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Creep Coefficient, ϕ		cl.7.3																																																																																																																																														
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		The creep coefficient may be estimated from Figure 7.1. In this Figure, the effective section thickness is defined, for uniform sections, as twice the cross-sectional area divided by the exposed perimeter. If drying is prevented by immersion in water or by sealing, the effective section thickness should be taken as 600mm. Suitable values of relative humidity for indoor and outdoor exposure in the UK are 45 % and 85 %, when using Figure 7.1 for general design purposes. It can be assumed that about 40 %, 60 % and 80 % of the final creep develops during the first month, 6 months and 30 months under load respectively, when concrete is exposed to conditions of constant relative humidity;																																																																																																																																														
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		AS3600																																																																																																																																														
TABLE 3.1.8.3 FINAL CREEP COEFFICIENTS (AFTER 30 YEARS) FOR CONCRETE FIRST LOADED AT 28 DAYS <table border="1"> <thead> <tr> <th rowspan="4">f'_c (MPa)</th> <th colspan="12">Final creep coefficient (ϕ_{cc})</th> </tr> <tr> <th colspan="3">Arid environment</th> <th colspan="3">Interior environment</th> <th colspan="3">Temperate inland environment</th> <th colspan="3">Tropical, near-coastal and coastal environment</th> </tr> <tr> <th colspan="3">t_h (mm)</th> <th colspan="3">t_h (mm)</th> <th colspan="3">t_h (mm)</th> <th colspan="3">t_h (mm)</th> </tr> <tr> <th>100</th> <th>200</th> <th>400</th> <th>100</th> <th>200</th> <th>400</th> <th>100</th> <th>200</th> <th>400</th> <th>100</th> <th>200</th> <th>400</th> </tr> </thead> <tbody> <tr> <td>25</td> <td>4.82</td> <td>3.90</td> <td>3.27</td> <td>4.48</td> <td>3.62</td> <td>3.03</td> <td>4.13</td> <td>3.34</td> <td>2.80</td> <td>3.44</td> <td>2.78</td> <td>2.33</td> </tr> <tr> <td>32</td> <td>3.90</td> <td>3.15</td> <td>2.64</td> <td>3.62</td> <td>2.93</td> <td>2.46</td> <td>3.34</td> <td>2.70</td> <td>2.27</td> <td>2.79</td> <td>2.25</td> <td>1.90</td> </tr> <tr> <td>40</td> <td>3.21</td> <td>2.60</td> <td>2.18</td> <td>2.98</td> <td>2.41</td> <td>2.02</td> <td>2.75</td> <td>2.23</td> <td>1.87</td> <td>2.30</td> <td>1.86</td> <td>1.56</td> </tr> <tr> <td>50</td> <td>2.75</td> <td>2.23</td> <td>1.89</td> <td>2.56</td> <td>2.07</td> <td>1.73</td> <td>2.36</td> <td>1.91</td> <td>1.60</td> <td>1.97</td> <td>1.59</td> <td>1.33</td> </tr> <tr> <td>65</td> <td>2.07</td> <td>1.75</td> <td>1.53</td> <td>1.95</td> <td>1.66</td> <td>1.46</td> <td>1.84</td> <td>1.59</td> <td>1.38</td> <td>1.61</td> <td>1.38</td> <td>1.23</td> </tr> <tr> <td>80</td> <td>1.56</td> <td>1.40</td> <td>1.29</td> <td>1.50</td> <td>1.36</td> <td>1.25</td> <td>1.45</td> <td>1.32</td> <td>1.22</td> <td>1.33</td> <td>1.23</td> <td>1.14</td> </tr> <tr> <td>100</td> <td>1.15</td> <td>1.14</td> <td>1.11</td> <td>1.15</td> <td>1.14</td> <td>1.11</td> <td>1.15</td> <td>1.14</td> <td>1.11</td> <td>1.15</td> <td>1.14</td> <td>1.11</td> </tr> </tbody> </table>					f'_c (MPa)	Final creep coefficient (ϕ_{cc})												Arid environment			Interior environment			Temperate inland environment			Tropical, near-coastal and coastal environment			t_h (mm)			t_h (mm)			t_h (mm)			t_h (mm)			100	200	400	100	200	400	100	200	400	100	200	400	25	4.82	3.90	3.27	4.48	3.62	3.03	4.13	3.34	2.80	3.44	2.78	2.33	32	3.90	3.15	2.64	3.62	2.93	2.46	3.34	2.70	2.27	2.79	2.25	1.90	40	3.21	2.60	2.18	2.98	2.41	2.02	2.75	2.23	1.87	2.30	1.86	1.56	50	2.75	2.23	1.89	2.56	2.07	1.73	2.36	1.91	1.60	1.97	1.59	1.33	65	2.07	1.75	1.53	1.95	1.66	1.46	1.84	1.59	1.38	1.61	1.38	1.23	80	1.56	1.40	1.29	1.50	1.36	1.25	1.45	1.32	1.22	1.33	1.23	1.14	100	1.15	1.14	1.11	1.15	1.14	1.11	1.15	1.14	1.11	1.15	1.14	1.11
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Concepts in Prestressed Concrete (Calculation of Secondary Effects Using E/L)										

APPENDIX D: Calculation of Secondary Effects Using Equivalent Loads

Equivalent loads can be used to represent the forces from prestress. These will automatically generate the combined primary and secondary effects when applied to the structure. Figure D1 shows the commonly occurring equivalent loads for typical prestress situations.

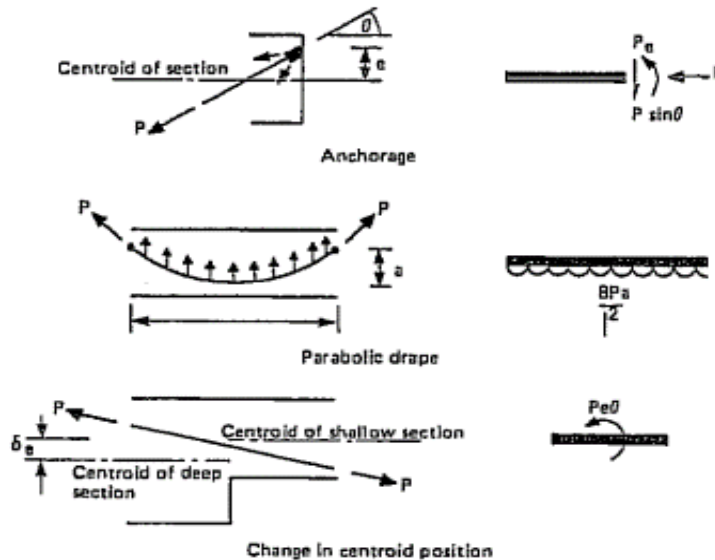


Figure D1: Commonly occurring equivalent loads

One method of separating the secondary from the primary effects is to use a frame analysis with the equivalent prestress load acting alone. The resultant moment and shear diagrams include both the primary and secondary effects. In order to obtain the secondary effects, it is only necessary to consider the moments and forces at the supports and subtract the primary effects from them. The secondary moments along each span vary linearly from end to end. This method will be known as method A.

To illustrate method A, the Ultimate Limit State for the transverse direction in Example A1 of Appendix A is used and the secondary effects obtained as follows:

1. Calculate the equivalent prestress loads in the spans using a load factor of 1.0.

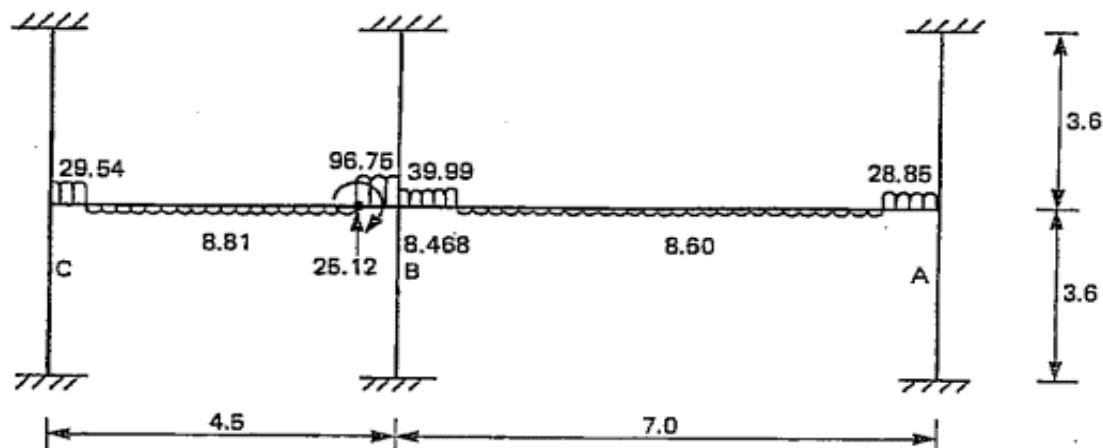


Figure D2: Equivalent balanced loads

Figure D4: Bending moment diagram due to secondary effects

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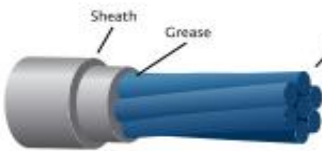
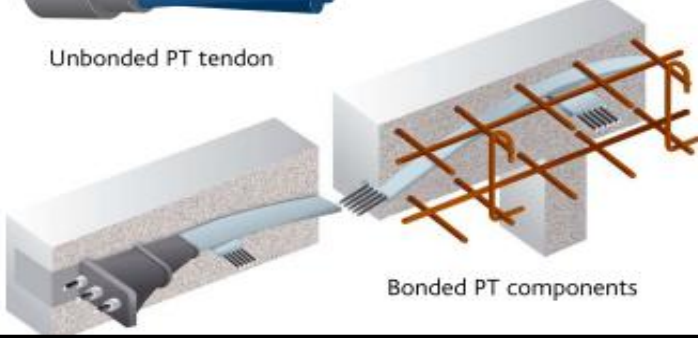
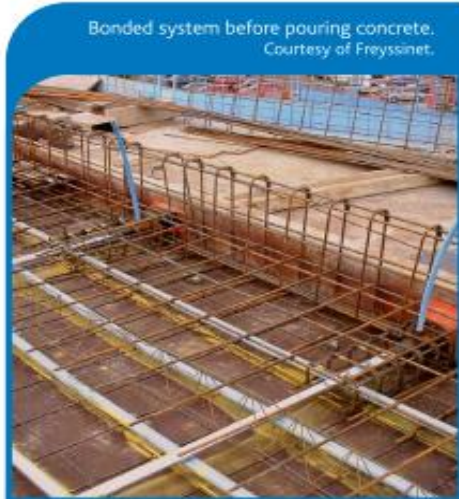
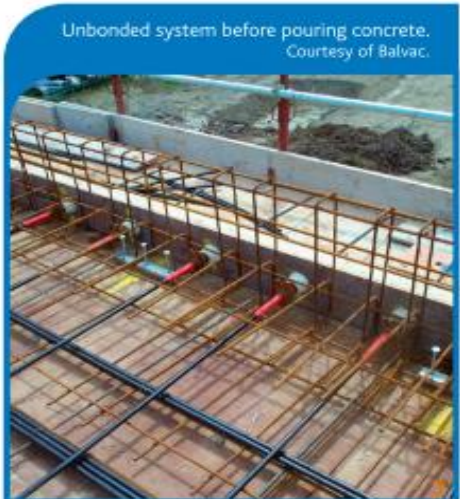
Figure D5: Shear force diagram due to secondary effects

An alternative method of calculating secondary effects is detailed below. This will be known as method B.

As there are no primary prestress forces in the columns, the column moments and reactions are entirely due to secondary effects. So the secondary effects in the slab can be easily obtained by applying these column reactions and moments to the slab as shown in Figure D6.

Figure D6: Column reactions and moments due to secondary forces

This results in the secondary moments and shears in the slab as shown in Figures D4 and D5.

