CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
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Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	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} ≥ 40N/mm^2 (pre-T) or 35N/mm^2 (post-T) cl.4.1.8.1 BS8110, usually 40N/mm^2, ≤ 105N/mn								
Characteristic strength of concrete at transfer (PT beam and slab),					25 ▼	20 ▼	N/mm^2	**OK**
Note require f_{ci} ≥ 25N/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} ≤ 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/m3 ▼		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, $E_{uncracked,28}$ = 20kN/mm^2 + 0.2f_{cu}				100%	**27.0**	GPa	*7.2 BS8110*
	Uncracked long term (creep), $E_{uncracked,28,cp}$ = C_{MF}.$E_{uncracked,28}$					**9.0**	GPa	
	Cracked, E_{ck} = $E_{uncracked,28}$. [0.5-1.0 beam, 0.5-1.0 slab]				0% Crack ▼	**27.0**	GPa	*.8.3 BS811*
	Cracked long term (creep), $E_{ck,cp}$ = $E_{uncracked,28,cp}$. [0.5-1.					**9.0**	GPa	*.8.3 BS811*
Column Elastic Modulus	Uncracked, $E_{uncracked,28}$ = 20kN/mm^2 + 0.2f_{cu}				100%	**28.0**	GPa	*7.2 BS8110*
	Uncracked long term (creep), $E_{uncracked,28,cp}$ = C_{MF}.$E_{uncracked,28}$					**9.3**	GPa	
	Cracked, E_{ck} = $E_{uncracked,28}$. [0.5-1.0 column]				0% Crack ▼	**28.0**	GPa	
	Cracked long term (creep), $E_{ck,cp}$ = $E_{uncracked,28,cp}$. [0.5-1.					**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					**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 concentratior			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 (*		<=0.2mm 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} hog, 0.33f_{ci} sag} for FTW-FS-DS*						*cl.4.3.5.1*
TLS permissible tens σ, f'_{min}		**-1.25**	**-1.25**	√f_{ci} / √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 √f_{ci} (pre-T), −0.36 √f_{ci} (post-T)*				-1.8	N/mm^2	*cl.4.3.5.2*
	Class 3	*f'_{min} = −0.25f_{ci} or −0.45 √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} hog, 0.33f*						*cl.4.3.4.2*
SLS permissible tens σ, f_{min}		**-0.45**	**-0.45**	√f_{cu} / √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	*$f_{min,fcu ≤60N/mm2}$ = −0.45 √f_{cu} (pre-T), −0.36 √f_{cu} (post-*				-2.1	N/mm^2	*cl.4.3.4.3*
		$f_{min,fcu>60N/mm2}$ = −0.23 (f_{cu})$^{2/3}$ (pre-T), −0.18 (f_{cu})$^{2/3}$ (				N/A	N/mm^2	*l.8.1 TR.49*
	Class 3	*$f_{min,fcu<60N/mm2}$ = MAX{ −0.25f_{cu}, −f (T.4.2, T.*			-2.7	-2.7	N/mm^2	*cl.4.3.4.3*
		$f_{min,fcu ≥60N/mm2}$ = MAX{ −0.25f_{cu}, −f (T.9) −[4			N/A	N/A	N/mm^2	*l.8.1 TR.49*
Note by convention, positive stress is compressive and negative st					**Top**	**Bottom**		
Note for flat slabs, if the full tributary width flat slab design strip (FTW-FS-DS) is employed, then to								*cl.6.10.1 TR.4*
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}]={0.24$f_{ci/cu}$ hog, 0.33$f_{ci/cu}$ sag}*							*T.2 TR.43*
	BD: permissible tensile stress [f'_{min},f_{min}]={ −0.45 √$f_{ci/cu}$ hog, −0.45 √$f_{ci/cu}$ sag}							*T.2 TR.43*
(ii)	*Un-BD: permissible compressive stress [f'_{max},f_{max}]={0.24$f_{ci/cu}$ hog, 0.33$f_{ci/cu}$ sag}*							*T.2 TR.43*
	Un-BD: permissible tensile stress [f'_{min},f_{min}]={ −0.45 √$f_{ci/cu}$ hog, −0.45 √$f_{ci/cu}$ sag} assu							*T.2 TR.43*

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Section Dimensions BS8110 ▼

Span, L (usually ≥ 7.0m s/s or cont and ≥ 3.5m cant *cl.3.1 TR.43*) 10.000 m **OK**

Available beam spacing 5.000 m

(effective width calcs, section properties flanged beam; usual spacing for interior beams; half for edge beams)

Section type at TLS T - Section ▼ **OK**

Section type at (SLS/ULS) T - Section ▼ **OK**

(section type for section properties, bending calcs)

Support condition (and continuous end span moment ?) N/A ▼ Continuous ▼ **Note**

(support condition for LTB restraint, effective width, prestress [x=0] Continues ▼

force losses, action effects, deflection, longitudinal shear calcs) [x=L] Continues ▼

Design section hogging or sagging moment ? Hogging Moment ▼ **OK**

(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)

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 ≤ 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 ≈ L/25+100mm (L ≤ 36m), h ≈ L/20 (L > 36m); Note cant h ≈ L/8; *MOSLEY*

Note usually h ≈ 70% of equivalent non-prestressed member; *cl.6.4 TR.43*

Depth of flange, h_f 200 mm

(section properties flanged beam, bending flanged beam, longitudinal shear calcs)

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_{hface} 0 mm

Column head depth, d_h 0 mm

43 Column head actual depth (rect.), $l_{h0,h}$ = h + (1 or 2).l_{hface} or actua 800 mm

: - Column head actual width (rect.), $l_{h0,b}$ = b + (1 or 2).l_{hface} or N/A (800 mm

Column head max. depth (rect.), $l_{hmax,h}$ = h + 2.(d_h-40) or max. di 720 mm

Column head max. width (rect.), $l_{hmax,b}$ = b + 2.(d_h-40) or N/A (cir 720 mm

Column head eff. depth (rect.), $l_{h,h}$ = MIN ($l_{h0,h}$, $l_{hmax,h}$) or eff. dia. (circ.), $l_{h,D}$ 800 mm

Column head eff. width (rect.), $l_{h,b}$ = MIN ($l_{h0,b}$, $l_{hmax,b}$) or N/A (circ.) 800 mm

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Type of Construction

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Type of construction: Type IV - Insitu Beam ▼

Input Item	**Type I** **Insitu Slab**	**Type II** **Insitu Transfer Slab**	**Type III** **Precast Bridge Beam**	**Type IV** **Insitu Beam**	**Type V** **Insitu Transfer Beam**
Concrete grade (cube) at TLS and (SLS/ULS)	≥25MPa ≥35MPa	≥25MPa ≥35MPa	≥35MPa ≥35MPa	≥25MPa ≥35MPa	≥25MPa ≥35MPa
Section type at TLS and (SLS/ULS)	Rect or T / L	Rect or T / L	Rect	T / L	T / L
Creep modulus factor, C_{MF}	Normal Loading	Storage Loading	Normal Loading	Normal Loading	Storage Loading
Banding of prestress tendons and/or longitudinal steel (hogging and sagging)	Banded Flat Slab	Banded Flat Slab	Not Banded	Not Banded	Not Banded
Dead load, DL (on plan), DL_h	- or DL_h	$DL_{v,STG(i-1)}$ or DL_h +	-	DL_h	DL_h + $DL_{v,STG(i-1)}$
Superimposed dead load, SDL (on plan), SDL_h	SDL_h	SDL_h + $SDL_{v,STG(i-}$	DL_h+SDL_h	SDL_h	SDL_h + $SDL_{v,STG(i-}$
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	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

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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		$h_{C1} \approx h_{C2}/3$	h_{C2}	h_{C2}
Tendons		$[N_T \times N_s]_{C1}$	$[N_T \times N_s]_{C2,STG(1)}$	$\Sigma[N_T \times N_s]_{STG(1,2,3..)}$
Tendon profile		Within h_{C1}	Within h_{C2}	Within h_{C2}
Additional bottom compressive stress		$0N/mm^2$	$\geq 0N/mm^2$	$\geq 0N/mm^2$
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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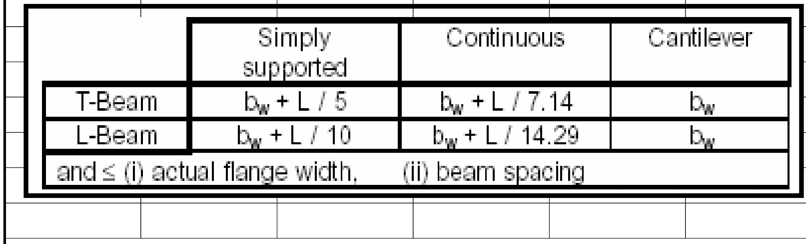
Section Properties BS8110 ▼



	Simply supported	Continuous	Cantilever
T-Beam	$b_w + L / 5$	$b_w + L / 7.14$	b_w
L-Beam	$b_w + L / 10$	$b_w + L / 14.29$	b_w

and ≤ (i) actual flange width, (ii) beam spacing

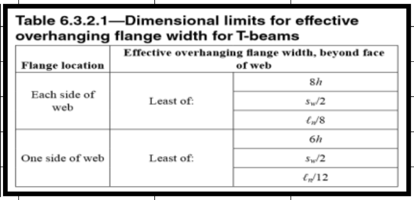


Table 6.3.2.1—Dimensional limits for effective overhanging flange width for T-beams

Flange location	Effective overhanging flange width, beyond face of web	
Each side of web	Least of:	$8h$
		$s_w/2$
		$\ell_n/8$
One side of web	Least of:	$6h$
		$s_w/2$
		$\ell_n/12$

			TLS	SLS/ULS)		
Effective width, b = MIN(b_w + function (span, section, structure), b			1901	1901	mm	*cl.3.4.1.5*

(section properties flanged beam, bending flanged beam)

	TLS	(SLS/ULS)	TLS	SLS/ULS)	
Beam area, $A_{TLS/(SLS/ULS)}$	100%	100%	**14000**	**14000**	cm^2
Rectangular section, $A_{TLS/(SLS/ULS)} = b_w.h$			N/A	N/A	cm^2
T-section, $A_{TLS/(SLS/ULS)} = h.b_w+((beam spacing)-b_w).h$			14000	14000	cm^2
L-section, $A_{TLS/(SLS/ULS)} = (beam spacing).h_f+(h-h_f).b_w$			N/A	N/A	cm^2
Beam centroid, $x_{c,TLS/(SLS/ULS)}$	100%	100%	**356**	**356**	mm
Note that the centroid, $x_{c,TLS/(SLS/ULS)}$ is measured from the top face of the beam section;					
Rectangular section, $x_{c,TLS/(SLS/ULS)} = h/2$			N/A	N/A	mm
T-section, $x_{c,TLS/(SLS/ULS)} = h-((b.h_f).(h-h_f/2)+(h-h_f)^2/$			356	356	mm
L-section, $x_{c,TLS/(SLS/ULS)} = h-((b.h_f).(h-h_f/2)+(h-h_f)^2/$			N/A	N/A	mm
Beam second moment of area, $I_{TLS/(SLS/ULS)}$	100%	100%	**713**	**713**	$x10^4cm^4$
Rectangular section, $I_{TLS/(SLS/ULS)} = b_w.h^3/12$			N/A	N/A	$x10^4cm^4$
T-section, $I_{TLS/(SLS/ULS)} = (b.(h_f)^3+b_w.(h-h_f)^3)/12+b.$			713	713	$x10^4cm^4$
L-section, $I_{TLS/(SLS/ULS)} = 1/12.b_w.h^3+b_w.h.((h-x_{c,TLS/(S}$			N/A	N/A	$x10^4cm^4$

Note that for simplicity, for all classes, the modification to section properties that affects both stress estimations (adversely in E/E and favourably in E/L computations) and deflections not performed herewith;

Note that for Class 3, although cracking is allowed it is assumed that the section is uncracked; cl.4.3.4.3 BS8

Note that for Class C, stresses at SLS shall be calculated using the cracked transformed section; cl.24.5.2.3 ACI

Note however that modifications to the elastic modulus E as the method to account for cracked deflections indeed performed herewith in the deflection calculation subsection;

	TLS	SLS/ULS)		
Beam top elastic section modulus, $Z_{t,TLS/(SLS/ULS)} = I_{TLS/(SLS/ULS)} / x_{c,TL}$	**200**	**200**	$x10^3cm^3$	
Min beam top elastic section modulus at design section, $Z_{t,TLS/(SLS/U}$	**-68**	**-68**	$x10^3cm^3$	
Min beam top elastic section modulus at design section, $Z_{t,TLS/(SLS/U}$	**91**	**91**	$x10^3cm^3$	
Min beam top elastic section modulus at design section utilisation	**45%**	**45%**		**OK**
Beam bottom elastic section modulus, $Z_{b,TLS/(SLS/ULS)} = I_{TLS/(SLS/ULS)} /$	**111**	**111**	$x10^3cm^3$	
Min beam bottom elastic section modulus at design section, $Z_{b,TLS/($	**-91**	**-91**	$x10^3cm^3$	
Min beam bottom elastic section modulus at design section, $Z_{b,TLS/($	**60**	**60**	$x10^3cm^3$	
Min beam bottom elastic section modulus at design section utilisati	**54%**	**54%**		**OK**

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 contrary to bending effects, there is no effective width for axial prestress effects as the entire width of the section (to the limit of the beam spacing) becomes mobilised (Aalami, 2014).

Note that furthermore, if the FE analysis method is employed (as opposed to the equivalent frame method), the combined bending and axial stresses are calculated without the necessity to use the effective width concept as long as the FE analysis formulation correctly models the offset of the slab with respect to the centroid of the beam (Aalami, 2014). The difference lies in the fact that the equivalent frame method calculates the stresses from the axial forces and bending moments (utilising the section area and effective width respectively), whilst the FE analysis method obtains the stresses (with an explicitly defined sectional geometry) as part of its analysis post-processing and in turn integrates them to yield the design strip axial forces and bending moments;

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Code of Practice

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

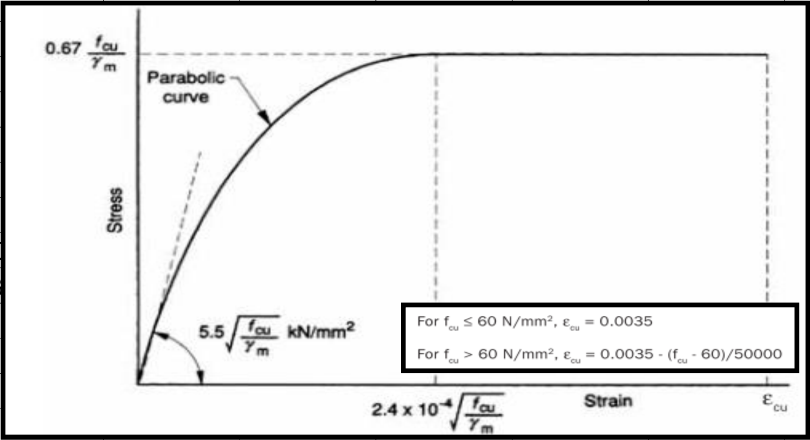
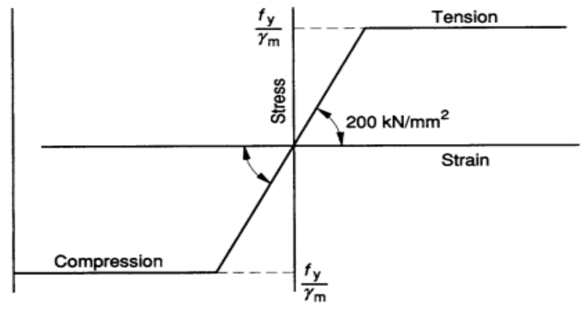
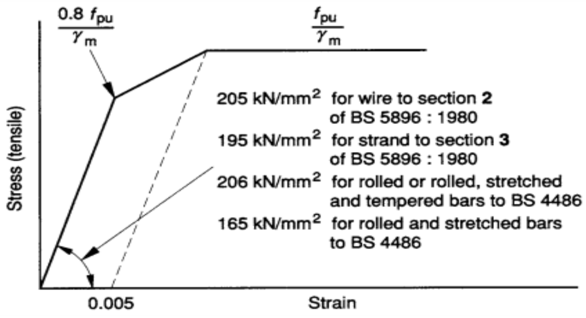


Figure 2.1 — Short term design stress-strain curve for normal-weight concrete



NOTE f_y is in N/mm².