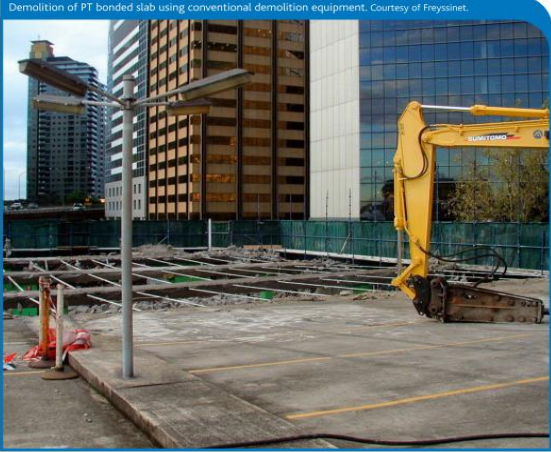
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Pre-Tension or Post-Tension																												
<table border="1"> <thead> <tr> <th>Aspect</th> <th>Pre-Tensioning</th> <th>Post-Tensioning</th> </tr> </thead> <tbody> <tr> <td>Timing for Tensioning</td> <td>Before Concrete Hardening</td> <td>After Concrete Hardening</td> </tr> <tr> <td>Construction</td> <td>Precast Offsite</td> <td>Insitu</td> </tr> <tr> <td>Member Size</td> <td>Small</td> <td>Large</td> </tr> <tr> <td>Tendon Configuration</td> <td>Straight / Deflected</td> <td>Curved</td> </tr> <tr> <td>Bonded or Unbonded</td> <td>Bonded</td> <td>Bonded or Unbonded</td> </tr> </tbody> </table>											Aspect	Pre-Tensioning	Post-Tensioning	Timing for Tensioning	Before Concrete Hardening	After Concrete Hardening	Construction	Precast Offsite	Insitu	Member Size	Small	Large	Tendon Configuration	Straight / Deflected	Curved	Bonded or Unbonded	Bonded	Bonded or Unbonded
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Table 6.1 Advantages and disadvantages of pre- and post-tensioning <table border="1"> <thead> <tr> <th>Type of construction</th> <th>Advantages</th> <th>Disadvantages</th> </tr> </thead> <tbody> <tr> <td>Prefensioned</td> <td> - no need for anchorages - tendons protected by concrete without the need for grouting or other protection - prestress is generally better distributed in transmission zones - factory produced precast units </td> <td> - heavy stressing bed required - more difficult to incorporate deflected tendons </td> </tr> <tr> <td>Post-tensioned</td> <td> - no external stressing bed required - more flexibility in tendon layout and profile - draped tendons can be used <i>in-situ</i> on site </td> <td> - tendons require a protective system - large concentrated forces in end blocks </td> </tr> </tbody> </table>											Type of construction	Advantages	Disadvantages	Prefensioned	- no need for anchorages - tendons protected by concrete without the need for grouting or other protection - prestress is generally better distributed in transmission zones - factory produced precast units	- heavy stressing bed required - more difficult to incorporate deflected tendons	Post-tensioned	- no external stressing bed required - more flexibility in tendon layout and profile - draped tendons can be used <i>in-situ</i> on site	- tendons require a protective system - large concentrated forces in end blocks									
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 Unbonded PT tendon						More versatile bonded systems suitable for floor slabs were developed in Australia. Bonded systems became popular in the UK in the 1990s. In the UK, bonded construction is now widely used; having approximately 90% of the PT suspended floor market.																						
 Bonded PT components																												
 Bonded system before pouring concrete. Courtesy of Freyssinet.						 Unbonded system before pouring concrete. Courtesy of Balvac.																						

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Standard Design Concepts (Restraint to Elastic, Creep and Shrinkage Shortening due to Prestressing)						
Restraint At the early stages of a project using post-tensioned floors, care must be taken to avoid the problems of restraint. This is where the free movement in the length of the slab under the prestress forces is restrained, for example by the unfavourable positioning of shear walls or lift cores (see Figure 9). All concrete elements shrink due to drying and early thermal effects but, in addition, prestressing causes elastic shortening and ongoing shrinkage due to creep. Stiff vertical members such as stability walls restrain the floor slab from shrinking, which prevents the prestress from developing and thus reduces the strength of the floor. Where the restraining walls are in a favourable arrangement and the floor is in an internal environment, the maximum length of the floor without movement joints can be up to 50m. However, full consideration should be given to the effects of shrinkage due to drying, early thermal effects, elastic shortening and creep in the design.				Where the walls are unfavourably arranged then a calculation of the effects of movement should be carried out and suitable measures taken to overcome them. This could involve: • Using infill strips which are usually cast around 28 days after the remainder of the floor, to allow initial shrinkage to occur (see Figure 10). • Increasing the quantity of conventional reinforcement, to control the cracking. • Using temporary release details (see Figure 11). • Reducing the stiffness of the restraining elements. The effect of the floor shortening on the columns should also be considered in their design as this may increase their design moments.		
FIGURE 9: TYPICAL FLOOR LAYOUTS a) Favourable layout of restraining walls (low restraint) b) Unfavourable layout of restraining walls (high restraint) FIGURE 10 : TYPICAL INFILL STRIP 1000 mm RC infill strip Post-tensioned slab 50mm seating Slab to remain fully propped until infill strip cured FIGURE 11: TEMPORARY RELEASE DETAIL Infill later Post-tensioned slab 2 layers of slip strip 100mm bearing						

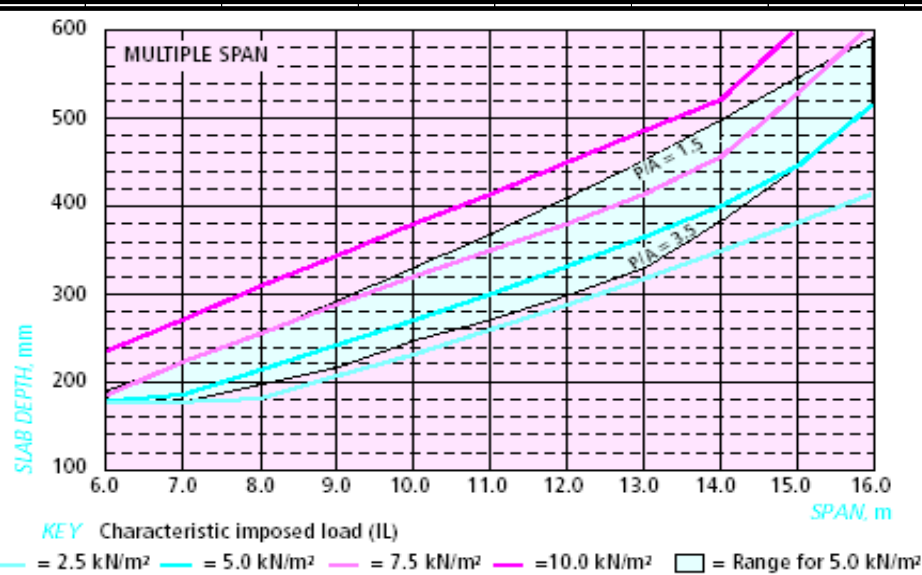
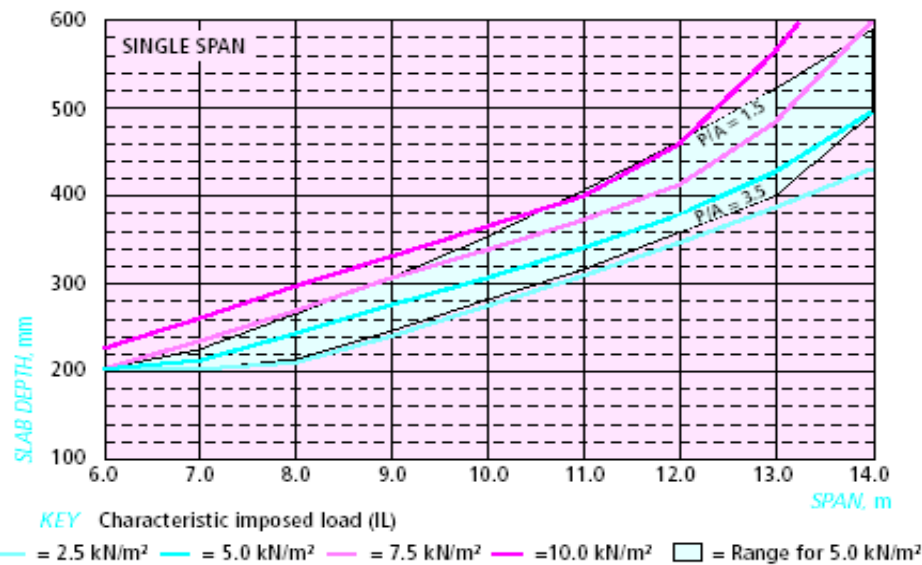
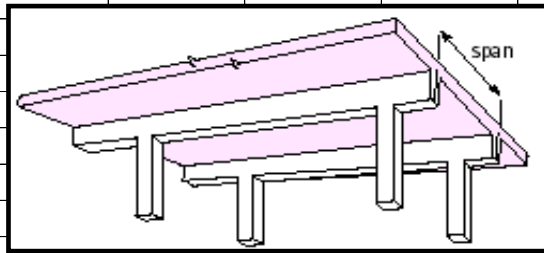
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Standard Design Concepts (Disproportionate Collapse Requirements)									
Design to prevent disproportionate collapse PT floor systems are usually designed to resist disproportionate collapse through detailing of the tendons and reinforcement. In bonded systems the tendons can be considered to act as horizontal ties. In unbonded systems, the tendons cannot be relied on and the conventional reinforcement acts as the horizontal ties.									
Standard Design Concepts (Slab Soffit Marking)									
Slab soffit marking Various methods exist for marking the slab soffit to identify where groups of tendons are fixed. The most common is to use paint markings, usually on the soffit. Alternatively a thin ply sheet may be laid between the tendons to give a physical demarcation. This enables areas for small holes and fixings to be drilled after completion, safe in the knowledge that tendons will not be damaged. The position and maximum depth of fixings should be agreed and clearly conveyed to follow-on trades.									
Standard Design Concepts (Stressing)									
Stressing Ideally after 24 hours, when the concrete has attained a strength of typically 12.5 N/mm², initial stressing of tendons to about 25% of their final jacking force is carried out. (The actual concrete strength and tendon force will vary depending on loadings, slab type and other requirements.) This controls restraint stresses and may also enable the slab to be self-supporting so that formwork can be removed. The tendon is stressed with a hydraulic jack, and the resulting force is locked into the tendon by means of a split wedge located in the barrel of the recessed anchor. At about three to five days, when the concrete has attained its design strength, the remaining stress is applied to the tendons. The extension of each tendon under load is recorded and compared against the calculated value. Provided that it falls within an acceptable tolerance, the tendon is then trimmed. With an unbonded system, a greased cap is placed over the recessed anchor and the remaining void dry-packed. With a bonded system the anchor recess is simply dry-packed and the tendon grouted.									