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



NOTE f_{pu} is in N/mm².

Figure 2.3 — Short term design stress-strain curve for prestressing tendons

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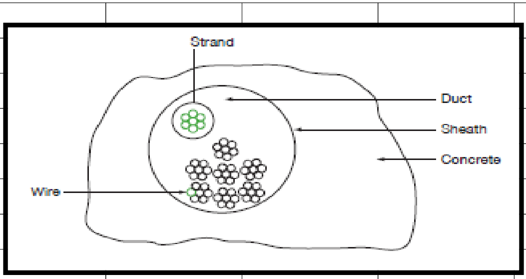
Prestress Reinforcement and Physical Tendon Profile



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	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($\phi_{link}$, $cover_{add}$)-[$D_{T,V}$+($n_{layers,PT}$-1)($D_{T,V}$+$s_{r,PT}$)]/2-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t

Bottom limit of (positive) physical eccentricity of prestress tendon(s), $e_{max,b}$	525	mm

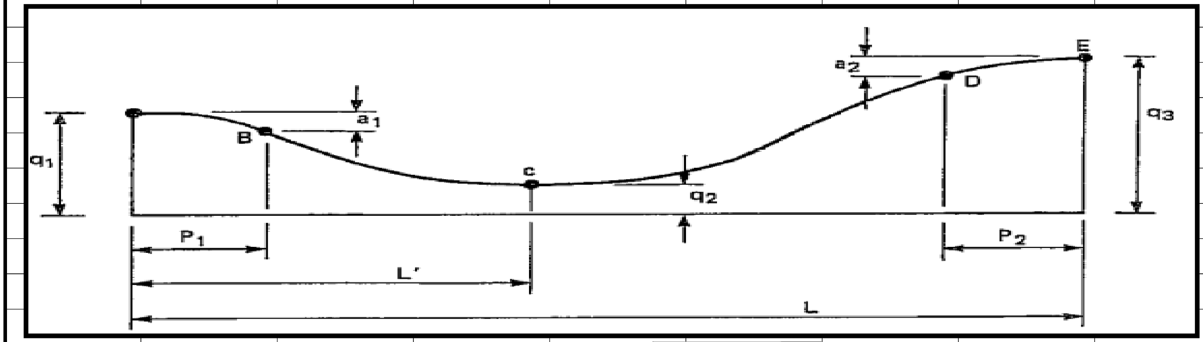
Note $e_{max,b}$ = h-$x_{c,(SLS/ULS)}$-cover-ϕ_{link}-[$D_{T,V}$+($n_{layers,PT}$-1)($D_{T,V}$+$s_{r,PT}$)]/2-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)] [exte

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 ($e_{min,t} \leq e_{HOG}$ and $e_{SAG} \leq e_{max,b}$);

Physical eccentricity of prestress tendon(s) at design section utilisation	100%	OK



Dimension, q_1	Continues ▼		840	mm	Goal Seek q1, q2, q3
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	

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Coefficient, $l = q_1 - q_3$ (note l=N/A if cantilever)	0	mm
Coefficient, $m = (p_2 - 2L).(q_1 - q_2) + p_1.(q_3 - q_2)$ (note m=N/A if cantile	-1.3E+07	mm^2
Coefficient, $n = (q_1 - q_2).(L - p_2).L$ (note n=N/A if cantilever)	6.5E+10	mm^3
Dimension, $L' = [-m - \sqrt{(m^2 - 4l.n)}]/(2l)$ (note L'=L/2 if l=0 or L if car	5000	mm
Dimension, $a_1 = (q_1 - q_2).p_1/L'$	144	mm
Dimension, $a_2 = (q_3 - q_2).p_2/(L - L')$ (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;

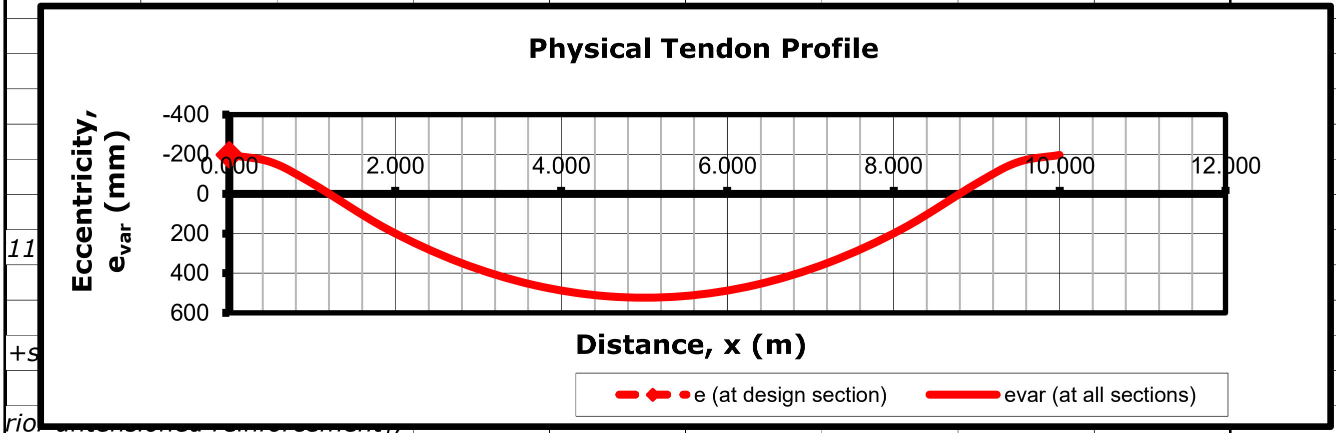
Note tendon profile equations are as follows: -

if $x < p_1$ then $e_{var} = a_1/p_1^2.(x)^2 + h - q_1 - x_{c,(SLS/ULS)}$,

if $x <= L'$ then $e_{var} = -(q_1 - a_1 - q_2)/(L' - p_1)^2.(L' - x)^2 + h - q_2 - x_{c,(SLS/ULS)}$,

if $x <= L - p_2$ then $e_{var} = -(q_3 - a_2 - q_2)/(L - L' - p_2)^2.(x - L')^2 + h - q_2 - x_{c,(SLS/ULS)}$,

if $x > L - p_2$ then $e_{var} = a_2/p_2^2.(L - x)^2 + h - q_3 - x_{c,(SLS/ULS)}$;



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, $A_{s,prov} = n_t.\pi.\phi_t^2/4$			**3142**	**2454**	mm^2	
Number of layers of untensioned steel, $n_{layers,tens}$			2	1	layer(s)	**OK**
Spacer for untensioned steel, $s_{r,tens}$ = MAX (ϕ_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, $A_{sv,prov} = \pi.\phi_{link}^2/4.n_{leg}$				**314**	mm^2	
Pitch of links, S				100	mm	
No., $n_{l,2/3}$ | area, $A_{sv,prov,2/3} = n_{l,2/3}.\pi.\phi_{link}^2/4$	**N/A**	**N/A**	**N/A**	**N/A**	mm^2	
No., $n_{l,4/5}$ | area, $A_{sv,prov,4/5} = n_{l,4/5}.\pi.\phi_{link}^2/4$	**N/A**	**N/A**	**N/A**	**N/A**	mm^2	

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Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

BS8110

External Loading

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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	**OK**
Superimposed dead load (on plan), $\{SDL_h, SDL_v\}$	15.00	0.00	kPa	**OK**
Live load (on plan), $\{LL_h, LL_v\}$	10.00	0.00	kPa	**OK**
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, $DL_b = h.b_w.\rho_c$		12.5	kN/m	
TLS beam loading, $\omega_{TLS,E/E} = k_S.[DL_h].t_w+k_S.DL_b$		**35.0**	kN/m	
DL+SDL beam loading, $\omega_{DL+SDL} = [DL_h+DL_v+SDL_h+SDL_v].t_w+DL_b$		**110.0**	kN/m	
LL beam loading, $\omega_{LL} = [LL_h+LL_v].t_w$		**50.0**	kN/m	
SLS beam loading, $\omega_{SLS,E/E} = [DL_h+DL_v+SDL_h+SDL_v+LL_h+LL_v].t_w+DL_b$		**160.0**	kN/m	
ULS beam loading, $\omega_{ULS,E/E} = [k_G.(DL_h+DL_v+SDL_h+SDL_v)+k_Q.(LL_h+l$	**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, $f_{se} = \%.Kf_{pk}$	**1082**	N/mm^2

Note f_{se} usually 1100 to 1200N/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.	$\omega_{SLS,E/L} = -8KP_0.e_d/s^2$	N/A	kN/m
Cont.	$\omega_{SLS,E/L} = -8KP_0.e_d/s^2$	-131.0	kN/m
Cant.	$\omega_{SLS,E/L} = -2KP_0.e_d/s^2$	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, $\lvert\omega_{SLS,E/L}\rvert/\omega_{TLS,E/E/DL+SDL/SLS,E/E}$	**kN/m**	**%**
of TLS beam loading, $\omega_{TLS,E/E} + DL_{point,h}/L$	35.0	**374%**
of DL+SDL beam loading, $\omega_{DL+SDL} + DL_{point,h}/L + DL_{point,v}$	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**	**R-Sup**	
	Distance between points of inflexion, s	**2.000**	**8.000**	**2.000**	m
S/S.	$s=\{2p_1$ (l-sup), $L-p_1-p_2$ (span), $2p_2$ (r-sup)$\}$	N/A	N/A	N/A	m
Cont.	$s=\{2p_1$ (l-sup), $L-p_1-p_2$ (span), $2p_2$ (r-sup)$\}$	2.000	8.000	2.000	m
Cant.	$s=\{2p_1$ (l-sup), $L-p_1$ (span), N/A (r-sup)$\}$	N/A	N/A	N/A	m
	Total drape between points of inflexion, e_d	**144**	**576**	**144**	mm
S/S.	$e_d=\{a_1$ (l-sup), $e_C-[e_B+e_D]/2$ (span), a_2 (r-s	N/A	N/A	N/A	mm
Cont.	$e_d=\{a_1$ (l-sup), $e_C-[e_B+e_D]/2$ (span), a_2 (r-s	144	576	144	mm
Cant.	$e_d=\{a_1$ (l-sup), e_C-e_B (span), N/A (r-sup)$\}$	N/A	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, $\lvert\omega_{SLS,E/L}\rvert/\omega_{TLS,E/E/DL+SDL/SLS,E/E}$	**82%**	**OK**

CONSULTING ENGINEERS Engineering Calculation Sheet
Consulting Engineers

Job No. jXXX | Sheet No. 9 | Rev.
Member/Location
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Drg. Ref.
Member Design - PC Beam and Slab
Made by XX Date 18/08/2025 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' ≤ 75%.($N_T.N_s.F_{pk}$); *cl.4.7.1*

Max allowable prestress force at transfer (w. restraint, w. ST losses) utilisatio **90%** **OK**

Prestress Force at TLS (With Restraint, Without Short Term Losses) BS8110

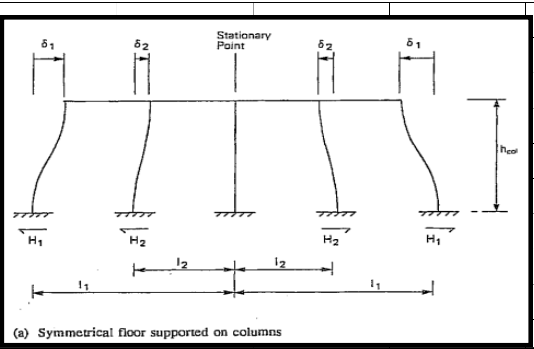
Prestress force at transfer (w.o. restraint, w.o. ST losses), $P_{0,free}$ **2346** kN

Note prestress force at transfer (w.o. restraint, w.o. ST losses), $P_{0,free}$ = %.($N_T.N_s.F_{pk}$);

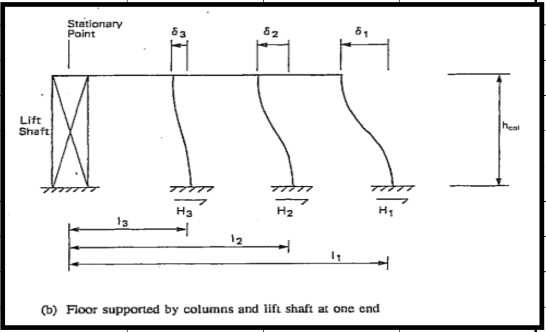
Percentage of tensile capacity, % ($P_{0,free} \leq$ 80%.($N_T.N_s.F_{pk}$) cl.4.7.1) 75.0 % **OK**

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

$$H_i = \frac{12E_cI_i\delta_i}{(h_{col})^3}$$

$$\delta_i = \varepsilon_{LT} \times l_i$$

Total long term strain, $\varepsilon_{LT} = \varepsilon_{es} + \varepsilon_{cp} + \varepsilon_{sh}$ **1486** x10^{-6} l.3.3 TR.43

Elastic shortening strain, ε_{es} 396 x10^{-6}

Note $\varepsilon_{es} = \sigma_{es} / E_{uncracked,28} = [(P_{0,free}/A_{TLS}).(1+e^2A_{TLS}/I_{TLS})] / E_{uncracked,28}$ MOSLEY

noting that e above is taken as MAX[$|e_{HOG}|$,$|e_{SAG}|$];

Creep strain, ε_{cp} 990 x10^{-6}

Note creep strain, $\varepsilon_{cp} = 2.5\,\varepsilon_{es}$; *cl.3.3 TR.43*

Shrinkage strain, ε_{sh} 100 x10^{-6}

Note ε_{sh} usually 100x10^{-6} for UK outdoor exposure conditions; *cl.4.8.4*

Note ε_{sh} usually 300x10^{-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 = MAX ($P_{0,free} - \Sigma H_i$, 0);

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Member Design - PC Beam and Slab		Made by XX	Date **18/08/2025**	Chd.

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Prestress Force Losses

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Goal Seek Losses Factor, K

Prestress force (total) losses factor, K = (P_0 – P_L) / P_0	100%	**0.78**		
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	

Goal Seek ST Losses Factor

Prestress force (short term) losses factor, (P_0 – $P_{L,DF}$ – $P_{L,ES}$) / P_0		**0.90**		
Note prestress force (short term) losses factor, (P_0 – $P_{L,DF}$ – $P_{L,ES}$) / P_0 is usually 0.90 i.e. 10% (				
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*

$$\boxed{P_{L,DF} = P_0\left(1-e^{-(\mu x/r_{ps}+kx)}\right)}$$

Note x = {L/2 simply supported, L/2 continuous, L cantilever}

Coefficient of friction, μ (usually 0.25 BD, 0.07 un-BD (Aalami, 201	0.25	/rad	*cl.4.9.4.3*
Lightly-rusted strand in galvanized steel duct: 0.25 ▼			
Wobble factor, k (usually 46x10^{-4}rad/m (Aalami, 2014))	33	x10^{-4}rad/m	*cl.4.9.3.3*
General: 33x10-4 ▼			
Tendon curvature, C	28.800	x10^{-6}	

$$\boxed{C = \begin{cases} \begin{matrix}\text{simply supported}\\ \text{continuous}\\ \text{hogging / sagging}\end{matrix} & \begin{matrix}\text{cantilever}\\ \\ \text{hogging / sagging}\end{matrix} \\ \dfrac{\left|MAX(e_{SAG},e_{var})-MIN(e_{HOG},e_{var})\right|}{(L/2)^2} & \dfrac{\left|MAX(e_{SAG},e_{var})-MIN(e_{HOG},e_{var})\right|}{L^2} \end{cases}}$$

Tendon radius of curvature, r_{ps}	17.4	m

$$\boxed{r_{ps} \approx \frac{1}{d^2y/dx^2} = \frac{1}{2C}}$$

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*

$$\boxed{\begin{gathered} P_{L,ES} = P_0 - \frac{P_0}{1+\text{factor}.\dfrac{E_p}{E_{uncracked,28,transfer}}.\dfrac{N_T.N_s.A_s}{A_{TLS}}.\left(1+\dfrac{\left(5.MAX\left(|e_{HOG}|,|e_{SAG}|\right)/8\right)^2.A_{TLS}}{I_{TLS}}\right)} \\ \text{factor} = \begin{Bmatrix} 1.0 & \text{Pre-Tension} \\ 0.5 & \text{Post-Tension} \end{Bmatrix} \end{gathered}}$$

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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		Member/Location		
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Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

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BS8110 ▼

Prestress Force Loss due to Concrete Creep Under Sustained Compression (Long Term)

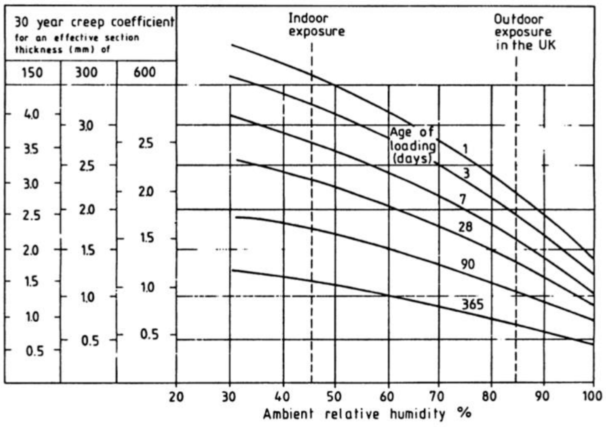
Prestress force loss due to concrete creep, $P_{L,CC}$ **109** kN *cl.4.8.5*

$$P_{L,CC} = E_p . \frac{N_T . N_s . A_s}{A} . P' . \left(1 + \frac{\left(5.MAX\left(\left|e_{HOG}\right|, \left|e_{SAG}\right|\right)/8\right)^2 . A_{(SLS/ULS)}}{I_{(SLS/ULS)}} \right) . \frac{\phi}{E_{uncracked,28}}$$

Creep coefficient, ϕ 2.0

Note ϕ usually 1.8 for 3-day transfer, 1.4 for 28-day transfer for UK outdoor exposure c *cl.4.8.5.2*

3);



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*

$$P_{L,TR} = MAX\left(0.0,\ 0.08 - 0.08 \frac{70\% - P'/\left(N_T.N_s.F_{pk}\right)x100\%}{70\% - 40\%} \right) .P'$$

Prestress Force Loss due to Concrete Shrinkage (Long Term)

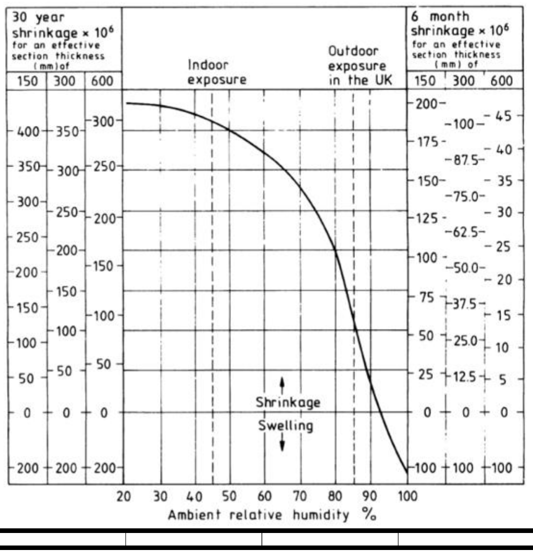
Prestress force loss due to concrete shrinkage, $P_{L,CS}$ **31** kN *cl.4.8.4*

$$P_{L,CS} = \varepsilon_{sh} . E_p . N_T . N_s . A_s$$

Shrinkage strain, ε_{sh} 100 $x10^{-6}$

Note ε_{sh} usually $100x10^{-6}$ for UK outdoor exposure conditions; *cl.4.8.4*

Note ε_{sh} usually $300x10^{-6}$ for UK indoor exposure conditions; *cl.4.8.4*



Note the effective thickness is taken as twice the cross sectional area divided by the

-2

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BS8110 ▼

Input Summary

Item					
Job Title				3110 (EC2), ACI318, AS3600 v2024.02.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, $\lvert\omega_{SLS,E/L}\rvert$		**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	N/A
Shear and Punching Shear Links Area	3,142	N/A	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 x l_{hface}				0 x 0	mm
Column Position and Orientation				Interior	
Design Strip Direction				Along h	
Punching Shear $M_0V_{ult}/\lvert M_{ult}\rvert$?				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	

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Utilisation Summary

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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 sectio		**Converged**	63%	OK	
Rect. beam shear ultimate stress at critical section			64%	OK	
Rect. beam shear design capacity at (shear) design section			67%	OK	
Rect. beam shear design capacity at all sections			67%	OK	
Rect. beam punching shear at column face perimeter			N/A	OK	
Rect. beam punching shear at column 1st shear perime		**N/A**	N/A	N/A	
Rect. beam punching shear at column 2nd shear perime		**N/A**	N/A	N/A	
Rect. beam punching shear at column 3rd shear perime		**N/A**	N/A	N/A	
Rect. beam punching shear at column 4th shear perime		**N/A**	N/A	N/A	
Detailing requirements				**OK**	

Note calculate UTs for cases with / without restraint to prestress force;

				Bending
Overall utilisation	Hogging Moment ▼	**100%**		**Pch. Shea**
Inclusion of restraint to prestress force, ΣH_i		Exclude ▼		
Inclusion of prestress force losses, K		Include ▼		**OK**
Inclusion of secondary effects ?		Include ▼		**OK**

% Tensioned reinforcement (rectangular or flanged)				**0.34**	%
7850 . [(N_T.N_s.A_s) / b_w.h];					
% Untensioned reinforcement (rectangular or flanged) hog / sag			**0.63**	**0.49**	%
7850 . [($A_{s,prov,h}$ +$A_{s,prov,s}$) / b_w.h + ($A_{sv,prov}$.(h+b_w)/S) / b_w.h]; No curtailment; No laps;					
Estimated tensioned reinforcement quantity				**26**	kg/m^3
Estimated untensioned reinforcement quanti	**49**	**39**	**74**	**162**	kg/m^3
[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/					
Material coconcrete, c	315	units/m^3	tendon, t	12500	steel, s
				3600	units/tonne
Reinforced concrete material cost = [c+(est. tendon quant).t+(est. rebar qua				**614**	units/m
Degree of partial prestressing, PI = N_T.N_s.A_s.f_{pk} / [N_T.N_s.A_s.f_{pk}+$A_{s,prov}$.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} = MIN (60(b_w or b), 250(b_w or b)^2/h) and cant L_{LTB} = MIN (25b_w, 100b_w^{2}/h);

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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

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Action Effects From Structural Analysis (External Effects)

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
	$M_{SAG,TLS,E/E}=[0.125(\omega_{TLS,E/E})].L^2+DL_{point,h}.a_h.(L-a_h)/L$	N/A	kNm
	$V_{TLS,E/E}=[0.500(\omega_{TLS,E/E})].L+DL_{point,h}.(L-a_h)/L$	N/A	kN
SLS	$M_{HOG,SLS,E/E}=0$	N/A	kNm
	$M_{SAG,SLS,E/E}=[0.125(\omega_{SLS,E/E})].L^2+DL_{point,h}.a_h.(L-a_h)/L+DL_{p}$	N/A	kNm
	$V_{SLS,E/E}=[0.500(\omega_{SLS,E/E})].L+DL_{point,h}.(L-a_h)/L+DL_{point,v}.(L$	N/A	kN
ULS	$M_{HOG,ULS,E/E}=0$	N/A	kNm
	$M_{SAG,ULS,E/E}=[0.125(\omega_{ULS,E/E})].L^2+k_G.DL_{point,h}.a_h.(L-a_h)/L+$	N/A	kNm
	$V_{ULS,E/E}=[0.500(\omega_{ULS,E/E})].L+k_G.DL_{point,h}.(L-a_h)/L+k_G.DL_{po}$	N/A	kN

Continuous (Infinitely, Encastre) — Continues ▼ — Continues ▼ — **VALID**

						Note
TLS	$M_{HOG,TLS,E/E}=-[\% \text{ x } 0.083(\omega_{TLS,E/E})].L^2-[\% \text{ x } f($		100%	-292	kNm	
	$M_{SAG,TLS,E/E}=M_{HOG,TLS,E/E}+V_{TLS,E/E}.[L/2]*-\omega_{TLS,E/E}.(L/2)^2/2$			146	kNm	
	$V_{TLS,E/E}=[\% \text{ x } 0.500(\omega_{TLS,E/E})].L+[$	100%	100%	175	kN	
SLS	$M_{HOG,SLS,E/E}=-[\% \text{ x } 0.083(\omega_{SLS,E/E})].L^2-[\% \text{ x } f($		100%	-1333	kNm	
	$M_{SAG,SLS,E/E}=M_{HOG,SLS,E/E}+V_{SLS,E/E}.[L/2]*-\omega_{SLS,E/E}.(L/2)^2/2$			667	kNm	**Goal Seek BMD**
	$V_{SLS,E/E}=[\% \text{ x } 0.500(\omega_{SLS,E/E})].L+[$	100%	100%	800	kN	
ULS	$M_{HOG,ULS,E/E}=-[\% \text{ x } 0.083(\omega_{ULS,E/E})].L^2-[\% \text{ x } k$		100%	-1950	kNm	
	$M_{SAG,ULS,E/E}=k_{PAT}.[M_{HOG,ULS,E/E}+V_{ULS,E/E}.[L/2]*-\omega_{ULS,E/E}.(L/2$			975	kNm	**Goal Seek Shear**
	$V_{ULS,E/E}=[\% \text{ x } 0.500(\omega_{ULS,E/E})].L+[$	100%	100%	1170	kN	

Cantilever — **N/A**

TLS	$M_{HOG,TLS,E/E}=-[0.500(\omega_{TLS,E/E})].L^2-DL_{point,h}.a_h$	N/A	kNm
	$M_{SAG,TLS,E/E}=0$	N/A	kNm
	$V_{TLS,E/E}=[(\omega_{TLS,E/E})].L+DL_{point,h}$	N/A	kN
SLS	$M_{HOG,SLS,E/E}=-[0.500(\omega_{SLS,E/E})].L^2-DL_{point,h}.a_h-DL_{point,v}.a_v$	N/A	kNm
	$M_{SAG,SLS,E/E}=0$	N/A	kNm
	$V_{SLS,E/E}=[(\omega_{SLS,E/E})].L+DL_{point,h}+DL_{point,v}$	N/A	kN
ULS	$M_{HOG,ULS,E/E}=-[0.500(\omega_{ULS,E/E})].L^2-k_G.DL_{point,h}.a_h-k_G.DL_{point}$	N/A	kNm
	$M_{SAG,ULS,E/E}=0$	N/A	kNm
	$V_{ULS,E/E}=[(\omega_{ULS,E/E})].L+k_G.DL_{point,h}+k_G.DL_{point,v}$	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;

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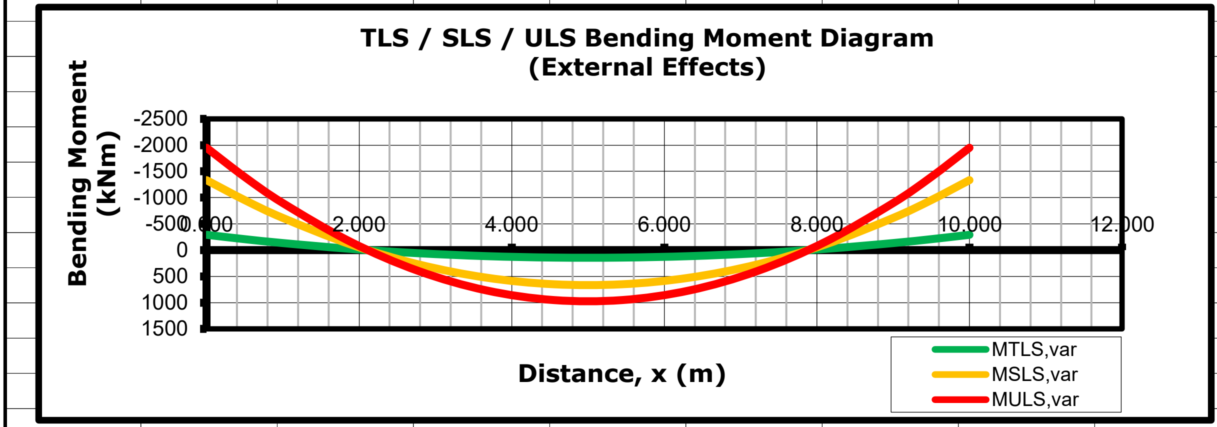
TLS / SLS / ULS Bending Moment Diagram (kNm)

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;

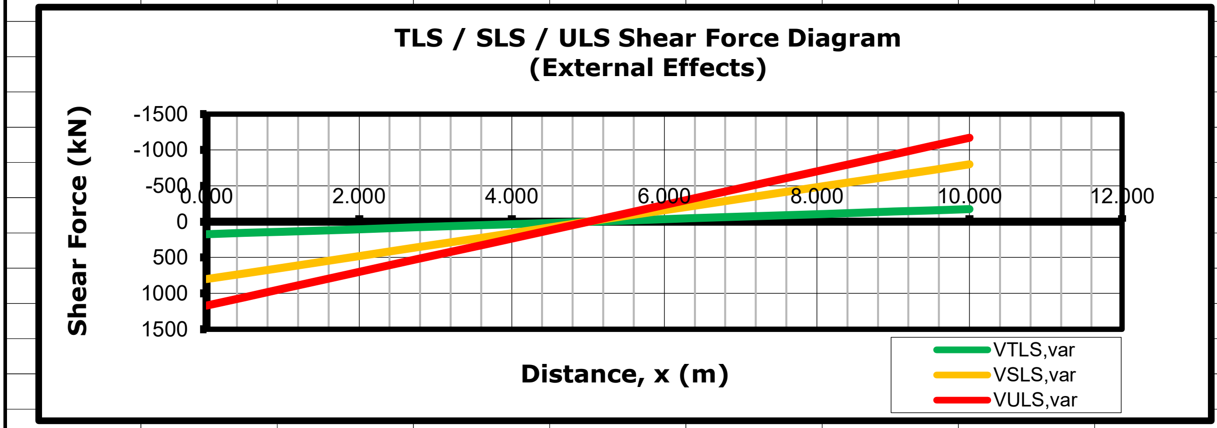


Note by convention, a negative bending moment indicates hogging moment;

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;

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

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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 drape between points of inflexion, e_d	144	576	144	mm
TLS equivalent load, $\omega_{TLS,E/L}$ = ±[8|2]k_PP'.e_d/s^2	700.7	-175.2	700.7	kN/m
SLS equivalent load, $\omega_{SLS,E/L}$ = ±[8|2]KP_0.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, Σ{p_1, L-p_1-p_2, p_2}.$\omega_{SLS,E/L}$	524	-1048	524	kN	**0**