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						BS8110
Standard Design Concepts (Accommodation of Service Openings)						
Holes and tendon layout A particular design feature of post-tensioned slabs is that the distribution of tendons on plan within the slab does not significantly effect its ultimate strength. There is some effect on strength and shear capacity, but this is generally small. This allows an even prestress in each direction of a flat slab to be achieved with a number of tendon layouts (see Figure 12). This offers considerable design flexibility to allow for penetrations and subsequent openings, and the adoption of differing slab profiles, from solid slabs through to ribbed and waffle construction. Layout (a) of Figure 12 shows the layout of tendons banded over a line of columns in one direction and evenly distributed in the other direction. This layout can be used for solid slabs, ribbed slabs, or band beam and slab floors. It offers the advantages that holes through the slab can be easily accommodated and readily positioned at the construction stage. Layout (b) shows the tendons banded in one direction, and a combination of banding and even distribution in the other direction. This does not provide quite the same flexibility in positioning of holes, but offers increased shear capacity around column heads. Again, this layout can be used for both solid and ribbed slabs and banded beam construction. Layout (c) shows banded and distributed tendons in both directions and is logically suitable for waffle flat slabs, but may be employed for other slabs, depending on design requirements. The disadvantage of this layout is that it requires 'weaving' of the tendons. Holes through post-tensioned slabs can be accommodated easily if they are identified at the design stage. Small holes (less than 300mm x 300mm) can generally be positioned anywhere on the slab, between tendons, without any special requirements. Larger holes are accommodated by locally displacing the continuous tendons around the hole. It is good detailing practice to overlap any stopped off (or 'dead-ended') tendons towards the corners of the holes in order to eliminate any cracking at the corners. In ribbed slabs, holes can be readily incorporated between ribs or, for larger holes, by amending rib spacings or by stopping-off ribs and transferring forces to the adjoining ribs. With flat slabs it is possible to locate holes adjacent to faces of columns. It is important to note that this significantly reduces the punching shear capacity. Holes are more difficult to accommodate once the slab has been cast. They can, however, be carefully cut if the tendon positions have been accurately recorded or can be identified (see page 20). A better approach is to identify at the design stage zones where further penetrations may be placed. These zones can then be clearly marked on the soffit and topside of the slab.						
FIGURE 12: COMMON LAYOUT OF TENDONS FIGURE 13: DETAILING OF TENDONS AROUND AN OPENING FIGURE 14: LAYOUT OF TENDONS TO ALLOW SERVICES TO BE PLACED CLOSE TO COLUMN FACE						

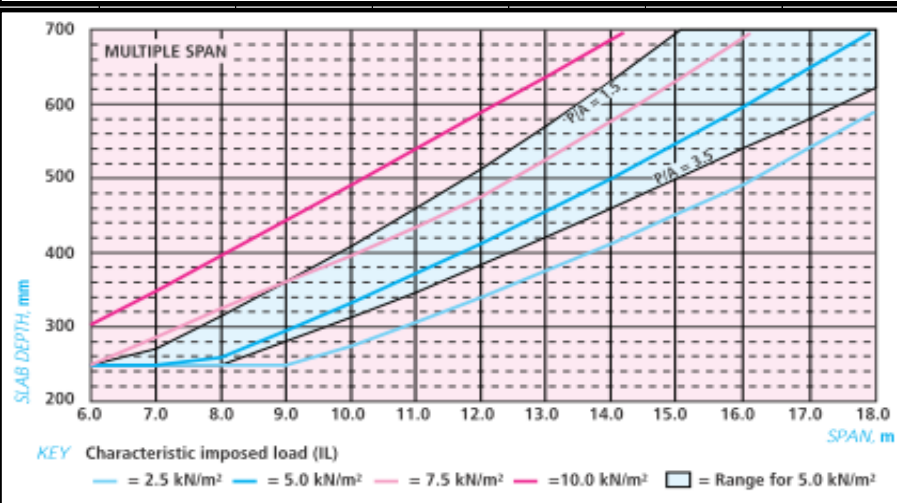
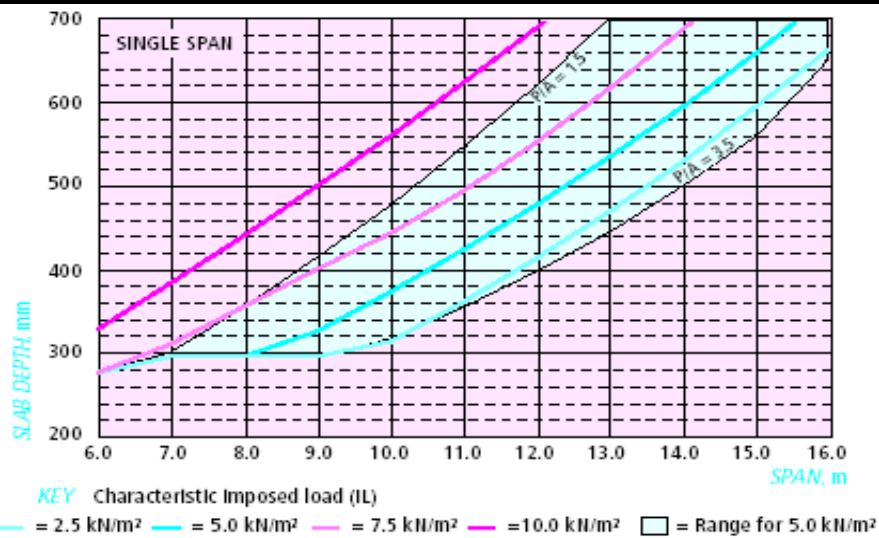
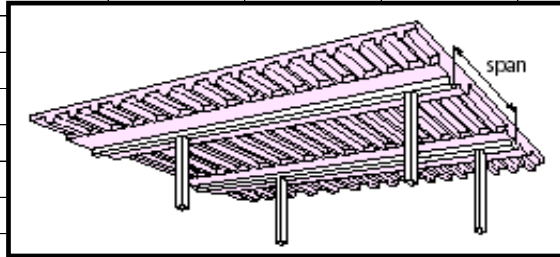
CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
				jXXX	70	
				Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab			Drg. Ref.		
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Standard Design Concepts (Construction Joints and Pour Size)						
Construction joints There are three types of construction joint that can be used between areas of slab; these are shown in Figure 14. When used they are typically positioned in the vicinity of a quarter or third points of the span. The most commonly used joint is the infill or closure strip, as this is an ideal method of resolving problems of restraint, and it also provides inboard access for stressing, removing or reducing the need for perimeter access from formwork or scaffolding. Construction joint with no stressing (Figure 15a) The slab is cast in bays and stressed when all the bays are complete. For large slab areas, control of restraint stresses may be necessary and ideally the next pour should be carried out on the following day. Construction joint with intermediate stressing (Figure 15b) On completion of the first pour containing embedded bearing plates, intermediate anchorages or couplers are fixed to allow the tendons to be stressed. After casting of the adjacent pour, the remainder of the tendon is stressed. It is sometimes necessary to leave a pocket around the intermediate anchorage to allow the wedges that anchor the tendons during the first stage of stressing to move during the second stage of stressing. This option is most suitable for use with unbonded tendons. Infill or closure strips (Figure 15c) The slabs on either side of the strip are poured and stressed, and the strip is infilled after allowing time for temperature stresses to dissipate and some shrinkage and creep to take place.				Pour size/joints Large pour areas are possible in post-tensioned slabs, and the application of an early initial prestress, at a concrete strength of typically 12.5 N/mm², can help to control restraint stresses. There are economical limits on the length of tendons used in a slab. Typically these are 35m for tendons stressed from one end only and 70m for tendons stressed from both ends. The slab can be divided into appropriate areas by the use of stop ends and, where necessary, bearing plates are positioned over the unbonded tendons to allow for intermediate stressing.		
				FIGURE 15: CONSTRUCTION JOINTS a) No intermediate stressing b) Intermediate stressing (unbonded tendons) c) Infill or closure strip		
TYPICAL CONSTRUCTION JOINT DETAIL <i>Figure 6 – 'Stitched' construction joint detail</i>						

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																BS8110	
Standard Design Concepts (Demolition and Alterations)																	
Demolition There is only a very small additional risk associated with the demolition of a post-tensioned structure. The demolition methods are similar to those used for reinforced concrete (RC) structures, with some modifications as noted below. Prestressing tendons are made of extremely tough high-strength steel and are therefore difficult to sever. In contrast, separating the steel and concrete is slightly simpler than for RC structures because there is less steel. A bonded slab should not require any significant changes of approach to an RC slab. If percussion methods are used, the breaking up of the concrete around the ducts will release the prestressing forces locally in the same way as tension is released from reinforcement in an RC slab. Using cutting methods will have a similar effect. For unbonded slabs, the approach is often to prop the floor and then release the tension in the tendons by either: • Heating the wedges until the tendon slip occurs • Breaking out the concrete behind the anchorage until detensioning occurs • De-tensioning the tendon, using jacks • Cutting through the strands at high points, whilst protecting around anchorages. It has been shown by testing and from experiences on-site that anchorages and/or dry packing are not ejected from the slab edge at high velocity. This is due to the friction between strand and the sheath which dissipates. More detailed guidance can be found in <i>Demolition and hole cutting in post tensioned concrete buildings [13]</i>. Demolition of transfer structures should be treated with due consideration. The forces involved are significantly higher than for a single floor slab and the prestressing forces may have been increased as additional floors were constructed. Provided the demolition method takes account of these issues, the risks can be identified and managed.									Alterations As with demolition, structural alterations are no more difficult than for other construction forms, and can be easier to adapt. This means that the benefits of existing post-tensioned floor construction can be used when altering existing buildings (e.g. redundant office space being reused for residential accommodation). When it comes to minor alterations, PT slabs are often easier to work with than other structural forms. They derive their tensile strength from high strength steel tendons which are often spaced at well over 1m centres. Depending on the specific circumstances, the concrete can generally be cut out between the strands without the need for strengthening. This could potentially be an opening of 1m square, or perhaps even larger. An experienced structural engineer should always be employed to check the effects of the proposed alterations. More substantial alterations can also be undertaken using tried and tested techniques. Procedures vary slightly depending on whether the PT slab has bonded or unbonded tendons. Currently, bonded tendons are used for the vast majority of new PT construction in the UK. In this system the steel strand is bonded via the grout and duct to the concrete, so that any cut through the tendon has a local effect only. At a bond length away the tensile strength is unaffected. A typical procedure for bonded tendons would be as follows: 1 Mark the tendon positions. 2 Using appropriate equipment for the type and size of project, demolish the concrete between tendons, taking care to avoid damage to the tendons. 3 Remove the concrete, leaving the tendons. 4 Cut the tendons to length for the new layout. 5 Cast new concrete. Experience has shown there is no explosive release of energy when the concrete is broken out because the concrete is broken out in relatively small areas. For major refurbishment projects new tendons and anchorages can be installed to work in combination with the existing post-tensioning. Many of the older PT slabs in the UK were constructed using unbonded tendons, and the techniques for altering these are similar, but require slightly more planning and possibly disruption. This is because unbonded construction relies on the anchorages at either end to transmit forces between the slab and tendons so cutting the tendon releases the tension throughout its length. Therefore, before breaking out any concrete, the slab must be propped throughout the length of the strand to be cut, and then de-tensioning of the strand should be carried out. The same procedure detailed for a bonded system can then be adopted except that the severed unbonded tendons should be restressed using new anchorages cast into the edge of the opening.								
 Demolition of PT bonded slab using conventional demolition equipment. Courtesy of Freyssinet.																	

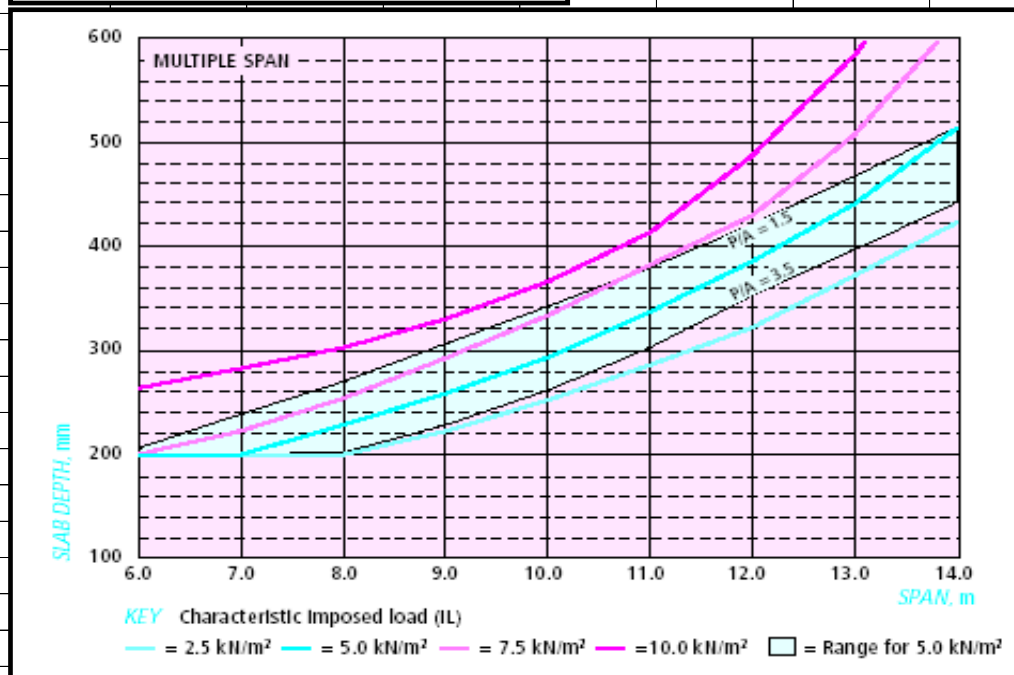
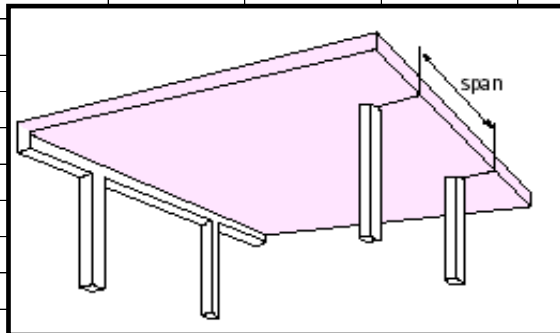
Scheme Design: PT One Way Spanning Solid Slab



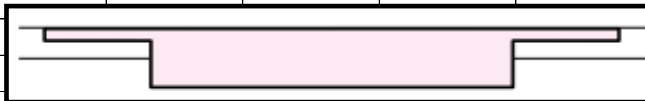
Scheme Design: PT One Way Spanning Ribbed Slab



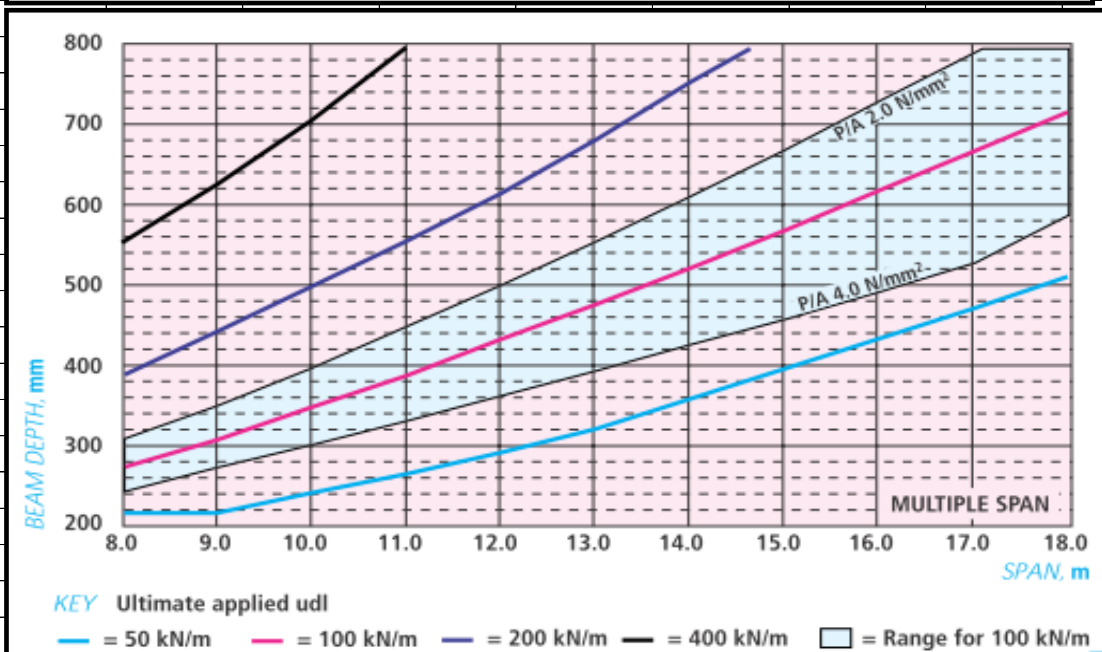
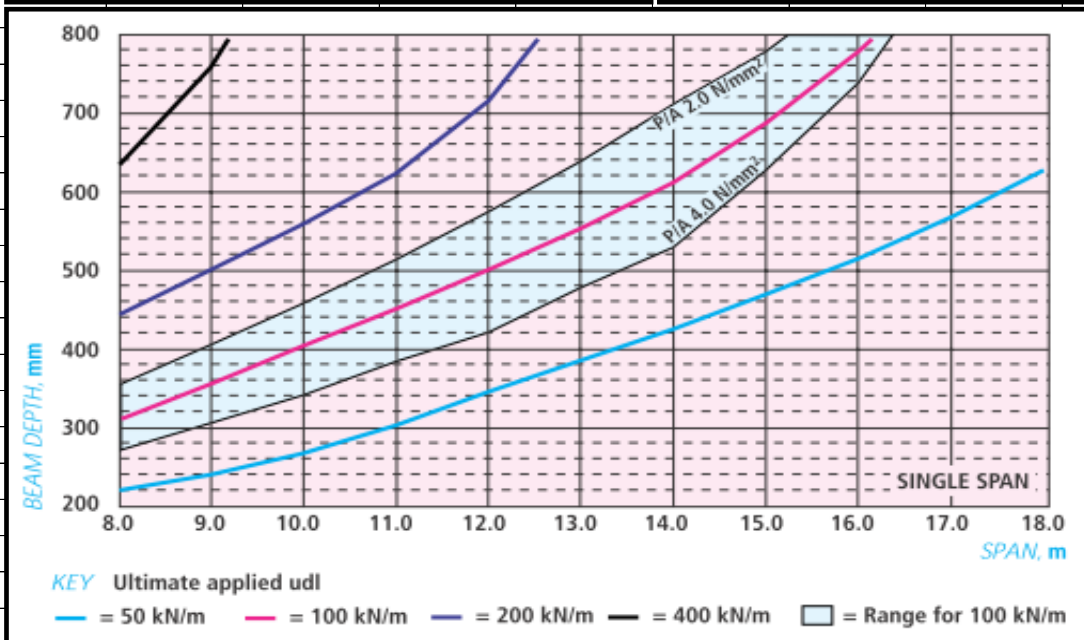
Scheme Design: PT Flat Slab With Edge Beams



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						BS8110
Scheme Design: PT Beam						
Rectangular 1000 mm wide						
800 700 600 500 400 300 200 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 BEAM DEPTH, mm SPAN, m P/A 2.0 N/mm² P/A 4.0 N/mm² SINGLE SPAN KEY Ultimate applied udl = 25 kN/m = 50 kN/m = 100 kN/m = 200 kN/m = Range for 100 kN/m						
800 700 600 500 400 300 200 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0 BEAM DEPTH, mm SPAN, m P/A 2.0 N/mm² P/A 4.0 N/mm² MULTIPLE SPAN KEY Ultimate applied udl = 25 kN/m = 50 kN/m = 100 kN/m = 200 kN/m = Range for 100 kN/m						

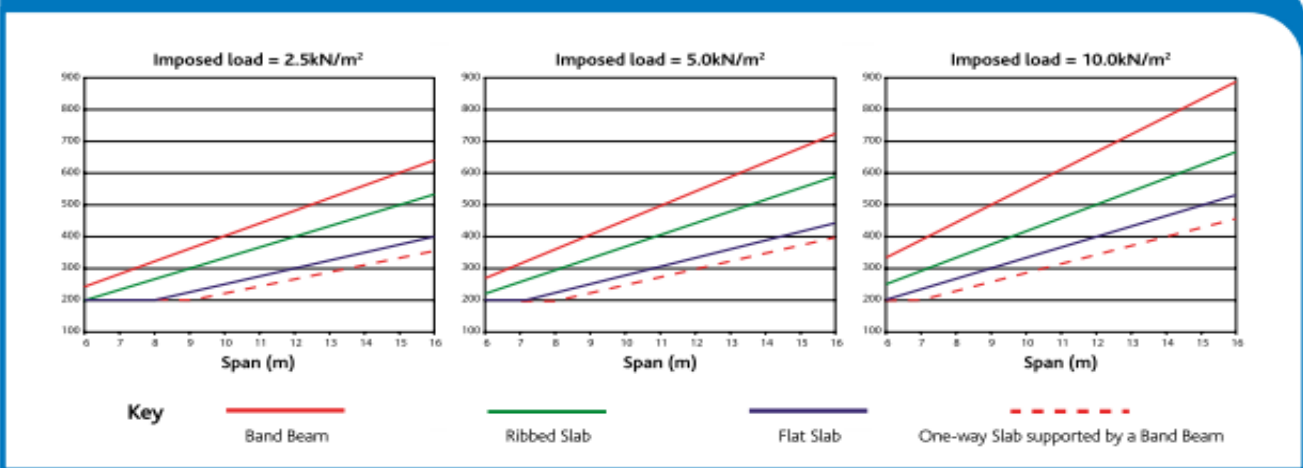


'T' beams **2400 mm** wide web



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		jXXX	77	
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				BS8110
Scheme Design: PT Beam and PT Slab				

FIGURE 6: SPAN TO DEPTH RATIOS FOR PT FLOORS



RULES OF THUMB

Advantages of using prestressed concrete

- Increased clear spans
- Thinner slabs
- Lighter structures
- Reduced cracking and deflections
- Reduced storey height
- Rapid construction
- Water tightness

Note: use of prestressed concrete does not significantly affect the ultimate limit state (except by virtue of the use of a higher grade of steel).

Maximum length of slab

50m, bonded or unbonded, stressed from both ends.
25m, bonded, stressed from one end only.

Mean prestress

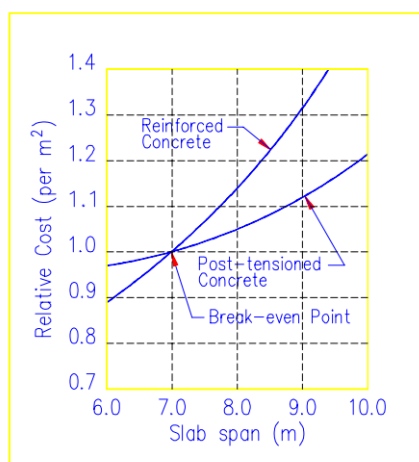
Typically: $P/A \approx 1$ to 2 N/mm^2

Cover

Take minimum cover to be 25mm.
Allow sufficient cover for (at least) nominal bending reinforcement over the columns, in both directions (typically T16 bars in each direction).

Effect of restraint to floor shortening

Post-tensioned floors must be able to shorten to enable the prestress to be applied to the floor.

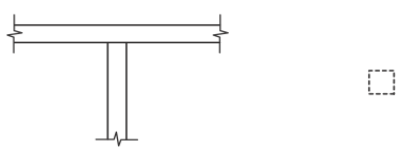
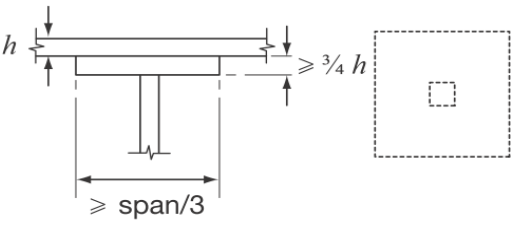
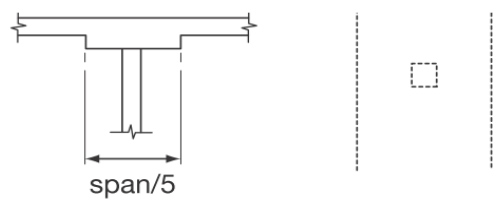
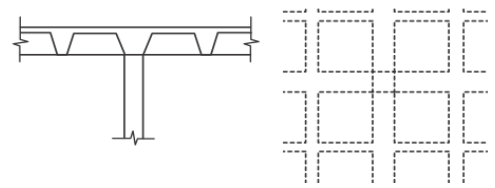
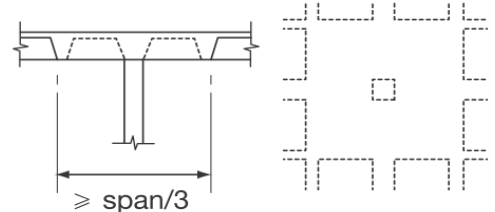


Cost comparison - Reinforced vs Post-tensioned flat slab.