Inclusion of secondary effects ? Include ▼

Simply Supported, Continuous (Infinitely, Encastre), Cantilever

Note primary effects P/E and secondary effects S/E equations: -

P/E	$M_{HOG,TLS,P/E} = -k_P P'.e_{HOG}$	$M_{HOG,SLS,P/E} = -KP_0.e_{HOG}$
	$M_{SAG,TLS,P/E} = -k_P P'.e_{SAG}$	$M_{SAG,SLS,P/E} = -KP_0.e_{SAG}$
	$V_{TLS,P/E} \approx dM_{TLS,P/E}/dx$	$V_{SLS,P/E} \approx dM_{SLS,P/E}/dx$
S/E	$M_{HOG,TLS/SLS,S/E} = M_{HOG,TLS/SLS,E/L} - M_{HOG,TLS/SLS,P/E}$	
	$M_{SAG,TLS/SLS,S/E} = M_{SAG,TLS/SLS,E/L} - M_{SAG,TLS/SLS,P/E}$	
	$V_{TLS/SLS,S/E} = V_{TLS/SLS,E/L} - V_{TLS/SLS,P/E}$	

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

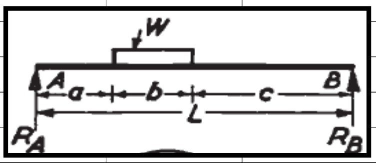
Note equivalent load effects E/L equations: -

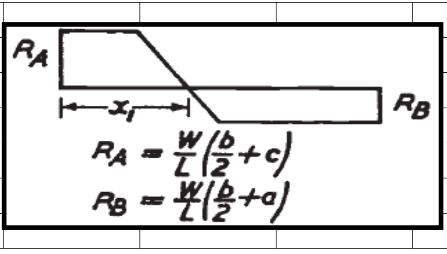
E/L

$M_{HOG,TLS/SLS,E/L} = 0 - [k_P P' \text{ or } KP_0].e_{var}(x=0)$

$M_{SAG,TLS/SLS,E/L} = 0 + V_{TLS/SLS,E/L}.L/2 - f[\omega_{TLS/SLS,E/L}, x=L/2] - [k_P P' \text{ or } KP_0].e_{var}(x=0) + [k_P P'$

$V_{TLS/SLS,E/L} = f[\omega_{TLS/SLS,E/L}, x=0] + [k_P P' \text{ or } KP_0].e_{var}(x=0)/L - [k_P P' \text{ or } KP_0].e_{var}(x=L)/L$





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

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Note equivalent load effects E/L equations: -

E/L

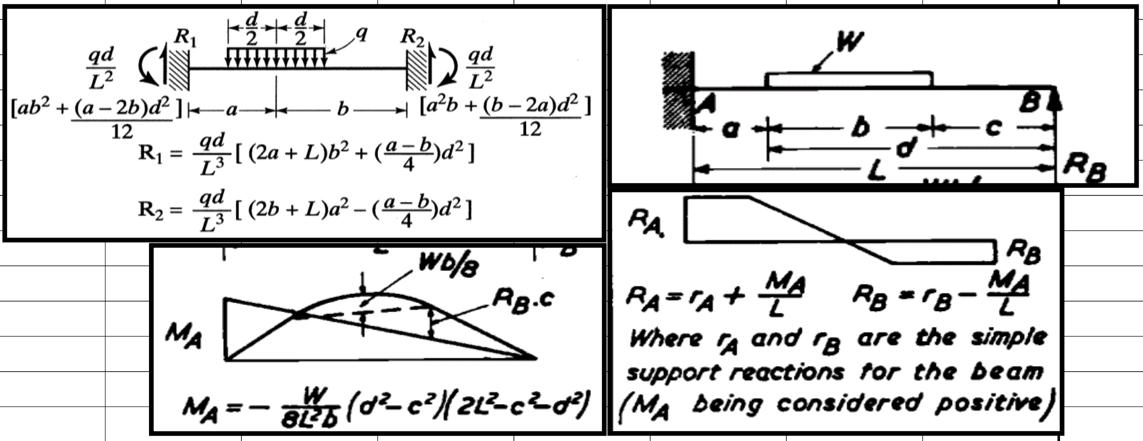
$M_{HOG,TLS/SLS,E/L} = -\% \; x \; f[\; \omega_{TLS/SLS,E/L}, x=0] - [k_P P' \text{ or } KP_0].e_{var}(x=0)$

$M_{SAG,TLS/SLS,E/L} = M_{HOG,TLS/SLS,E/L} + V_{TLS/SLS,E/L}.L/2 - f[\; \omega_{TLS/SLS,E/L}, x=L/2] - [k_P P' \text{ or } KP_0].e_{var}($

$V_{TLS/SLS,E/L} = \% \; x \; f[\; \omega_{TLS/SLS,E/L}, x=0] + [k_P P' \text{ or } KP_0].e_{var}(x=0)/L - [k_P P' \text{ or } KP_0].e_{var}(x=L)$

Tendon termination at x=0 ? Continues ▼

Tendon termination at x=L ? Continues ▼



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: -

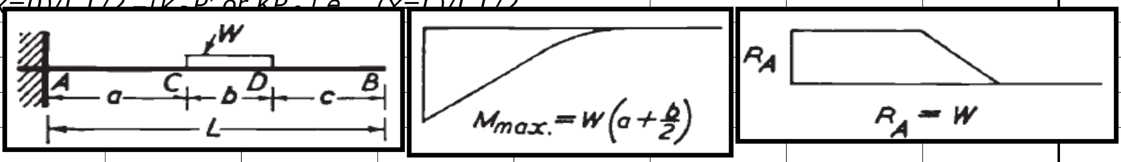
E/L

$M_{HOG,TLS/SLS,E/L} = -f[\; \omega_{TLS/SLS,E/L}, x=0\;]$

$M_{SAG,TLS/SLS,E/L} = M_{HOG,TLS/SLS,E/L} + V_{TLS/SLS,E/L}.L - f[\; \omega_{TLS/SLS,E/L}, x=L] - [k_P P' \text{ or } KP_0].e_{var}(x=L)$

$V_{TLS/SLS,E/L} = f[\; \omega_{TLS/SLS,E/L}, x=0] - [k_P P' \text{ or } KP_0].e_{var}(x=L)/L$

$\text{or } KP_0].e_{var}(x=0)/L.L/2 - [k_P P' \text{ or } KP_0].e_{var}(x=L)/L.L/2$



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;

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 19	Rev.
		Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

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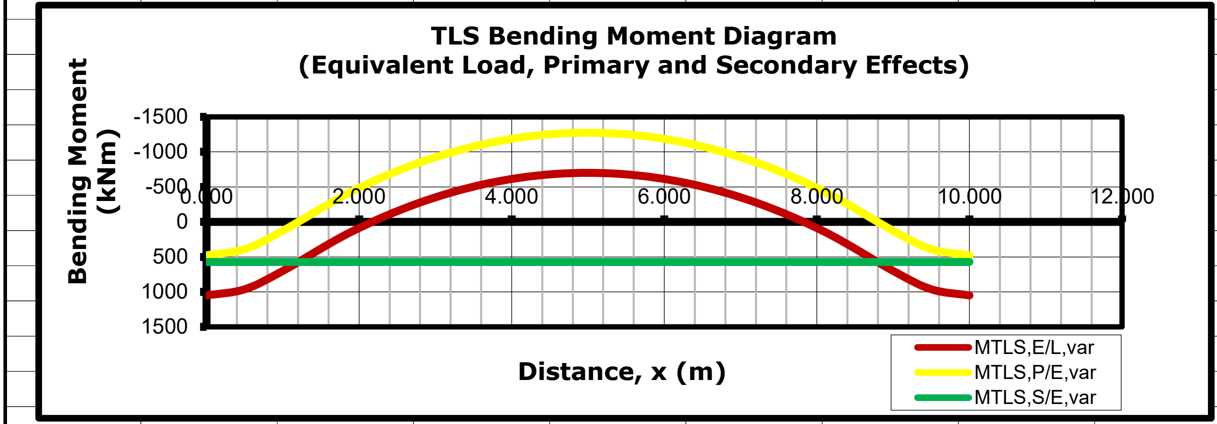
TLS Bending Moment Diagram (kNm)

Dist, x	**0.000**	**0.500**	**1.000**	**1.889**	**2.778**	**3.667**	**4.556**	**m**
$M_{TLS,E/L,var}$	1051	963	701	147	-268	-545	-683	kNm
$M_{TLS,P/E,var}$	478	390	127	-426	-841	-1118	-1257	kNm
$M_{TLS,S/E,var}$	573	573	573	573	573	573	573	kNm
Dist, x	**5.444**	**6.333**	**7.222**	**8.111**	**9.000**	**9.500**	**10.000**	**m**
$M_{TLS,E/L,var}$	-683	-545	-268	147	701	963	1051	kNm
$M_{TLS,P/E,var}$	-1257	-1118	-841	-426	127	390	478	kNm
$M_{TLS,S/E,var}$	573	573	573	573	573	573	573	kNm

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

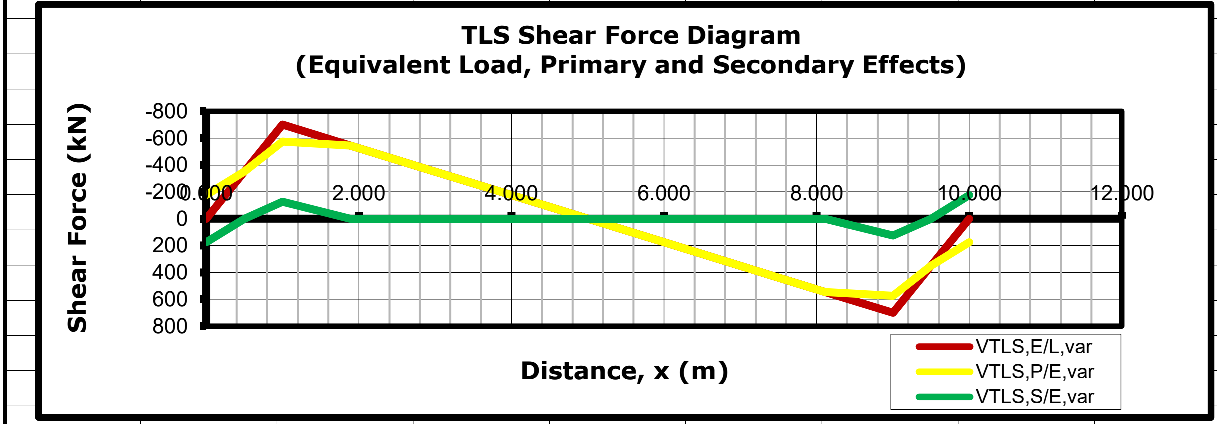


Note by convention, a negative bending moment indicates hogging moment;

TLS Shear Force Diagram (kN)

Dist, x	**0.000**	**0.500**	**1.000**	**1.889**	**2.778**	**3.667**	**4.556**	**m**
$V_{TLS,E/L,var}$	0	-350	-701	-545	-389	-234	-78	kN
$V_{TLS,P/E,var}$	-175	-350	-574	-545	-389	-234	-78	kN
$V_{TLS,S/E,var}$	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**
$V_{TLS,E/L,var}$	78	234	389	545	701	350	0	kN
$V_{TLS,P/E,var}$	78	234	389	545	574	350	175	kN
$V_{TLS,S/E,var}$	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;

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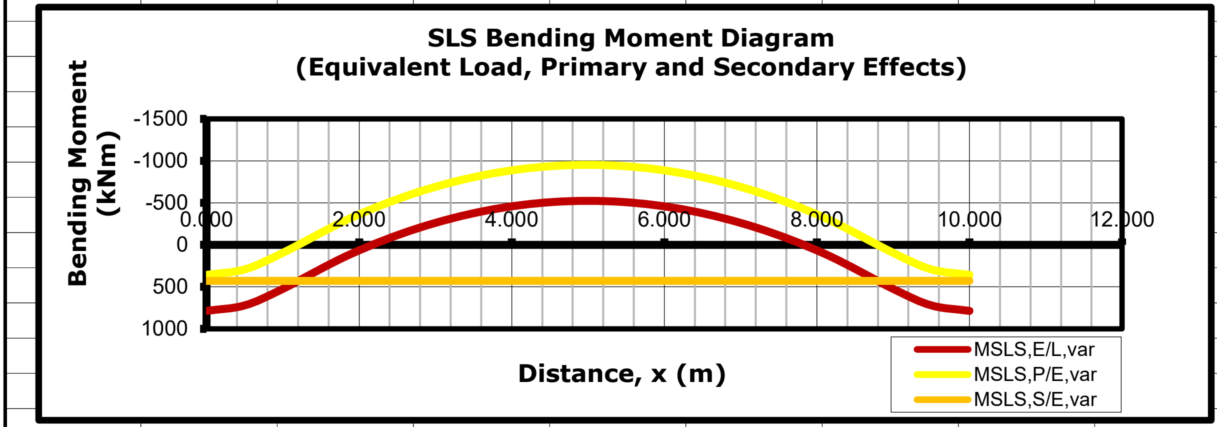
SLS Bending Moment Diagram (kNm)

Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$M_{SLS,E/L,var}$	786	721	524	110	-201	-408	-511	kNm
$M_{SLS,P/E,var}$	357	292	95	-319	-629	-836	-940	kNm
$M_{SLS,S/E,var}$	429	429	429	429	429	429	429	kNm
Dist, x	**5.444**	**6.333**	**7.222**	**8.111**	**9.000**	**9.500**	**10.000**	**m**
$M_{SLS,E/L,var}$	-511	-408	-201	110	524	721	786	kNm
$M_{SLS,P/E,var}$	-940	-836	-629	-319	95	292	357	kNm
$M_{SLS,S/E,var}$	429	429	429	429	429	429	429	kNm

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

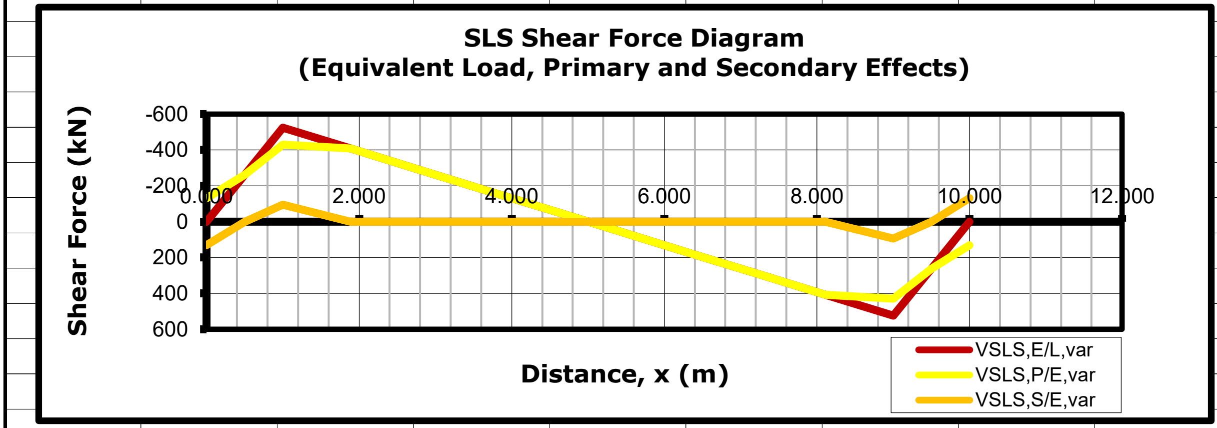


Note by convention, a negative bending moment indicates hogging moment;

SLS Shear Force Diagram (kN)

Dist, x	0.000	0.500	1.000	1.889	2.778	3.667	4.556	m
$V_{SLS,E/L,var}$	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;