Occupancy of building	Partitions and Other Superimposed Dead Load kPa	Live Load kPa	Load to Balance kPa
Car Parks	Nil	2.5	(0.7-0.85)SW
Shopping Centres	0.0 - 2.0	5.0	(0.85-1.0)SW
Residential (check transfer carefully)	2.0 - 4.0	1.5	SW + 30% of partition load
Office Buildings	0.5 - 1.0	3.0	(0.8-0.95)SW
Storage	Nil	2.4 kPa / m height	SW + 20% LL

Note: SW denotes self weight, LL denotes live load.

Table 1. General level of load to be balanced by post-tensioning tendons to give an eco structure.

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						jXXX	79			
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Table 1: Typical span/depth ratios for a variety of section types for multi-span floors.										
Section type		Total imposed load (kN/m)		Span/depth ratios $6m \leq L \leq 13m$ (kN/m)		Additional requirements for vibration				
1 Solid flat slab 		2.5 5.0 10.0		40 36 30		A				
2 Solid flat slab with drop panel 		2.5 5.0 10.0		44 40 36		A				
3 Banded flat slab 		2.5 5.0 10.0		Slab	Beam	A				
				45	25					
				40	22					
				35	18					
4 Coffered flat slab 		2.5 5.0 10.0		25 23 20		B				
5 Coffered flat slab with solid panels 		2.5 5.0 10.0		28 26 23		B				

- a Vibration. The following additional check should be made for normal office conditions if no further vibration checks are carried out:
 - A either the floor has at least four panels and is at least 250mm thick or the floor has at least eight panels and is at least 200mm thick.
 - B either the floor has at least four panels and is at least 400mm thick or the floor has at least eight panels and is at least 300mm thick.
- b All panels assumed to be square.
- c Span/depth ratios not affected by column head.
- d It may be possible that prestressed tendons will not be required in the banded sections and that untensioned reinforcement will suffice in the ribs, or vice versa.
- e The values of span/depth ratio can vary according to the width of the beam.

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		jXXX	81	
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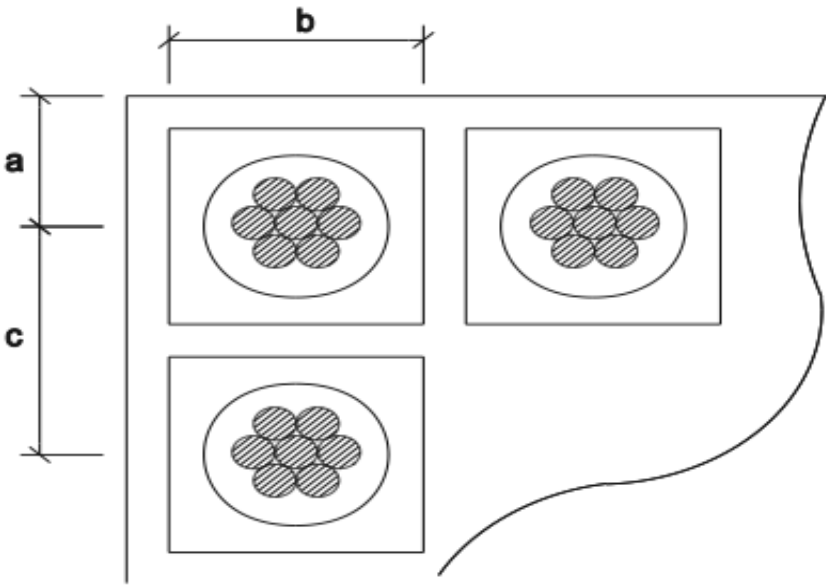
COMMON STRANDS⁴

	Nominal diameter (mm)	Steel area (mm ²)	Mass (kg/m)	Nominal tensile strength (N/mm ²)	Characteristic breaking load (kN)	Modulus of elasticity (kN/mm ² or GPa)
Standard	15.2	139	1.090	1670	232	195 ± 10
	12.5	93	0.730	1770	164	195 ± 10
	11.0	71	0.557	1770	125	195 ± 10
	9.3	52	0.408	1770	92	195 ± 10
Super	15.7	150	1.180	1770	265*	195 ± 10
	12.9	100	0.785	1860	186	195 ± 10
	11.3	75	0.590	1860	139	195 ± 10
	9.6	55	0.432	1860	102	195 ± 10
	8.0	38	0.298	1860	70	195 ± 10
Compact/ Dyform	18.0	223	1.750	1700	380	195 ± 10
	15.2	165	1.295	1820	300	195 ± 10
	12.7	112	0.890	1860	209	195 ± 10

* 279 also available, details not yet published

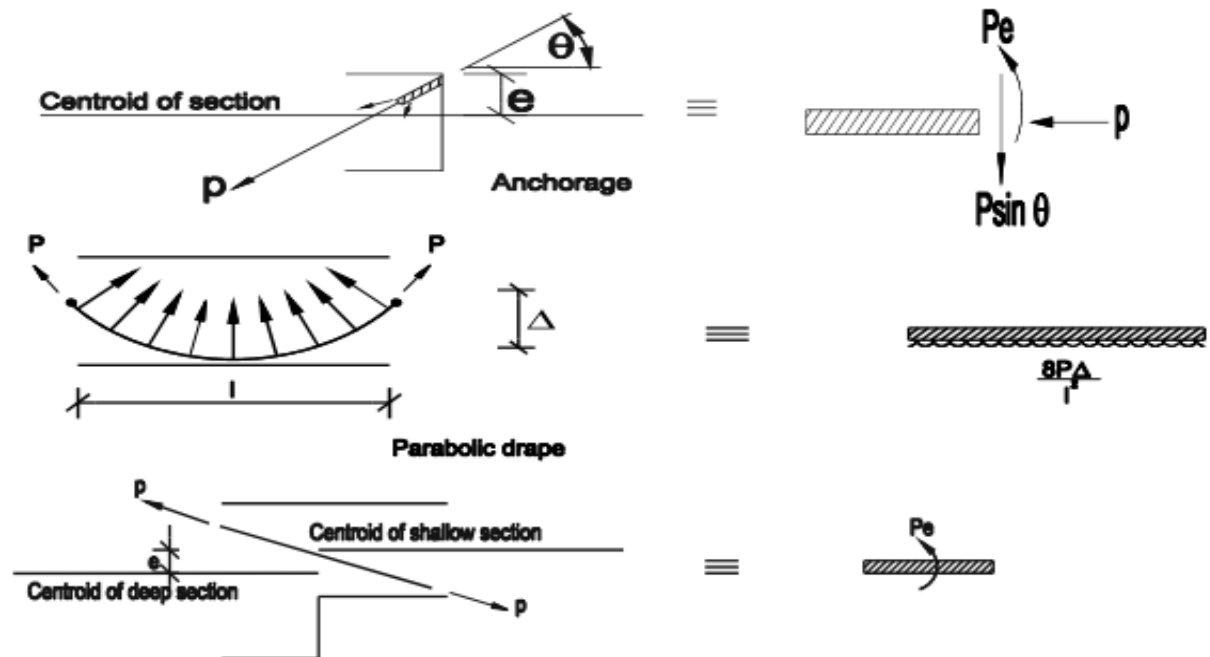
COMMON TENDONS¹

No. strands per duct for 15.7mm "super" strand	70% UTS (kN)	Internal sheath (mm)	Anchor sizes			Jack		
			a	b	c	Length (mm)	ϕ (mm)	Stroke (mm)
1	186	25						
7	1299	65	175	210	270	630	350	150
12	2226	75	200	245	300	750	390	250
15	2783	85				750	390	250
19	3525	95	250	315	375	900	510	250
27	5009	110	300	365	450	950	610	250
37	6864	130	375	450	525	1000	720	250



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4.3.4 EQUIVALENT LOADS⁶

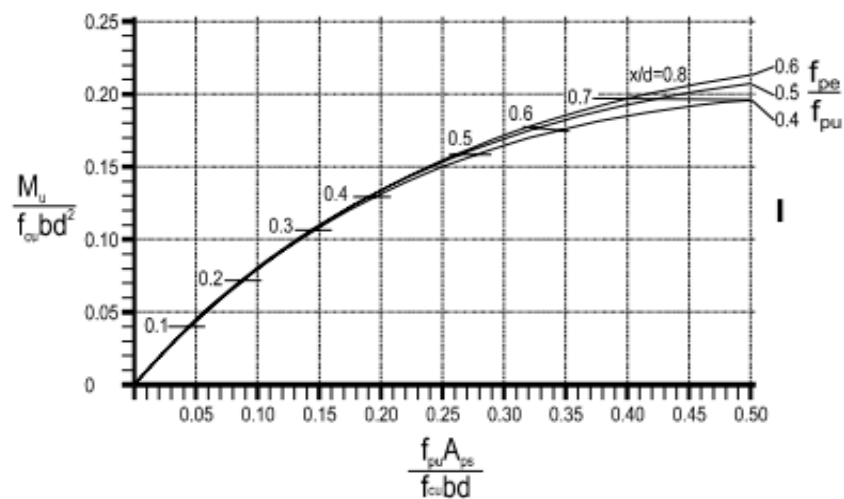


ALLOWABLE STRESSES AT SERVICE LOADS

	In service	At transfer
Compression	beams: $0.33f_{cu}$ ($0.4f_{cu}$ at supports for indeterminate beams) columns: $0.25f_{cu}$	bending: $0.5f_{cu}$ compression: $0.4f_{cu}$
Tension	Class 1: No tension Class 2: 2N/mm^2 post-tensioned 3N/mm^2 pre-tensioned Class 3: See BS 8110	1.0 N/mm^2 $0.45 \sqrt{f_{cu}}$ $0.36 \sqrt{f_{cu}}$

ULTIMATE BENDING STRENGTH⁶

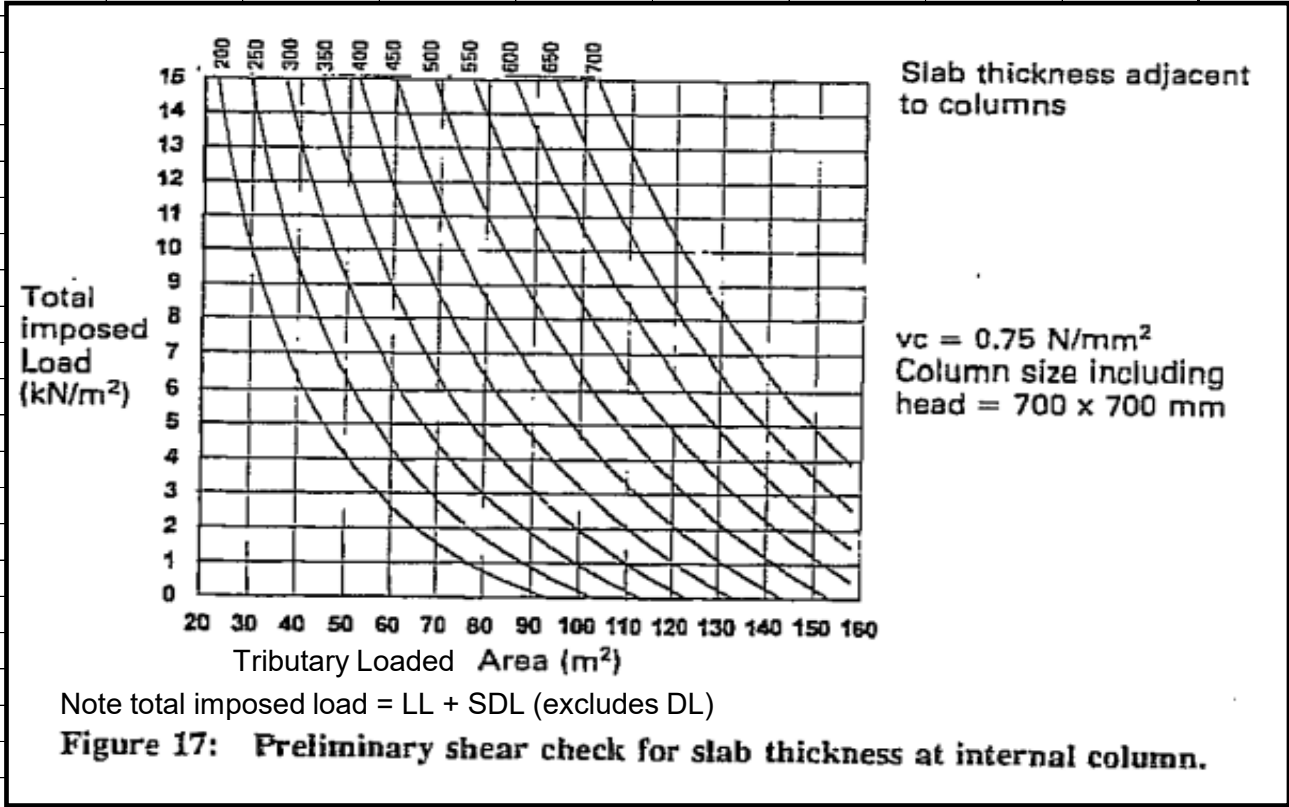
For rectangular beams or T beams with neutral axis in flange:



1. $f_{cu} = 40 \text{ N/mm}^2$
2. Dead load factor = 1.4
3. Live load factor = 1.6
4. The value of d/h is assumed to be 0.85
5. The ratio of V_{eff}/V is assumed to be 1.15
6. These curves do not take account of elastic distribution effects
7. The maximum shear stress for $f_{cu} = 40 \text{ N/mm}^2$ and more is 5 N/mm^2 .
 For $f_{cu} < 40 \text{ N/mm}^2$ the maximum shear stress is $0.8 \sqrt{f_{cu}}$
 For $f_{cu} = 35 \text{ N/mm}^2$ increase slab depth by a factor of 1.06
 For $f_{cu} = 30 \text{ N/mm}^2$ increase slab depth by a factor of 1.14

Figure 17: Punching Shear Check at Column Face

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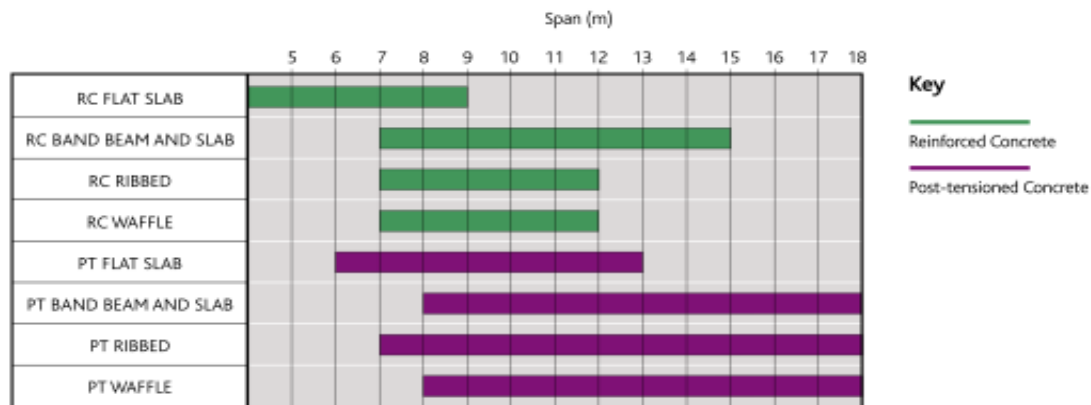


Figures 17 and 18 are set for internal columns. They may be used for external columns provided that the loaded area is doubled for edge and quadrupled for corner columns. This assumes that the edge of the slab extends to at least the centre line of the column.

Figures 17 and 18 present the required slab thickness (inclusive of slab drop and/or

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				Chd. BS8110
Typical Initial Span / Effective Depth Ratios				

FIGURE 5: TYPICAL ECONOMIC SPAN RANGES



Span-to-depth ratios for post-tensioned slabs and beams

spans are in the range 6 to 13 m.

Imposed load, Q_k (kN/m ²)	Flat slab	Flat slab with band beams		Ribbed slab	Waffle slab (with solid slab at column head)	One-way slab on deep beam	
		Slab	Beam			Slab	Beam
2.5	40	45	25	30	28	42	18
5.0	36	40	22	27	26	38	16
10.0	30	35	18	24	23	34	13

Table 7.5 Span/effective depth ratios for initial sizing of isolated beams

Cantilever	8
Simply supported	18
Continuous	22

TR.43 cl.6.14

Span to Depth Ratios

Solid Slab 42-48
Roof Slab 48-52

TABLE 4.3.2-1 Recommended Span/Depth Ratios (T124)

	Continuous spans		Simple spans	
	Roof	Floor	Roof	Floor
One-way solid slabs	50	45	45	40
Two-way solid slabs (supported on columns only)	45-48	40-45		
Beams	35	30	30	26

Note: The above ratios may be increased if calculations verify that deflection, camber, and vibrations are not objectionable.

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Flat Plate (figure 9)

This system is commonly used in Sydney for high rise residential construction where the span is usually 7 to 8 metres. The most attractive feature of this floor system is its flush soffit which requires simple formwork and greatly simplifies construction.

The depth of a flat plate is often dictated by shear requirements. Thinner slabs or longer spans can be constructed if column capitals or shear heads are employed.

Used

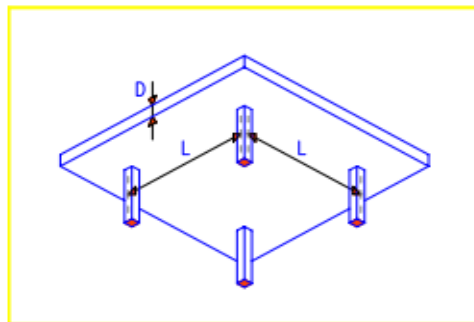
Where spans are similar both directions

Economic Span Range

7.0 to 9.0 m

Imposed Loads

Up to 7.5 kPa



	Imposed Load (kPa)	Span/Depth Ratio
Single Span	3	33
	5	31
	10	28
End Span	3	39
	5	36
	10	32
Internal Span	3	45
	5	42
	10	38

Figure 9: Flat plate

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Member Design - PC Beam and Slab

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Flat Slab (figure 10)

A widely used system today for many reasons - flat soffit, simple formwork and ease of construction, as well as flexibility for locating services.

The economical span range over a flat plate is increase by the addition of drop panels. The drop panels increase the flexural stiffness of the floor as well as improving its punching shear strength.

This system provides the thinnest floors and can lead to height reductions and substantial savings in facade costs.

Used

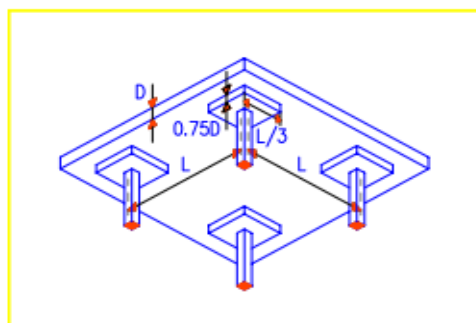
Where spans are similar both directions

Economic Span Range

Up to 13.0 m

Imposed Loads

Up to 10.0 kPa



	Imposed Load (kPa)	Span/Depth Ratio
Single Span	3	38
	5	35
	10	32
End Span	3	46
	5	43
	10	40
Internal Span	3	52
	5	49
	10	45

Figure 10. Flat slab

For structures requiring minimum floor to floor height and regular grids the two-way post-tensioned flat slab is usually the most cost effective solution.

The normal installation procedure would concentrate the tendons into 'column strips' along the column grids at approximately 600 mm centres with tendons away from the column strip at approximately 1400 mm centres.

Consequently small holes for services could be located without the need to cut tendons.

Using this structural system it is possible to leave the central panel as traditionally reinforced and designed as a 'soft zone' to easily accommodate large openings. The cost penalty for the extra reinforcement required would need to be offset against the perceived benefits.

The slabs however, are usually quite lightly prestressed with tendons in one direction only at approximately 1500 mm centres. Reasonable size openings or large slots are therefore easy to accommodate without the need to cut post-tensioning tendons.

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Member Design - PC Beam and Slab

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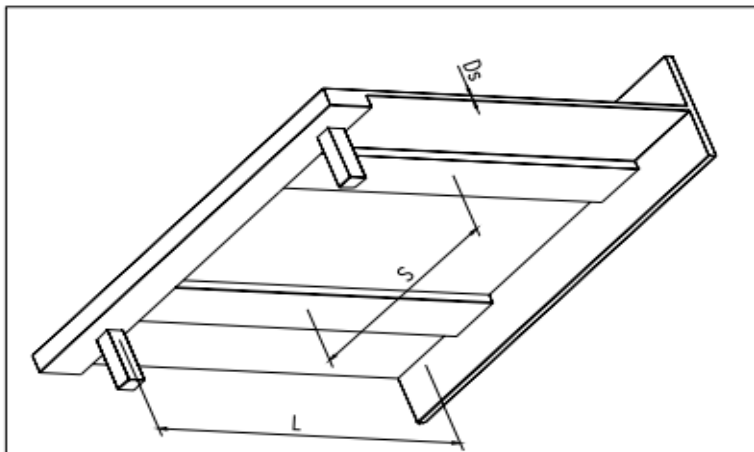
High Rise Banded Slab (figure 12)

This system has gained favour over the past 10 to 15 years for high rise construction and consists of band beams at relatively close centres spanning between a perimeter beam and the service core. The system suits system formwork due to the amount of re-use in high rise construction.

Services may either pass under the shallow bands or, alternatively, pass under service 'notches' in the band soffit.

For clear slab spans in excess of 4.5m the use of post-tensioning is economical perpendicular to the bands and assist in reducing the weight of slab carried by the bands.

Used	Long span high rise construction
Economic Span Range	Band Beam: 9.0 to 15.0 m
Imposed Loads	Up to 7.5 kPa



Slab Span	Band Width	Span / Depth Ratio
S= 4800, D _s = 120 RC	b _w = 1000	22
S= 6000, D _s = 120 P-T, 150 RC	b _w = 1500	20
S= 8400, D _s = 160 P-T	b _w = 1800	18

Figure 12. High Rise Banded Slab.

Typical Initial Span / Effective Depth Ratios

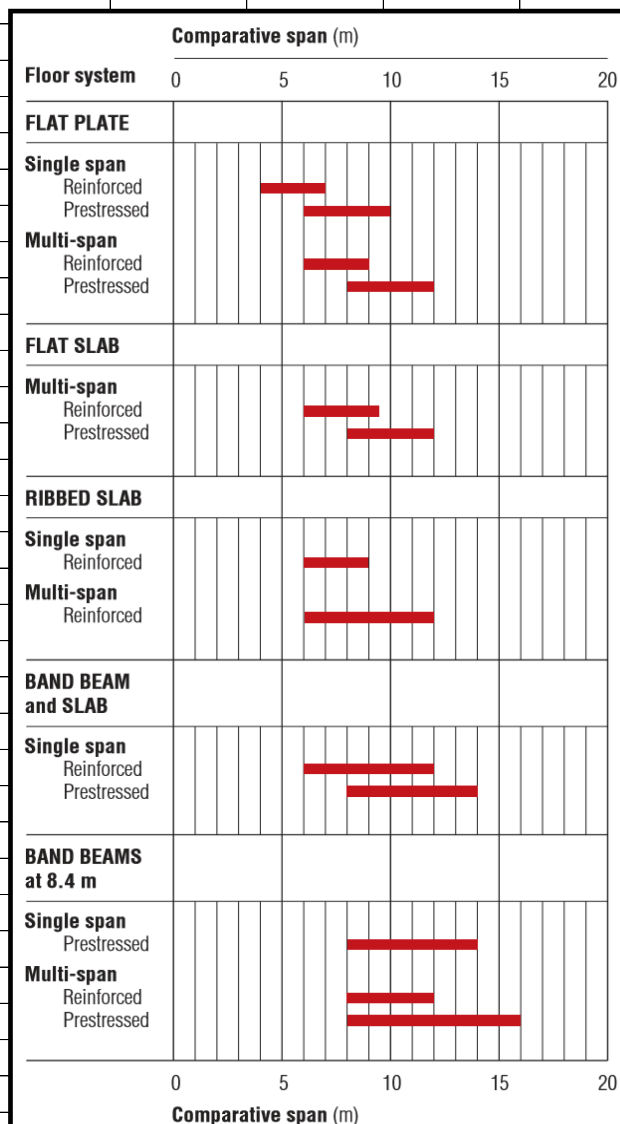


FIGURE 8: Quick selection guide for insitu floors

The span 'L' of a reinforced concrete flat-plate is approximately $D \times 28$ for simply supported, $D \times 30$ for an end span of a continuous system, to $D \times 32$ for internal continuous spans. The economical span of a flat plate can be extended by prestressing to approximately $D \times 30$, $D \times 37$ and $D \times 40$ respectively, where D is the depth of slab.