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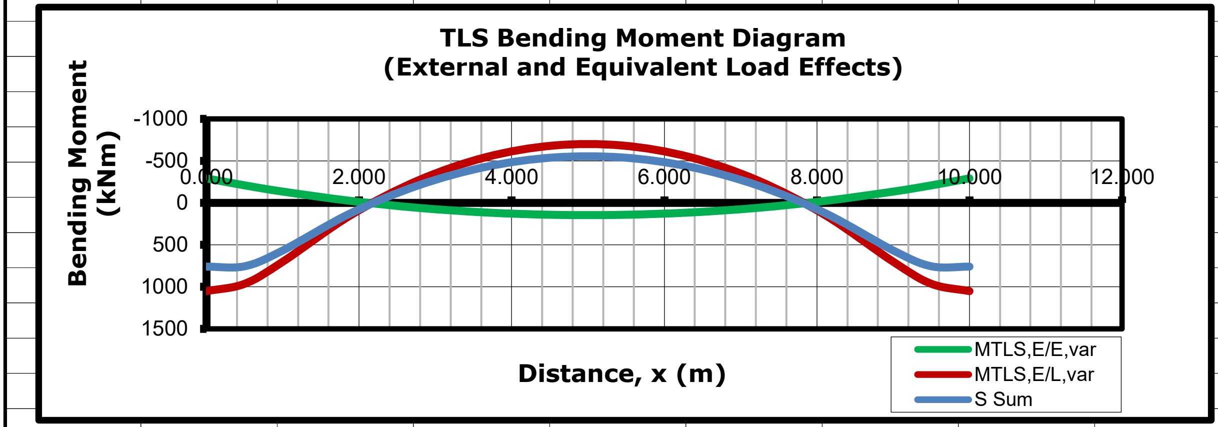
Action Effects From Structural Analysis (External, Equivalent Load and Secondary Effects)

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;

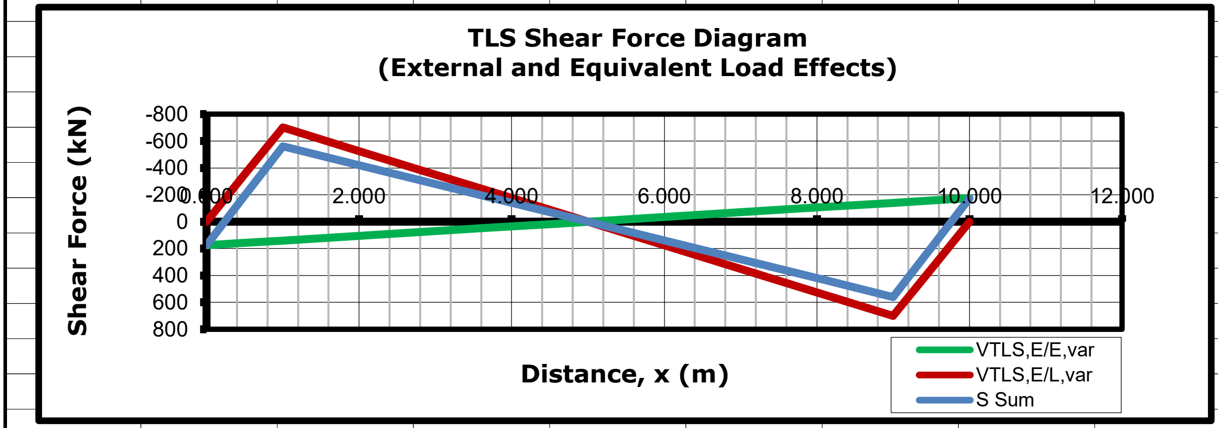

TLS Bending Moment Diagram (External and Equivalent Load Effects)

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**
$V_{TLS,E/E,var}$	175	158	140	109	78	47	16	kN
$V_{TLS,E/L,var}$	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**
$V_{TLS,E/E,var}$	-16	-47	-78	-109	-140	-158	-175	kN
$V_{TLS,E/L,var}$	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;


TLS Shear Force Diagram (External and Equivalent Load Effects)

Note an arbitrary shear force sign convention is employed;

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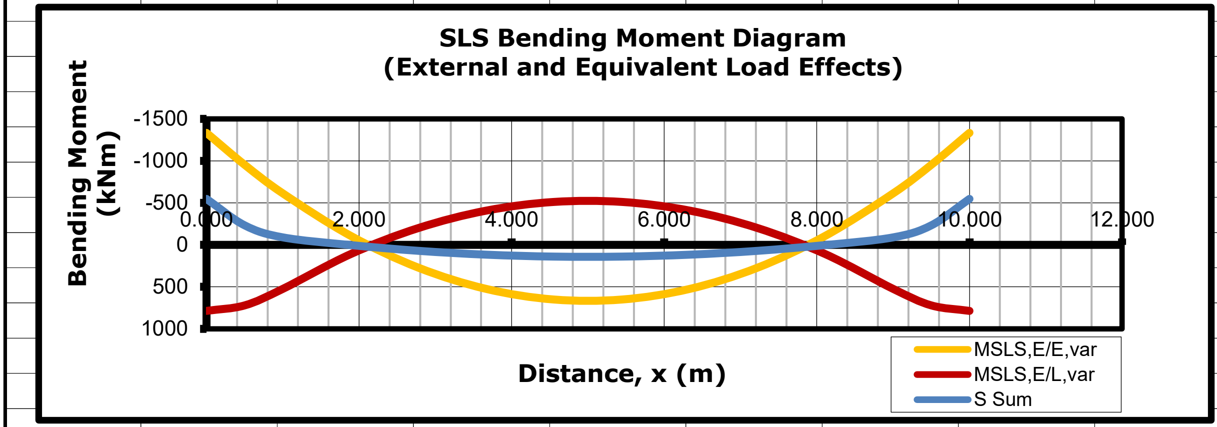
SLS Bending Moment Diagram (kNm)

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;

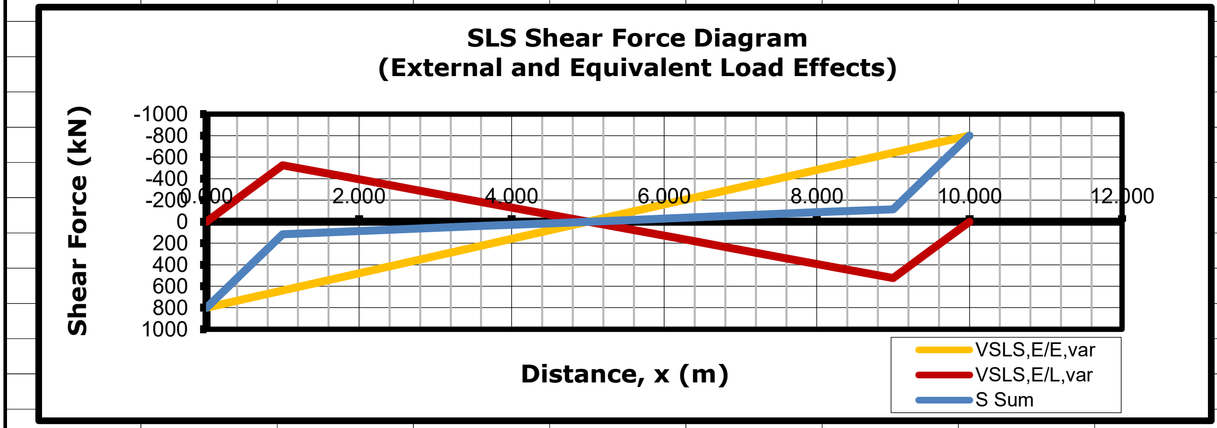


Note by convention, a negative bending moment indicates hogging moment;

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;

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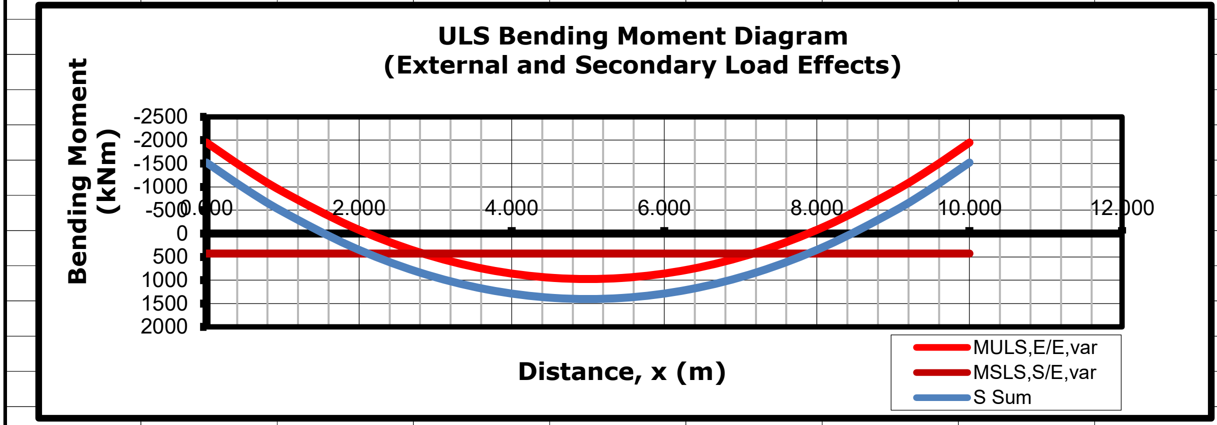
ULS Bending Moment Diagram (kNm)

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;

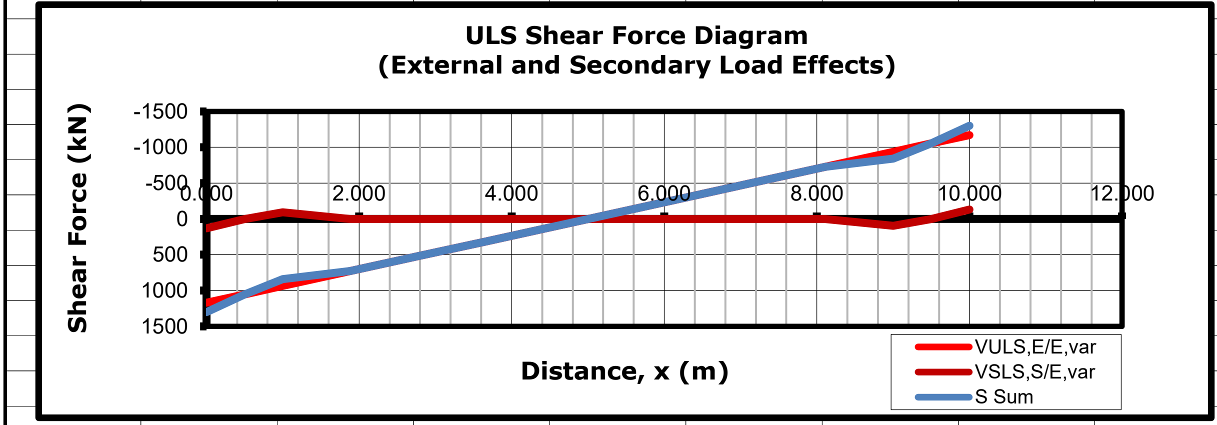


Note by convention, a negative bending moment indicates hogging moment;

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;

BS8110

Allowable Range of Prestress Force at Transfer (for Given Eccentricity) at Design Section

BS8110 ▼

Sagging Moment **Invalid** Hogging Moment **Valid**

$$P_0 \;>=\; \boxed{\frac{(Z_t f_{max} - M_{max})}{K(Z_t / A - e_{SAG})}} \quad \boxed{\frac{(Z_t f_{min} - M_{max})}{K(Z_t / A - e_{HOG})}}$$

MIN **1414** kN

Note A and Z_t in the above inequality refer to $A_{(SLS/ULS)}$ and $Z_{t,(SLS/ULS)}$ respectively;

$$P_0 \;<=\; \boxed{\frac{(Z_t f^{I}_{min} - M_{min})}{(Z_t / A - e_{SAG})} \frac{1}{k_P} + (P_{L,DF} + P_{L,ES})}$$

MAX **5920** kN

$$\boxed{\frac{(Z_t f^{I}_{max} - M_{min})}{(Z_t / A - e_{HOG})} \frac{1}{k_P} + (P_{L,DF} + P_{L,ES})}$$

Note A and Z_t in the above inequality refer to A_{TLS} and $Z_{t,TLS}$ respectively;

$$P_0 \;>=\; \boxed{\frac{(Z_b f_{min} + M_{max})}{K(Z_b / A + e_{SAG})}} \quad \boxed{\frac{(Z_b f_{max} + M_{max})}{K(Z_b / A + e_{HOG})}}$$

MIN **-7116** kN

Note A and Z_b in the above inequality refer to $A_{(SLS/ULS)}$ and $Z_{b,(SLS/ULS)}$ respectively;

$$P_0 \;<=\; \boxed{\frac{(Z_b f^{I}_{max} + M_{min})}{(Z_b / A + e_{SAG})} \frac{1}{k_P} + (P_{L,DF} + P_{L,ES})}$$

MAX **3280** kN

$$\boxed{\frac{(Z_b f^{I}_{min} + M_{min})}{(Z_b / A + e_{HOG})} \frac{1}{k_P} + (P_{L,DF} + P_{L,ES})}$$

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** ≤ **2346** ≤ **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

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Max economic upper limit to prestress force at transfer (w. restraint, w.o. ST **5915** kN

Sagging Moment **Invalid** Hogging Moment **Valid**

$$\boxed{P_0 = \frac{f_{max} Z_t + f_{min} Z_b}{K\left(\frac{Z_b + Z_t}{A}\right)}} \quad \boxed{P_0 = \frac{f_{min} Z_t + f_{max} Z_b}{K\left(\frac{Z_b + Z_t}{A}\right)}}$$

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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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

$$f_{min} \le \left[f_t = \frac{KP_0}{A} - \frac{KP_0 e}{Z_t} + \frac{M_{SLS,E/E}}{Z_t} + \frac{M_{SLS,S/E}}{Z_t} \right] \le f_{max}$$

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

$$f^I_{min} \le \left[f^I_t = \frac{k_P P'}{A} - \frac{k_P P' e}{Z_t} + \frac{M_{TLS,E/E}}{Z_t} + \frac{M_{TLS,S/E}}{Z_t} \right] \le f^I_{max}$$

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

$$f_{min} \le \left[f_b = \frac{KP_0}{A} + \frac{KP_0 e}{Z_b} - \frac{M_{SLS,E/E}}{Z_b} - \frac{M_{SLS,S/E}}{Z_b} \right] \le f_{max}$$

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

$$f^I_{min} \le \left[f^I_b = \frac{k_P P'}{A} + \frac{k_P P' e}{Z_b} - \frac{M_{TLS,E/E}}{Z_b} - \frac{M_{TLS,S/E}}{Z_b} \right] \le f^I_{max}$$

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_PP	0.9	≤	1.7	≤	6.0	N/mm^2
SLS average precompression, KP	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 of restraint to slab shortening by supports become important, otherwise they may be ignored; *cl.3.3 TR.43*

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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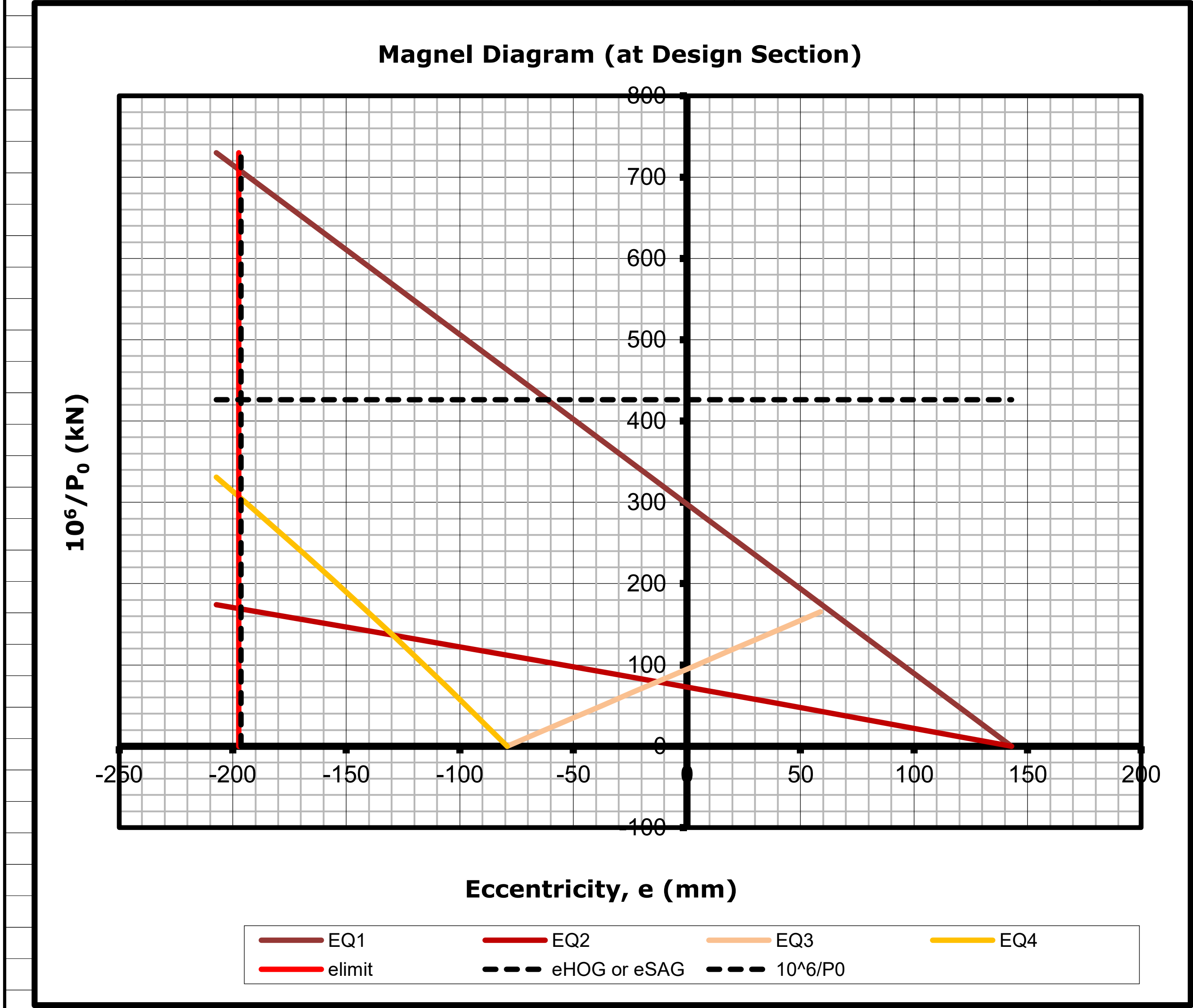
		Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

BS8110

Magnel Diagram at Design Section Rectangular or Flanged Beam

BS8110 ▼



Drawing Limits	Equation	$e_{min,t,magnel}$	$e_{max,b,magnel}$	
	Equation 1, EQ.1	-207	143	mm
	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

$$\frac{1}{P_o} \geq \frac{\left(1/A_{(SLS/ULS)} - e/Z_{t,(SLS/ULS)}\right)}{\left(f_{max} - M_{max}/Z_{t,(SLS/ULS)}\right)/K} \qquad \frac{1}{P_o} \leq \frac{\left(1/A_{(SLS/ULS)} - e/Z_{t,(SLS/ULS)}\right)}{\left(f_{min} - M_{max}/Z_{t,(SLS/ULS)}\right)/K}$$

Equation 2

$$\frac{1}{k_P P'} \leq \frac{\left(1/A_{TLS} - e/Z_{t,TLS}\right)}{\left(f^{l}_{min} - M_{min}/Z_{t,TLS}\right)} \qquad \frac{1}{k_P P'} \geq \frac{\left(1/A_{TLS} - e/Z_{t,TLS}\right)}{\left(f^{l}_{max} - M_{min}/Z_{t,TLS}\right)}$$

effec

Equation 3

$$\frac{1}{P_o} \leq \frac{\left(1/A_{(SLS/ULS)} + e/Z_{b,(SLS/ULS)}\right)}{\left(f_{min} + M_{max}/Z_{b,(SLS/ULS)}\right)/K} \qquad \frac{1}{P_o} \geq \frac{\left(1/A_{(SLS/ULS)} + e/Z_{b,(SLS/ULS)}\right)}{\left(f_{max} + M_{max}/Z_{b,(SLS/ULS)}\right)/K}$$

heme

Equation 4

$$\frac{1}{k_P P'} \geq \frac{\left(1/A_{TLS} + e/Z_{b,TLS}\right)}{\left(f^{l}_{max} + M_{min}/Z_{b,TLS}\right)} \qquad \frac{1}{k_P P'} \leq \frac{\left(1/A_{TLS} + e/Z_{b,TLS}\right)}{\left(f^{l}_{min} + M_{min}/Z_{b,TLS}\right)}$$

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 $e_{TLS} = e_{(SLS/ULS)} + x_{c,(SLS/ULS)} - x_{c,TLS}$;

BS8110

Allowable Tendon Profile (for Given Prestress Force at Transfer) at All Sections Rectangul BS8110

Sagging Moment **Invalid** Hogging Moment **Valid**

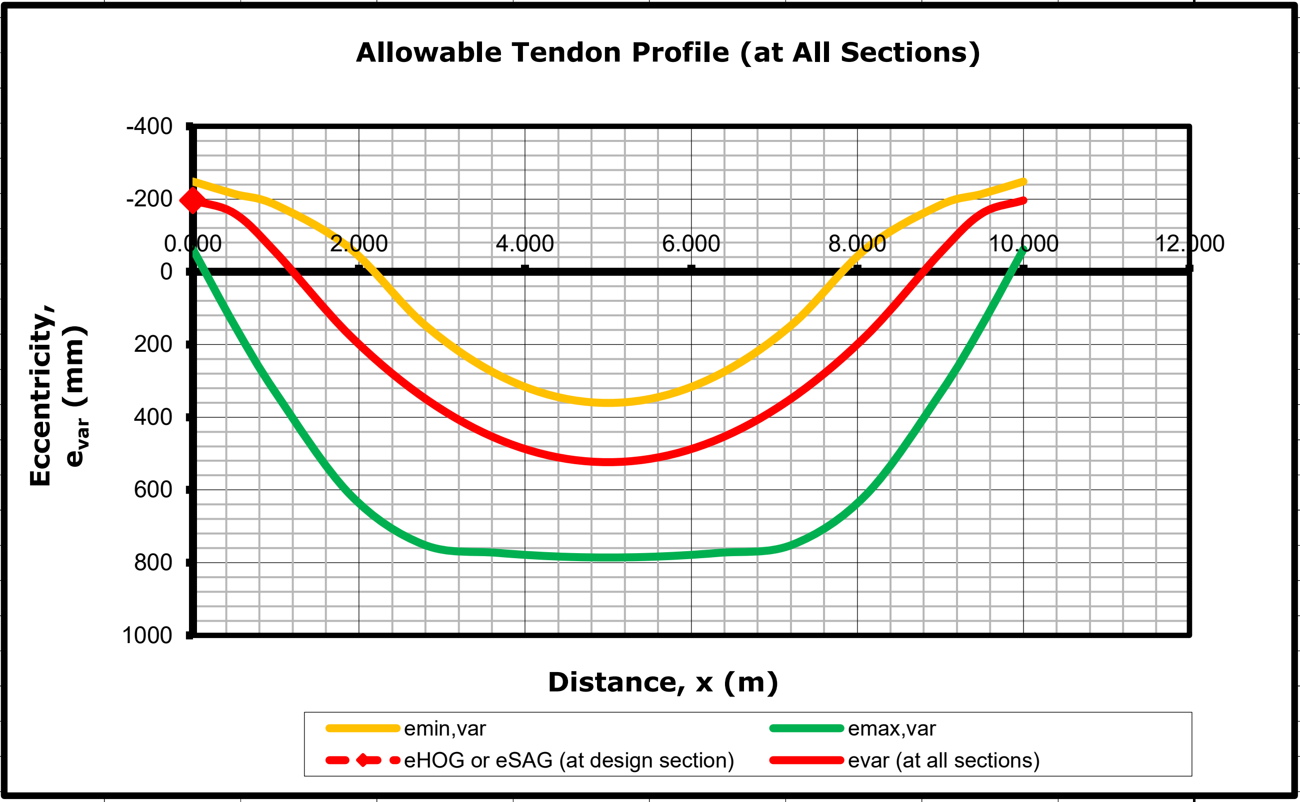
$e_{var} \geq \left[\frac{Z_t}{A} - \frac{f_{max}Z_t}{KP_o}\right] + \frac{M_{max,var}}{KP_o}$	$e_{var} \leq \left[\frac{Z_t}{A} - \frac{f_{min}Z_t}{KP_o}\right] + \frac{M_{max,var}}{KP_o}$	*Note A is $A_{(SLS/ULS)}$, and Z_t is $Z_{t,(SLS/ULS)}$*
$e_{var} \leq \left[\frac{Z_t}{A} - \frac{f^{l}_{min}Z_t}{k_pP'}\right] + \frac{M_{min,var}}{k_pP'}$	$e_{var} \geq \left[\frac{Z_t}{A} - \frac{f^{l}_{max}Z_t}{k_pP'}\right] + \frac{M_{min,var}}{k_pP'}$	*Note A is A_{TLS}, and Z_t is $Z_{t,TLS}$*
$e_{var} \geq \left[-\frac{Z_b}{A} + \frac{f_{min}Z_b}{KP_o}\right] + \frac{M_{max,var}}{KP_o}$	$e_{var} \leq \left[-\frac{Z_b}{A} + \frac{f_{max}Z_b}{KP_o}\right] + \frac{M_{max,var}}{KP_o}$	*Note A is $A_{(SLS/ULS)}$, and Z_b is $Z_{b,(SLS/ULS)}$*
$e_{var} \leq \left[-\frac{Z_b}{A} + \frac{f^{l}_{max}Z_b}{k_pP'}\right] + \frac{M_{min,var}}{k_pP'}$	$e_{var} \geq \left[-\frac{Z_b}{A} + \frac{f^{l}_{min}Z_b}{k_pP'}\right] + \frac{M_{min,var}}{k_pP'}$	*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;

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

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 $k_pP' = k_p[P_0 - (P_{L,DF} + P_{L,ES})]$; 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 range of eccentricity (f	**-248**	≤	**-196**	≤	**-62**	mm

Allowable range of eccentricity (for given P_0) at design section, e utilisation **79%** **OK**

Allowable range of eccentricity (for given P_0) at all sections, e_{var} utilisation **79%** **OK**

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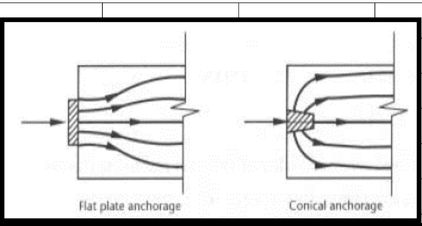
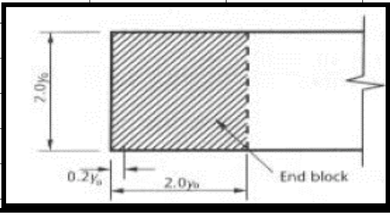
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

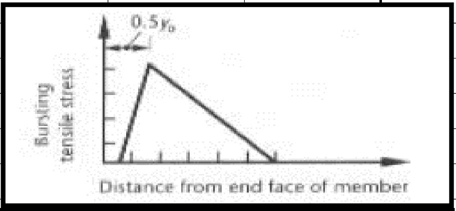
BS8110

End Block Design Rectangular or Flanged Beam

BS8110







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	>=	195	mm	**OK**
Horizontal anchorage spacing, $b_{w,e}/N_A$	500	>=	370	mm	**OK**
Vertical anchorage edge distance, MIN (A_{VED}, h–A_{VED}–(N	500	>=	195	mm	**OK**
Vertical anchorage spacing, A_{VS}	1000	>=	370	mm	**OK**
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	x $2y_{p0,w}$	270	mm	
Total equivalent depth of all anchorages, $\Sigma 2y_{p0,d}$	1	x $2y_{p0,d}$	270	mm	
Maximum local compressive bearing stress, $[P_{0,free}/N_T]/[2y_{p0,w}.2y_{p0,d}]$			32.2	N/mm^2	
Maximum local compressive bearing stress utilisation, $[P_{0,free}/N_T]/[2y_{p0,w}.2y_{p0,c}$			**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	*11.2 / cl.4.*
Jacking force at all anchorages, $P_{0,free}$			2346	kN	*11.2 / cl.4.*
Bursting tensile force (width ratio), $F_{bst,w} = f(2y_{p0,w}/2y_{0,w}).(P_{0,free}/N_T)$			**399**	kN	*T.4.7*
Bursting tensile force (depth ratio), $F_{bst,d} = f(2y_{p0,d}/2y_{0,d}).(P_{0,free}/N_T)$			**540**	kN	*T.4.7*
Bursting tensile force (width ratio), $\Sigma F_{bst,w} = f(\Sigma 2y_{p0,w}/\Sigma 2y_{0,w}).P_{0,free}$			**399**	kN	*T.4.7*
Bursting tensile force (depth ratio), $\Sigma F_{bst,d} = f(\Sigma 2y_{p0,d}/\Sigma 2y_{0,d}).P_{0,free}$			**540**	kN	*T.4.7*
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, $A_{sv,prov,e} = \pi.\phi_{link,e}{}^2/4$			**804**	mm^2	
End block pitch of links, S_e			150	mm	
Allowable stress in end block shear links, σ_e = 200N/mm^2			200	N/mm^2	*11.2 / cl.4.*
Provide shear links $A_{sv,e}/S_e > [F_{bst,w}/(2y_{0,w}\text{-}0.2y_{0,w})]/\sigma_e$ i	**4.43**	mm^2/mm/	**500**	mm	
Provide shear links $A_{sv,e}/S_e > [F_{bst,d}/(2y_{0,d}\text{-}0.2y_{0,d})]/\sigma_e$ i.	**3.00**	mm^2/mm/	**1000**	mm	
Provide shear links $A_{sv,e}/S_e > [\Sigma F_{bst,w}/(\Sigma 2y_{0,w}\text{-}\Sigma 0.2y_{0,w})]/$	**4.43**	mm^2/mm/	**500**	mm	
Provide shear links $A_{sv,e}/S_e > [\Sigma F_{bst,d}/(\Sigma 2y_{0,d}\text{-}\Sigma 0.2y_{0,d})]/$	**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 Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	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.*
	Min prestress tendon(s) clear spacing, S_T ≥ MAX (2$D_{T,H}$ pre-T or $D_{T,H}$ post-T, 5		N/A	mm	**N/A**
	Note $S_T = (b_w - 2.cover - 2.\phi_{link} - D_{T,H})/(N_T/n_{layers,PT} - 1) - D_{T,H}$;				*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 $S_T = (b_w - 2.cover - 2.\phi_{link} - D_{T,H})/(N_T/n_{layers,PT} - 1) - D_{T,H}$;				*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 = $(b_w - 2.cover - 2.\phi_{link} - \phi_t)/(n_t/n_{layers,tens} - 1)$;				
	Max untensioned hogging steel reinforcement pitch (<=3.h, <=500mm)		95	mm	**OK**
	Note max untensioned hogging steel reinforcement pitch $(b_w - 2.cover - 2.\phi_{link} - \phi_t)/(n_t/n_{layers,tens} - 1)$;				
	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		94	mm	**OK**
	Note min untensioned sagging steel reinforcement pitch = $(b_w - 2.cover - 2.\phi_{link} - \phi_t)/(n_t/n_{layers,tens} - 1)$;				
	Max untensioned sagging steel reinforcement pitch (<=3.h, <=500mm)		94	mm	**OK**
	Note max untensioned sagging steel reinforcement pitch $(b_w - 2.cover - 2.\phi_{link} - \phi_t)/(n_t/n_{layers,tens} - 1)$;				
	% Min [SLS] untensioned reinforcement, $A_{s,prov}/(b_w.h)$	*@ Support Top*	0.63	%	BS8110 ▼
	BD:- % Min [SLS] untensioned reinforcement (>= 0.0000$b_w h$)	**Valid**	0.00	%	*6.10.6 TR.*
	Flat slab hogging:- % Min [SLS] untensioned reinforcement (>= A	**Invalid**			*6.10.5 TR.*
	Un-BD:- % Min [SLS] untensioned reinforcement (>= $A_1/b_w h$)	**Invalid**			*6.10.5 TR.*
	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 reinforcem	**Invalid**			*.6.2.3 ACI*
	AS3600 class T/C:- % Min [SLS] untensioned reinforcement (>= A	**Invalid**			*, 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.*
	Tension zone, $(h-x) = (-f_b . h) / (f_t - f_b)$ for span bottom		N/A	mm	*6.10.5 TR.*
	Tension force, $F_1 = -f_{t/b} . \{x \text{ or } (h-x)\} . (b_w \text{ or } b) / 2$		255	kN	*6.10.5 TR.*
	Tension area, $A_1 = F_1 / [function(f_y)]$; ϕ_t = 20mm @	*288MPa*	886	mm^2	*6.10.5 TR.*
	Flat slab hogging:- % Min [SLS] untensioned reinforcement (>= A	**Invalid**	0.01	%	
	Note tension area, $A_1 = 0.00075 b_w h \times b_w/(2 \times 1.5 \times h + MIN(l_{h,h}, l_{h,b}))$;				*cl.6.10.6 TR.*
	Note concentrate rebar between 1.5 x slab thk either side of column width, extending ≥ 0.2L;				*cl.6.10.6 TR.*
	Un-BD:- % Min [SLS] untensioned reinforcement (>= $A_1/b_w h$)	**Invalid**	0.18	%	
	Note tension area, $A_1 = 0.0024\text{-}0.0032 b_w h$ G250; >= MAX (0.0013-0.0018, 0.0013-0.0018(f_{cu}/4				*l.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**	0.43	%	*.3.5.2 BS8*
	ACI318 beam, 1-way or 2-way slab class C:- % Min [TLS] untensio	**Invalid**			*.5.3.2.1 AC*
	AS3600 class T/C:- % Min [TLS] untensioned reinforcement (>= A	**Invalid**			*, 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.*
	Tension zone, $x = (-f_t . h) / (f_b - f_t)$ at span top		N/A	mm	*6.10.5 TR.*
	Tension force, $F_1 = -f_{b/t} . \{(h-x) \text{ or } x\} . (b_w \text{ or } b) / 2$		614	kN	*6.10.5 TR.*
	Tension area, $A_1 = F_1 / [function(f_y)]$; ϕ_t = 25mm @	*288MPa*	2136	mm^2	*6.10.5 TR.*
	% Min [TLS] untensioned reinforcement utilisation	*@ Support Bottom*	**87%**		**OK**