The principal features of a flat slab floor are a flat soffit, simple formwork and easy construction. The economical span 'L' of a reinforced concrete flat slab is approximately $D \times 28$ for simply supported, $D \times 32$ for an end span and $D \times 36$ for an interior span. Prestressing the slab increases the economical span to $D \times 35$, $D \times 40$ and $D \times 45$ respectively, where D is the depth of the slab excluding the drop panel.

In a single-span floor, the spacing of the band beams may coincide with the columns, or the bands may be more closely spaced to reduce the thickness of the slab spanning between walls or beams. For single-span reinforced concrete floors the economical span 'L' of the band beam is $D \times 20$ to $D \times 22$ depending on the width and spacing of the band beam, where D is the depth of the slab plus band beam. Prestressing the band beam gives economical band-beam spans

in the range of $D \times 24$ to $D \times 28$. In a multi-span floor, the spacing of the band beams is fixed by the transverse spacing of the columns.

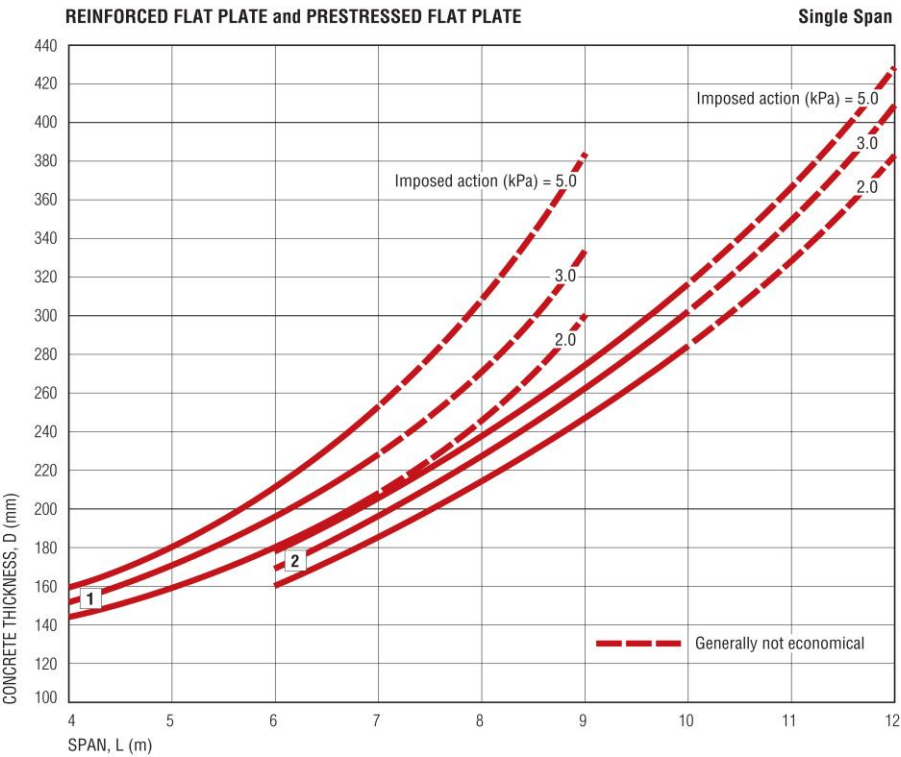
For initial sizing of the slab, the span-to-depth ratios from Section 6.3 can be used. For internal spans the slab thickness is based on the clear span between band beams, and for an external bay is from the edge of band to the column line of the external band. The depth of the band is typically 1.5 to 2 times the depth of the slab and the minimum economical span for a band beam is about 7–8 m.

In multiple spans using reinforced concrete, the economical slab of the band beam 'L' is approximately D x 22 for 1200-mm-wide band beams and D x 26 for a 2400-mm-wide beams at 8400-mm centres. Prestressing increases the economical span 'L' to D x 24 to D x 28 for similar beam widths. D is the depth of slab plus band beam in each case.

The maximum span for reinforced concrete bands should not normally exceed 12 m. Above this span, bands should be prestressed. The slab band width should be between band-spacing/3 to band-spacing/4 and, where possible, should be based on a module of a standard sheet of ply of 2.4 m x 1.2 m.

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FLAT PLATE

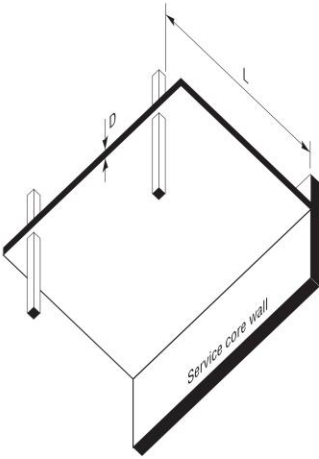


CURVES

- 1 Reinforced
- 2 Prestressed

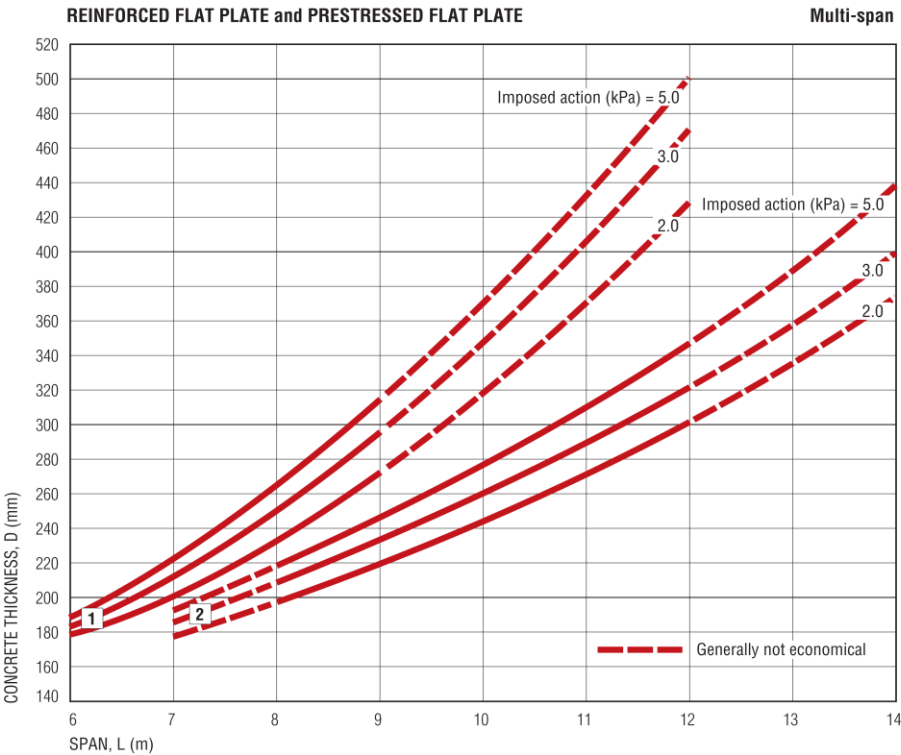
NOTES:

- For preliminary design and initial sizing only.
- Imposed action of 2.0 kPa typical for domestic etc, 3.0 kPa typical for offices etc and 5.0 kPa typical for assembly areas without fixed seating
- A 120/120/120 Fire Resistance Level assumed
- The following additional permanent actions (dead loads) are included with the imposed actions (live load) when preparing this chart:
 - For imposed action of 2.0 kPa an additional permanent action of 0.5 kPa has been allowed
 - For imposed actions of 3.0 kPa and 5.0 kPa an additional permanent action of 1.5 kPa has been allowed
- Full continuity at the core wall assumed
- Unsupported edges of floors may require stiffening for support of external walls and/or visual reasons
- For larger spans, shear heads or reinforcement may be required at the columns
- Deflection to be the lesser of span/250 or 35 mm



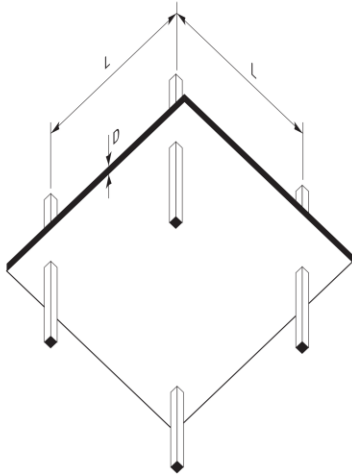
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		Member/Location		
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FLAT PLATE



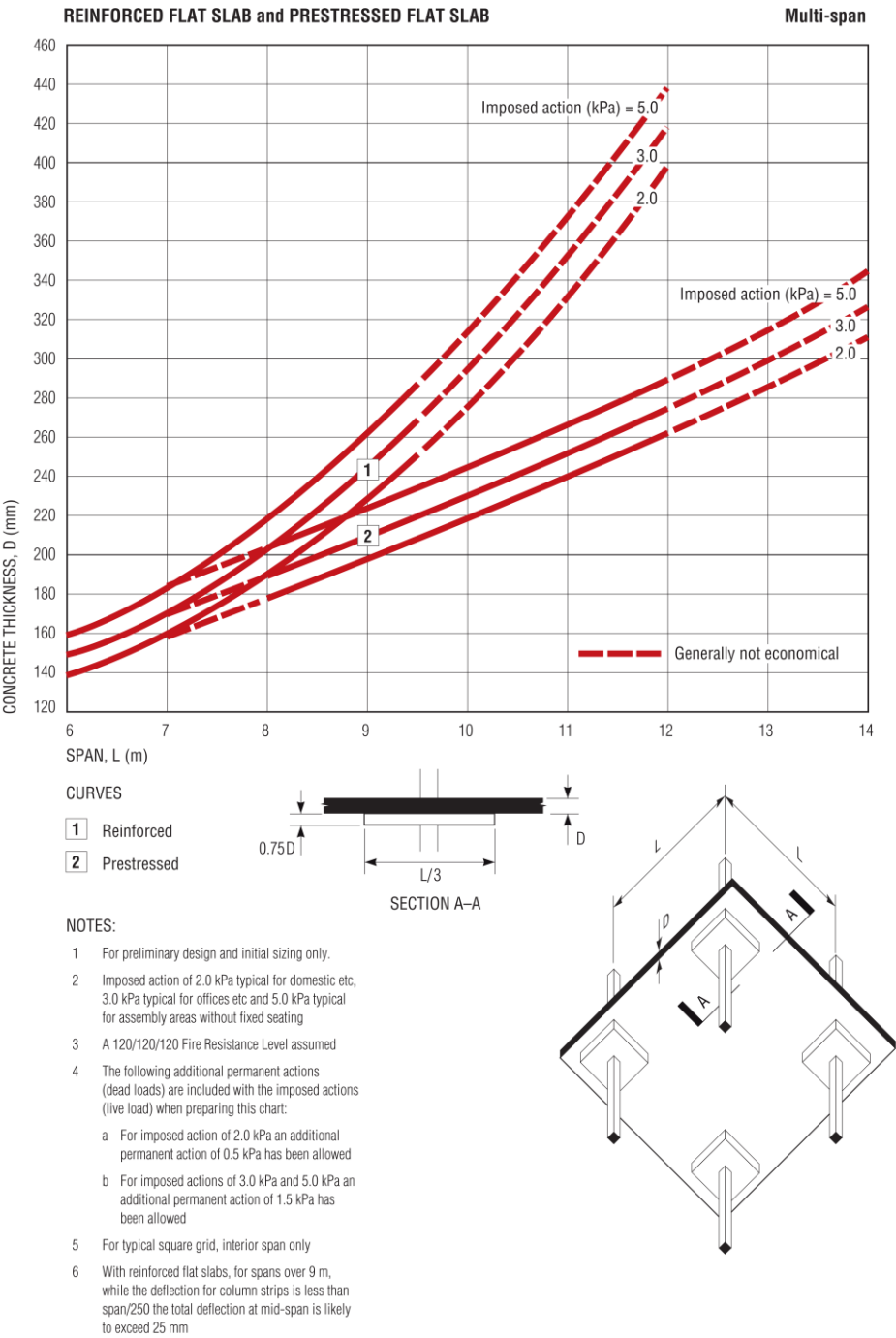
- CURVES
- 1 Reinforced
 - 2 Prestressed