BS8110

Deflection Criteria Rectangular or Flanged Beam BS8110 ▼

BS8110: The following deflection calculations assume an uncracked section for serviceability classes 1 and 2 and (user defined %) cracked section for serviceability class 3; *cl.4.3.6.2 BS8*

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 ACI* *cl.24.2.3.9 ACI*

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 A* *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**
Multiplier for span more or less than 10m $C_{1,2}$	Include if relevant ▼		10/span	**Note**
Multiplier for flat slab $C_{1,3}$		Exclude	1.0	**Note**
Creep + live load deflection criteria	Brittle finishes L/500 ▼			**Note**

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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**

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Inclusion of $\Sigma\delta_{limit,max}$ Include ▼ **Note**

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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. (>= 0.0024-0.0032$b_w h$ G250; >= MAX (0.0013-0.0018, 0			*cl.3.1.7 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. (<= 0.04$b_w h$)			
% Max tensioned and untensioned reinf. utilisation	**24%**		**OK**

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Min shear link diameter, ϕ_{link} (>=6mm)	10	mm	**OK**
Shear link pitch, S	100	mm	**OK**
Note require S (<=0.75d_{max} (<=0.50d_{max} if $V_d>1.8\phi V_c$), <=4b_w, <=300mm, >=MAX(100mm, 50+12.5n$_{leg}$			
$A_{sv,prov}$ / (b_w.S) (>0.10% G460; >0.17% G250)	0.63	%	**OK**

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

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

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Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

							BS8110
							BS8110 ▼
110							
Simply Supported					**N/A**		
Continuous (Infinitely, Encastre)					**VALID**		
Cantilever					**N/A**		
S3600							
AS3600		**Duration Term Limit State E =**	**Short Term TLS E_{st}**	**Long Term SLS E_{lt}**	**Long Term In-Service E_{lt}**		
	Elastic and Creep Deflections	**ω =**	**$\omega_{TLS,E/E}$**	**$\omega_{SLS,E/E}$**	**$k_C.(\omega_{DL+SDL})+\omega_{LL}$**		
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.	$\delta_{EL}=\omega L^4/(384EI_{TLS/(SLS/ULS)})+f(DL_{po}$	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 Drape)	**$\omega_{E/L}$ =**	**$\omega_{TLS,E/L}$**	**$\omega_{SLS,E/L}$**	**$k_{C,PT}$.($\omega_{SLS,E/L}$-**		
	Note for simplicity, the prestress deflection (due to drape) calculation excludes						
	the support peak tendon reverse curvature;						
S/S.	Total drape, e_d = 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.e_d/L^2			N/A		kN/m	
Cont.	Total drape, e_d = 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.e_d/L^2			-104.8		kN/m	
Cant.	Total drape, e_d = 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.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**	**$k_{C,PT}$.(KP_0-P')**		
S/S.	$\delta_{PT,E}$=–[P' or KP_0].(e_{LHS}+e_{RHS})/2.L^2/(8$EI_{TLS/(SL}$		**N/A**	**N/A**	**N/A**	mm	
Cont.	$\delta_{PT,E}$=–[P' or KP_0].(e_{LHS}+e_{RHS})/2.L	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' or KP_0].e_{RHS}.L^2/(2$EI_{TLS/(SLS/ULS)}$)		**N/A**	**N/A**	**N/A**	mm	
	Note for simplicity, E/L effects due to any change of section not computed;						**Note**
9							
	Total Deflection						
	$\Sigma\delta = \delta_{EL} + \delta_{PT,D} + \delta_{PT,E}$		**-1.2**	**2.2**	**3.4**	mm	
		Max Defl'n	**Max Upward**	**Max ownward**	**Max Creep+LL**		
		$\Sigma\delta_{limit}$	-L/350	L/250	L/500	$C_{1,1}$.$C_{1,2}$.$C_{1,3}$	
		$\Sigma\delta_{limit,max}$	-20.0	-	20.0	mm	
));	$\Sigma\delta_{limit}$		**-20.0**	**32.0**	**16.0**	mm	
	$\Sigma\delta$ / $\Sigma\delta_{limit}$		**6%**	**7%**	**21%**		**OK**
ar;							

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), $M_{u,s} = F_{t,s,t}.z_t = F_{t,s,t}.(d_{ps}–0.45x)$	**1803**	kNm	**OK**
Ultimate moment of resistance (concrete), $M_{u,c} = F_{c,c}.z_c = F_{c,c}.(d_{ps}–0.45x)$	**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, $0.9x \le h_f$?	**N/A**		
Total tensioned steel tensile strain, $\varepsilon_{t,s,t} = \varepsilon_{p,s,t} + \varepsilon_{b,s,t}$	**0.0101**		
Prestress strain, $\varepsilon_{p,s,t} = [KP_0/(N_T.N_s.A_s)] / E_p$	0.0058		
Bending strain, $\varepsilon_{b,s,t} = [(d_{ps}\text{-}x)/x].\varepsilon_{cu}$	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

- $\varepsilon_{t,s,t} \le 0.005$ [Not Yielded]

  $\Rightarrow \sigma_{t,s,t} = \varepsilon_{t,s,t}.E_p$

- $0.005 < \varepsilon_{t,s,t} < 0.005 + \dfrac{f_{pk}/\gamma_m}{E_p}$ [Partially Yielded]

  $$\Rightarrow \sigma_{t,s,t} = 0.8f_{pk}/\gamma_m + \frac{\left(f_{pk}/\gamma_m - 0.8f_{pk}/\gamma_m\right)}{\left(0.005 + \dfrac{f_{pk}/\gamma_m}{E_p} - 0.005\right)}\left(\varepsilon_{t,s,t} - 0.005\right)$$

- $\varepsilon_{t,s,t} \ge 0.005 + \dfrac{f_{pk}/\gamma_m}{E_p}$ [Fully Yielded]

  $\Rightarrow \sigma_{t,s,t} = f_{pk}/\gamma_m$

Total tensioned steel tensile force, $F_{t,s,t} = \sigma_{t,s,t}.N_T.N_s.A_s$	**2697**	kN	**OK**
Total concrete compressive force, $F_{c,c}$	**2697**	kN	**OK**
Note $F_{c,c}$ = $0.45f_{cu}.b_w.(0.9x)$ for rect- section or T- or L- sections (with hogging)			
Note $F_{c,c}$ = $\{0.45f_{cu}.b.(0.9x)$ if $0.9x \le h_f$ or $0.45f_{cu}.(b\text{-}b_w).h_f+0.45f_{cu}.b_w.0.9x$ if $0.9x>h_f\}$ for T-			
Ultimate moment of resistance at design section, ϕM_u = ± ϕAVERAGE($M_{u,s}$, M_u	**-1803**	kNm	
Ultimate moment of resistance at design section utilisat **Converged**	**84%**		**OK**
Ultimate moment of resistance at design section, ϕM_u	**-1544**	kNm	

Codified Approach (Bonded and Unbonded Tendons)

$$M_u = f_{pb}A_{ps}\left(d_{ps} - 0.45x\right) \text{ [Rectangular] or [Flanged - NA in Flange]}$$

cl.4.3.7.3

$$M_u = f_{pb}\left(A_{ps} - A_{pf}\right)\left(d_{ps} - 0.45x\right) + 0.45f_{cu}\left(b - b_w\right)h_f\left(d_{ps} - 0.45h_f\right) \text{ [Flanged - NA in Web]}$$

cl.7.3.2 Krishna

Area of prestress tendon(s), $A_{ps} = N_T.N_s.A_s$	1680	mm^2	*cl.4.3.7.4*
Equiv. area of prestress for flange, $A_{pf} = 0.45f_{cu}.(b\text{-}b_w).(h_f/f_{pk})$	N/A	mm^2	*.2 Krishna*
Ratio, $[f_{pu}A_{ps}]/[f_{cu}bd]$	0.21		
Note $[f_{pu}A_{ps}]/[f_{cu}bd] = [f_{pk}A_{ps}]/[f_{cu}b_wd_{ps}]$ [Rectangular];			*cl.4.3.7.3*
Note $[f_{pu}A_{ps}]/[f_{cu}bd] = [f_{pk}A_{ps}]/[f_{cu}bd_{ps}]$ [Flanged - NA in Flange];			*cl.7.3.2 Krishna*
Note $[f_{pu}A_{ps}]/[f_{cu}bd] = [f_{pk}(A_{ps}\text{-}A_{pf})]/[f_{cu}b_wd_{ps}]$ [Flanged - NA in Web];			*cl.7.3.2 Krishna*
Ratio, $f_{pe}/f_{pu} = KP_0/[N_T.N_s.A_s]/f_{pk} \le 0.60$	0.58		*T.4.4*

	BD	Un-BD	
Ratio, $f_{pb}/0.95f_{pu} = f_{pb}/0.95f_{pk}$	0.79	N/A	*T.4.4*
Ratio, $x/d = x/d_{ps}$	0.48	N/A	**OK**

Un-

$$f_{pb} = f_{pe} + \frac{7000}{L/d_{ps}}.\left(1 - 1.7\frac{f_{pu}A_{ps}}{f_{cu}bd}\right) \le 0.7f_{pu} = 0.7f_{pk},\ x = 2.47\left[\frac{f_{pu}A_{ps}}{f_{cu}bd}\frac{f_{pb}}{f_{pu}}d\right] = 2.47\left[\frac{f_{pu}A_{ps}}{f_{cu}bd}\frac{f_{pb}}{f_{pk}}d_{ps}\right]$$

Ultimate moment of resistance at design section utilisation, $M_{ULS}/\phi M_u$	**98%**	**OK**

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Member Design - PC Beam and Slab
Made by **XX** Date **18/08/2025** Chd.

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Bending at Design Section Rectangular or Flanged Beam (Tensioned and Untensioned Rein **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), $M_{u,s}$ **2424** kNm **OK**

Note $M_{u,s} = F_{t,s,t}.z_t + F_{t,s,u}.z_u = F_{t,s,t}.(d_{ps} - 0.45x) + F_{t,s,u}.(d_{rb} - 0.45x);$

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, $\varepsilon_{t,s,t} = \varepsilon_{p,s,t} + \varepsilon_{b,s,t}$ **0.0076**

Prestress strain, $\varepsilon_{p,s,t} = [KP_0/(N_T.N_s.A_s)] / E_p$ 0.0058

Bending strain, $\varepsilon_{b,s,t} = [(d_{ps}-x)/x].\varepsilon_{cu}$ 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

- $\varepsilon_{t,s,t} \leq 0.005$ [Not Yielded]

$$\Rightarrow \sigma_{t,s,t} = \varepsilon_{t,s,t}.E_p$$

- $0.005 < \varepsilon_{t,s,t} < 0.005 + \frac{f_{pk}/\gamma_m}{E_p}$ [Partially Yielded]

$$\Rightarrow \sigma_{t,s,t} = 0.8f_{pk}/\gamma_m + \frac{\left(f_{pk}/\gamma_m - 0.8f_{pk}/\gamma_m\right)}{\left(0.005 + \frac{f_{pk}/\gamma_m}{E_p} - 0.005\right)}\left(\varepsilon_{t,s,t} - 0.005\right)$$

- $\varepsilon_{t,s,t} \geq 0.005 + \frac{f_{pk}/\gamma_m}{E_p}$ [Fully Yielded]

$$\Rightarrow \sigma_{t,s,t} = f_{pk}/\gamma_m$$

Total tensioned steel tensile force, $F_{t,s,t} = \sigma_{t,s,t}.N_T.N_s.A_s$ **2546** kN **OK**

Total concrete compressive force, $F_{c,c}$ **3922** kN **OK**

Note $F_{c,c} = 0.45f_{cu}.b_w.(0.9x)$ *for rect- section or T- or L- sections (with hogging);*

Note $F_{c,c} = \{0.45f_{cu}.b.(0.9x)$ *if* $0.9x \leq h_f$ *or* $0.45f_{cu}.(b-b_w).h_f + 0.45f_{cu}.b_w.0.9x$ *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 **OK**

Ultimate moment of resistance at design section utilisat **Converged** **63%** **OK**

Ultimate moment of resistance at design section, ϕM_u **-971** **-1953** kNm **OK**

Codified Approach (Bo-nded and Unbonded Tendons) **Un-**

$M_u = f_{pb}A_{ps}(d_{cen} - 0.45x)$ [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] *.7.3.2 Krishna*

Equiv. area of prestress tendon(s), $A_{ps} = N_T.N_s.A_s+A_{s,prov}.f_y/f_{pk}$ 2457 mm^2 *cl.4.3.7.4*

Equiv. area of prestress for flange, $A_{pf} = 0.45f_{cu}.(b-b_w).(h_f/f_{pk})$ N/A mm^2 *.2 Krishna*

Ratio, $[f_{pu}A_{ps}]/[f_{cu}bd]$ 0.30

Note $[f_{pu}A_{ps}]/[f_{cu}bd] = [f_{pk}A_{ps}]/[f_{cu}b_w d_{cen}]$ *[Rectangular];* *cl.4.3.7.3*