- NOTES:
- For preliminary design and initial sizing only.
 - Imposed action of 2.0 kPa typical for domestic etc, 3.0 kPa typical for offices etc and 5.0 kPa typical for assembly areas without fixed seating
 - A 120/120/120 Fire Resistance Level assumed
 - The following additional permanent actions (dead loads) are included with the imposed actions (live load) when preparing this chart:
 - a For imposed action of 2.0 kPa an additional permanent action of 0.5 kPa has been allowed
 - b For imposed actions of 3.0 kPa and 5.0 kPa an additional permanent action of 1.5 kPa has been allowed
 - For typical square grid, interior span only
 - Deflection to be the lesser of span/250 or 35 mm

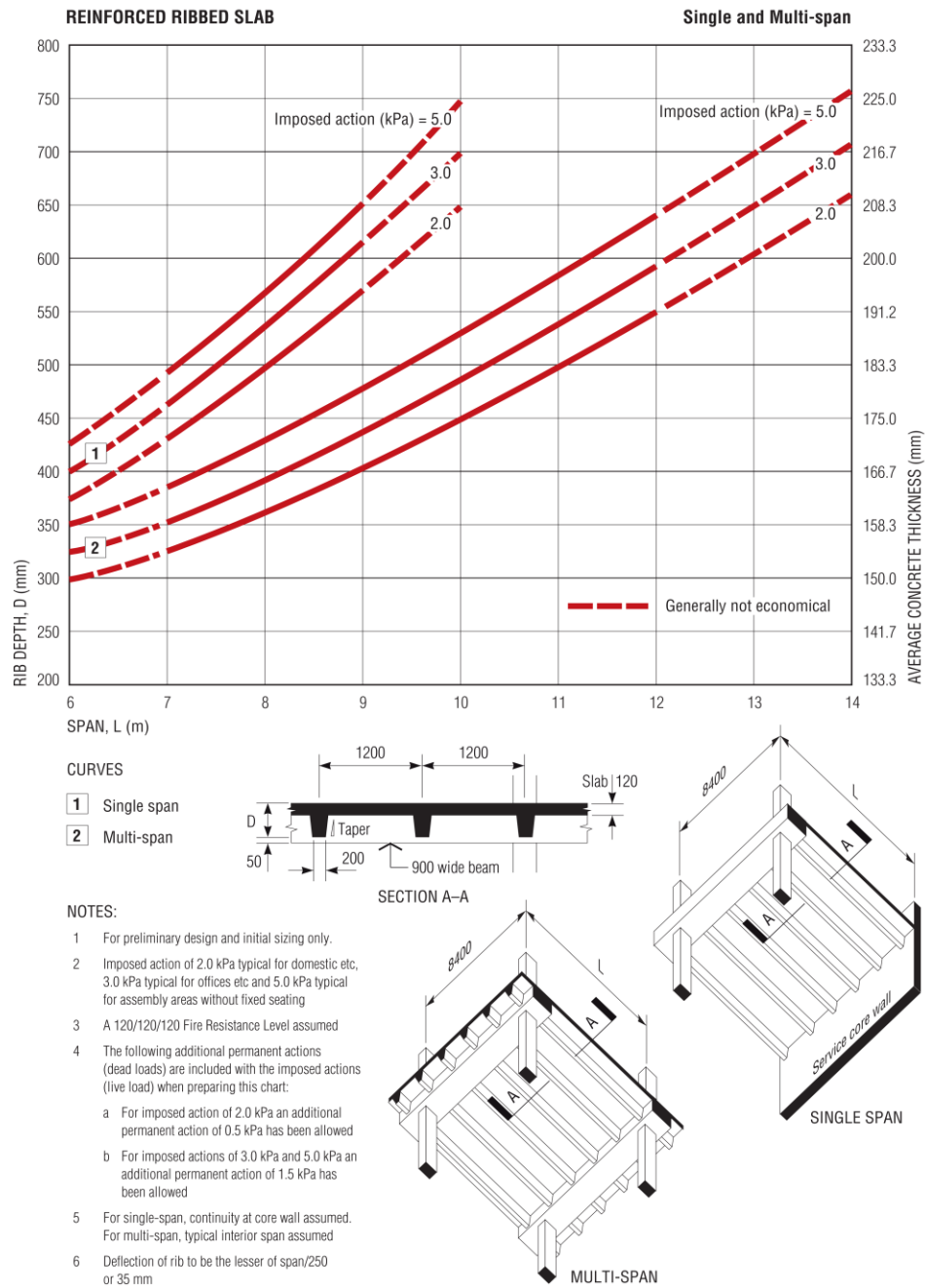


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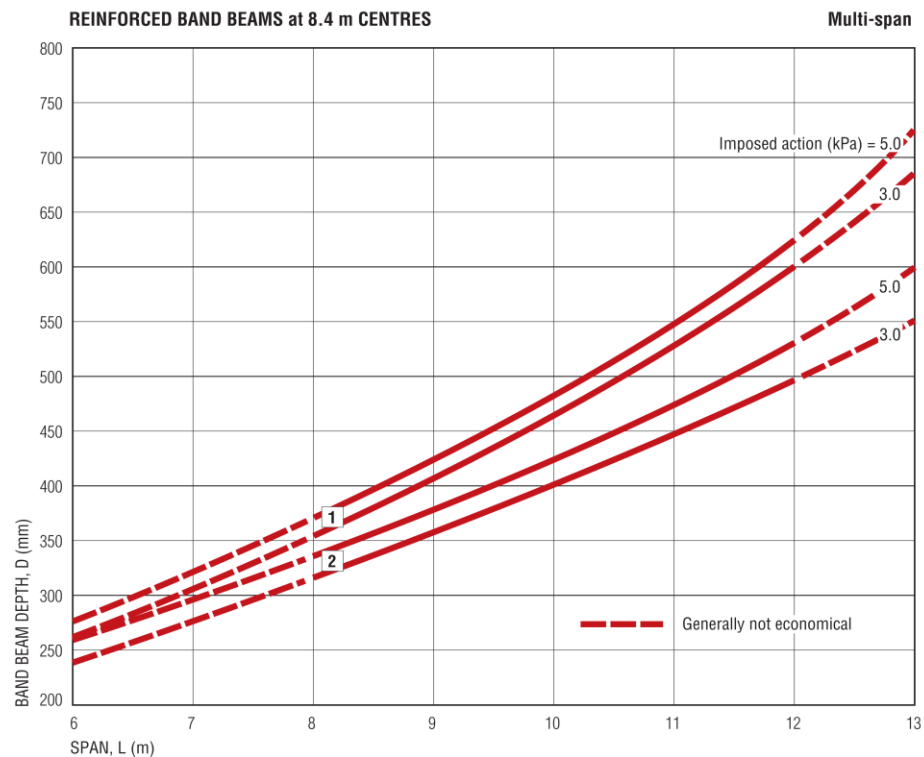
FLAT SLAB



RIBBED [WAFFLE] SLAB



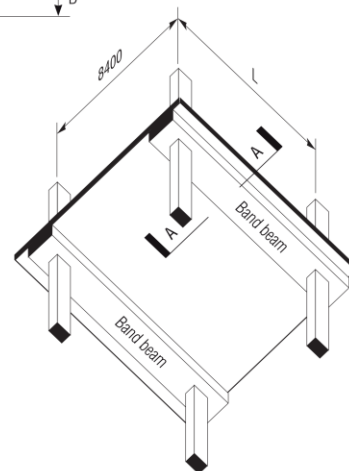
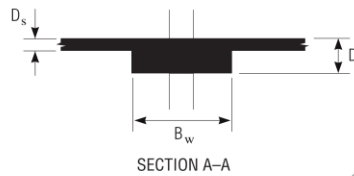
BAND BEAM AND SLAB



CURVES		B_w	D_s
1	Reinforced,	1200	200
2	Reinforced,	2400	170

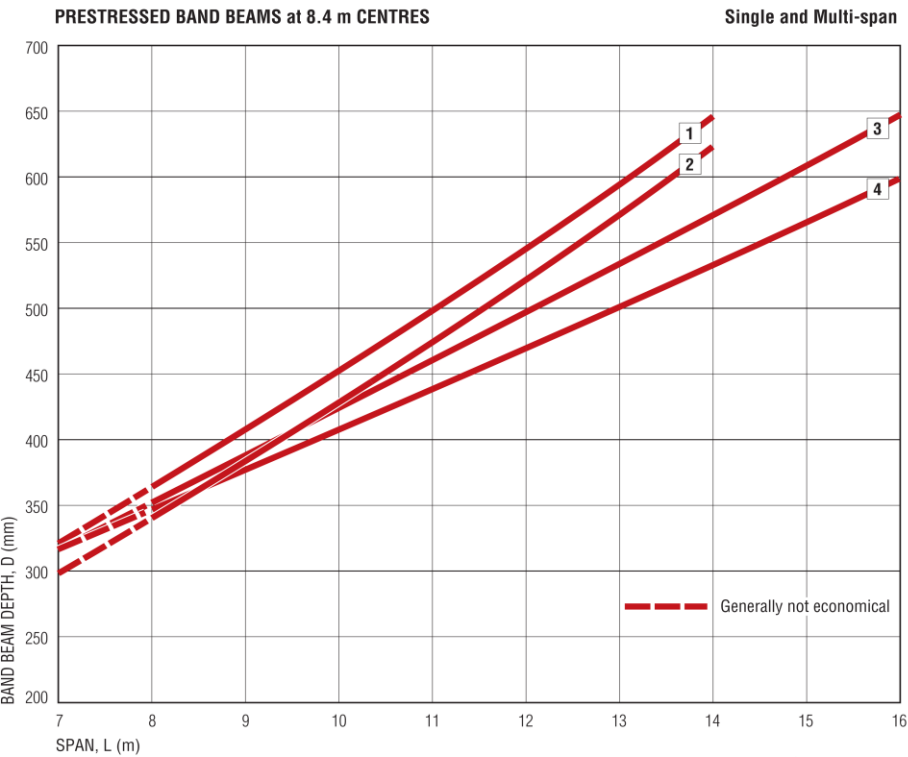
NOTES:

- 1 For preliminary design and initial sizing only.
- 2 Imposed action of 3.0 kPa typical for offices etc and 5.0 kPa typical for assembly areas without fixed seating
- 3 A 120/120/120 Fire Resistance Level assumed
- 4 For imposed actions of 3.0 kPa and 5.0 kPa, an additional permanent action (dead load) of 1.5 kPa has been included with the imposed actions (live load) when preparing this chart
- 5 Typical interior span assumed
- 6 Deflection of band beam to be the lesser of span/250 or 35 mm



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						Member/Location				
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BAND BEAM AND SLAB



CURVES	B_w	D_s
1	Single span, 1800	160
2	Single span, 2400	150
3	Multi-span, 1800	160
4	Multi-span, 2400	150

- NOTES:
- For preliminary design and initial sizing only.
 - Curves are for an imposed action of 5.0 kPa (typical for assembly areas without fixed seating) to which has been added a permanent action (dead load) of 1.5 kPa
 - A 120/120/120 Fire Resistance Level assumed
 - For single span, continuity at core wall assumed. For multi-span, typical interior span assumed
 - Deflection of band beam to be the lesser of span/250 or 35 mm