Note $[f_{pu}A_{ps}]/[f_{cu}bd] = [f_{pk}A_{ps}]/[f_{cu}bd_{cen}]$ *[Flanged - NA in Flange];* *cl.7.3.2 Krishna*

Note $[f_{pu}A_{ps}]/[f_{cu}bd] = [f_{pk}(A_{ps}-A_{pf})]/[f_{cu}b_w d_{cen}]$ *[Flanged - NA in Web];* *cl.7.3.2 Krishna*

Ratio, $f_{pe}/f_{pu} = KP_0/[N_T.N_s.A_s+A_{s,prov}.f_y/f_{pk}]/f_{pk} \leq 0.60$ 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**

$$f_{pb} = f_{pe} + \frac{7000}{L/d_{cen}}.\left(1 - 1.7\frac{f_{pu}A_{ps}}{f_{cu}bd}\right) \leq 0.7f_{pu} = 0.7f_{pk},\ x = 2.47\left[\frac{f_{pu}A_{ps}}{f_{cu}bd}\frac{f_{pb}}{f_{pu}}d\right] = 2.47\left[\frac{f_{pu}A_{ps}}{f_{cu}bd}\frac{f_{pb}}{f_{pk}}d_{cen}\right]$$

Ultimate moment of resistance at design section utilisation, $M_{ULS}/\phi M_u$ **78%** **OK**

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First Principles Approach (Bonded Tendons Only)

Eff. depth to untensioned reinf., d_{rb}	917	mm	

Sagging Moment **Invalid** **Hogging Moment** **Valid**

d_{rb} = h-cover-ϕ_{link}-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 [exterior untensioned reinforcement];

d_{rb} = h-cover-MAX(ϕ_{link}, $cover_{add}$)-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 [exterior untensioned reinforc

Eff. depth to centroid of tensioned and untensioned reinf., d_{cen}	864	mm	*cl.4.3.8.1*

Note d_{cen} = [N_T.N_s.A_s.d_{ps}+$A_{s,prov}$.f_y/f_{pk}.d_{rb}]/[N_T.N_s.A_s+$A_{s,prov}$.f_y/f_{pk}];

Eff. depth to max of tensioned and untensioned reinf., d_{max}	917	mm	*cl.4.3.8.1*

Note d_{max} = MAX (d_{ps}, d_{rb});

Total untensioned steel tensile strain, $\varepsilon_{t,s,u}$ = $\varepsilon_{b,s,u}$	**0.0023**	
Bending strain, $\varepsilon_{b,s,u}$ = [(d_{rb}-x)/x].ε_{cu}	0.0023	
Total untensioned steel tensile stress, $\sigma_{t,s,u}$	**438**	N/mm^2
Untensioned steel yielded ?	**Fully Yielded**	

Figure 2.2 BS8110-1

- $\varepsilon_{t,s,u} \le \dfrac{f_y/\gamma_m}{E_s}$ [Not Yielded]

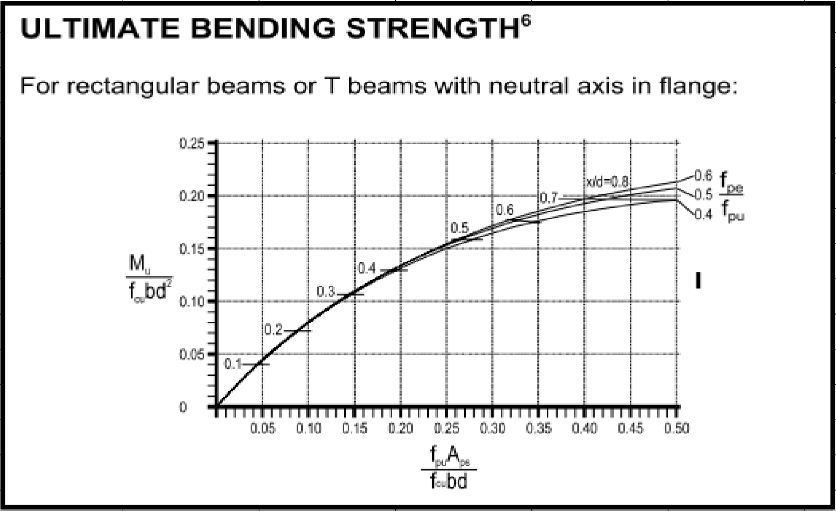
$\Rightarrow \sigma_{t,s,u} = \varepsilon_{t,s,u}.E_s$

- $\varepsilon_{t,s,u} > \dfrac{f_y/\gamma_m}{E_s}$ [Fully Yielded]

$\Rightarrow \sigma_{t,s,u} = f_y/\gamma_m$

Total untensioned steel tensile force, $F_{t,s,u}$ = $\sigma_{t,s,u}$.$A_{s,prov}$	**1376**	kN	**OK**

Codified Approach (Bonded and Unbonded Tendons)



ULTIMATE BENDING STRENGTH[6]

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

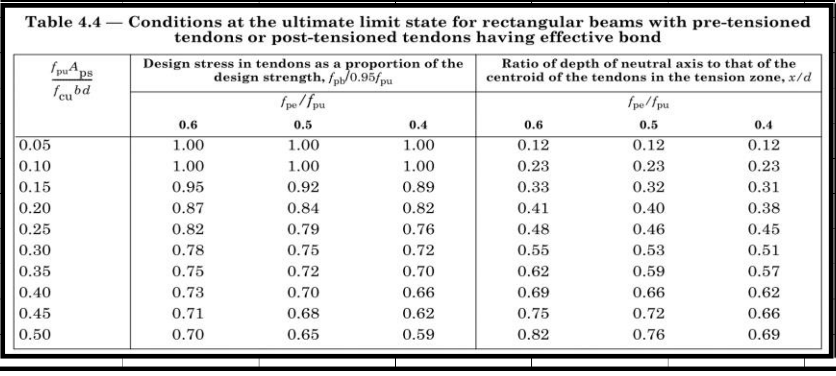


Table 4.4 — Conditions at the ultimate limit state for rectangular beams with pre-tensioned tendons or post-tensioned tendons having effective bond

$\frac{f_{pu}A_{ps}}{f_{cu}bd}$	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

BD

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Bending at All Sections Rectangular or Flanged Beam (Tensioned and Untensioned Reinfor | **Note**

$M_{ULS,var}$ and e_{var} at All Sections

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
$.9x_{var} \leq h_f$?	N/A	N/A	N/A	Yes	Yes	Yes	Yes	
$\varepsilon_{p,s,t}$	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058	
$\varepsilon_{b,s,t,var}$	0.0018	0.0016	0.0010	0.0089	0.0128	0.0155	0.0168	
$\varepsilon_{t,s,t,var}$	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	
$\sigma_{t,s,t,var}$	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
$\varepsilon_{b,s,u,var}$	0.0023	0.0023	0.0024	0.0183	0.0183	0.0183	0.0183	
$\varepsilon_{t,s,u,var}$	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	
$\sigma_{t,s,u,var}$	438	438	438	438	438	438	438	N/mm^2
$F_{t,s,u,var}$	1376	1376	1376	1075	1075	1075	1075	kN
$\phi M_{u,var}$	**-2424**	**-2328**	**-2047**	**2315**	**2823**	**3162**	**3331**	kNm
Converg'n	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	
UT	**63%**	**41%**	**23%**	**12%**	**29%**	**38%**	**41%**	**%**
Status	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	

Goal Seek

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
$.9x_{var} \leq h_f$?	Yes	Yes	Yes	Yes	N/A	N/A	N/A	
$\varepsilon_{p,s,t}$	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058	0.0058	
$\varepsilon_{b,s,t,var}$	0.0168	0.0155	0.0128	0.0089	0.0010	0.0016	0.0018	
$\varepsilon_{t,s,t,var}$	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	
$\sigma_{t,s,t,var}$	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
$\varepsilon_{b,s,u,var}$	0.0183	0.0183	0.0183	0.0183	0.0024	0.0023	0.0023	
$\varepsilon_{t,s,u,var}$	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	
$\sigma_{t,s,u,var}$	438	438	438	438	438	438	438	N/mm^2
$F_{t,s,u,var}$	1075	1075	1075	1075	1376	1376	1376	kN
$\phi M_{u,var}$	**3331**	**3162**	**2823**	**2315**	**-2047**	**-2328**	**-2424**	kNm
Converg'n	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	**Yes**	
UT	**41%**	**38%**	**29%**	**12%**	**23%**	**41%**	**63%**	**%**
Status	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	**OK**	

Goal Seek

irst Principles Approach
(Bonded Tendons Only)

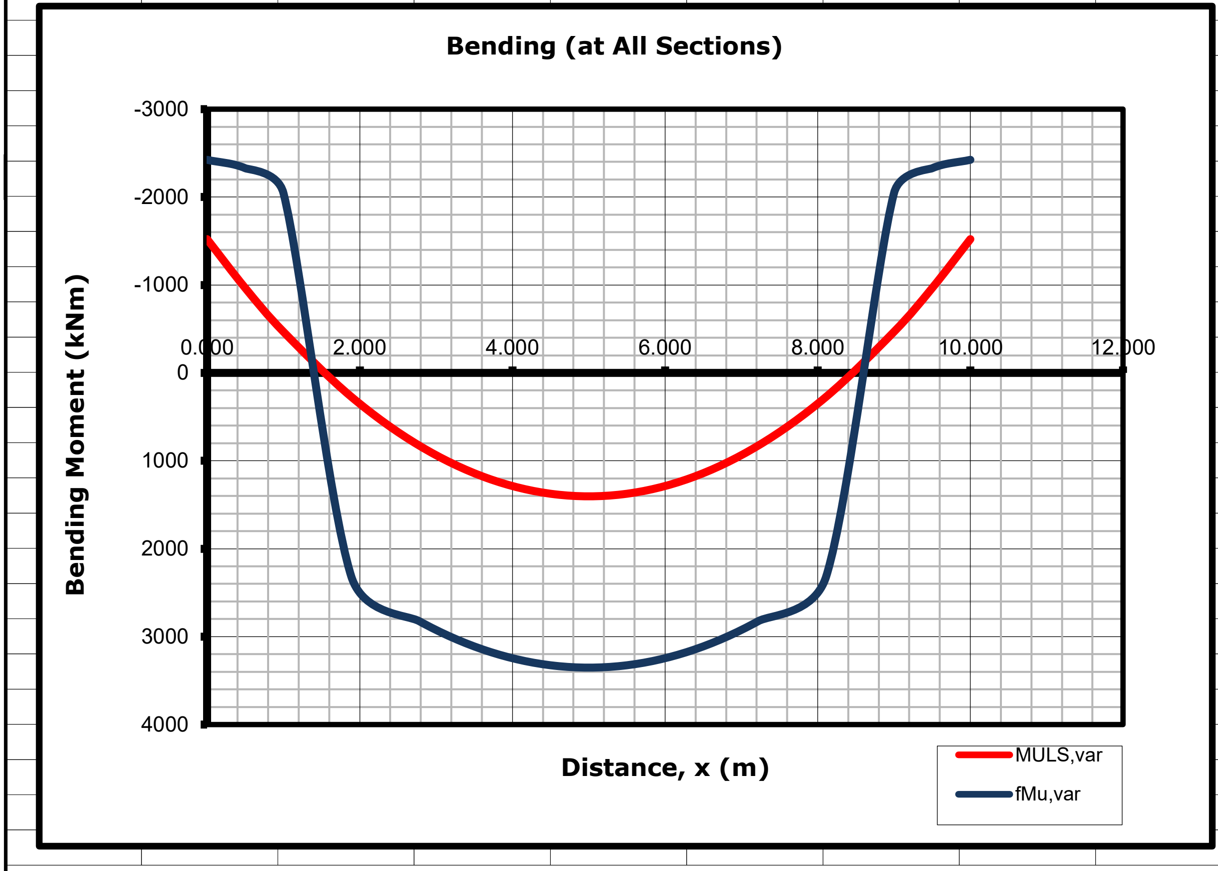
Note by convention, a negative bending moment indicates hogging moment; Note above $M_{ULS,var} = M_{ULS,E/E,var}$

Ultimate moment of resistance utilisation, MAX ($M_{ULS,var}/M_{u,var}$) **63%** **OK**

Convergence of moment of resistance equations **Converged**

BS8110

BS8110 ▼



Note by convention, a negative bending moment indicates hogging moment; Note above $M_{ULS,var} = M_{ULS,E/E,var}$ $+ M_{SLS,S/E,var}$;

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Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	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/r **3.01** N/mm^2 3.4.5.2, cl.

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.

Breadth, b_v = b_w–(2/3 BD, 1 un-BD).N_T.MAX($D_{T,H}$,$D_{T,V}$) Exclude duct ▼ 500 mm cl.4.3.8.1

ULS bending moment at critical section, M_{ult} = $M_{ULS,E/E,var}$(x=0) + $M_{SLS,S/E,var}$(x= **-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-ϕ_{link}-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 *[exterior untensioned reinforcement];*

Hog d_{rb} = h-cover-MAX(ϕ_{link}, $cover_{add}$)-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 *[exterior untensioned reinforc*

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

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-ϕ_{link}-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 *[exterior untensioned reinforcement];*

Hog d_{rb} = h-cover-MAX(ϕ_{link}, $cover_{add}$)-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 *[exterior untensioned reinforc*

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

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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]

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Uncracked design shear resistance, V_{co}			757	kN	cl.4.3.8.4
	$V_{co} = 0.67 b_v h \sqrt{(f_t^2 + 0.8 f_{cp} f_t)} + V_p$				
Uncracked design shear strength,	$V_{co}/b_v h$		1.51	N/mm^2	
	Vertical component of prestress force, $V_p = \gamma_p.KP_0\sin\beta$		131	kN	cl.4.3.8.4
	Maximum design principal tensile stress, $f_t = 0.24\sqrt{f_{cu}}$, $f_{cu} \leq 80N/mm^2$		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
	$V_{cr} = (1 - 0.55\frac{f_{pe}}{f_{pu}})\, v_c b_v d + M_o \frac{V}{M} \geq 0.1 b_v d\sqrt{f_{cu}}$ *Note V, M and d refer to V_d, $\lvert M_d\rvert$ and $d_{cen,d}$ respectively;*				
	$v_c = (0.79/1.25)(\rho_w f_{cu}/25)^{1/3}(400/d_{cen,d})^{1/4}$; $\rho_w<3$; $f_{cu}<80$; $(400/d_{cen,d})^{1/4}>(0.67$ or				
	Vertical component of prestress force, $V_p = \gamma_p.KP_0\sin\beta$		N/A	kN	
	Component $(1-0.55f_{pe}/f_{pu}).v_c b_v d_{cen,d}$		247	kN	cl.4.3.8.5
	Component $M_0.V/M$		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	
ement];	*AS3600*	*N/A*	*N/A*	kN	
	Ratio, $f_{pe}/f_{pu} = KP_0/[N_T.N_s.A_s+A_{s,prov}.f_y/f_{pk}]/f_{pk} \leq 0.60$		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 reinforcemen				
.8.8	$N_T.N_s.A_s+A_{s,prov}$		4822	mm^2	cl.4.3.8.1
	$\rho_w = 100(N_T.N_s.A_s+A_{s,prov})/b_w d_{cen,d}$		1.12	%	cl.4.3.8.1
	Bending moment for zero tensile stress, $M_0 = 0.8 f_{pt} Z_{b/t,(SLS/ULS)}$		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 $Z_{b/t,(SLS/ULS)}$ prope				
Shear enhancement near support, $k_{enh} = 2d_{cen,d}/x_d$		Exclude ▼	1.00		cl.3.4.5.8, cl.4.
Design shear resistance, V_c = {V_{co} uncracked, MIN (V_{co}, V_{cr}) cracked}			670	kN	cl.4.3.8.3
	Uncracked section ($\lvert M_d\rvert < M_{0/ct}$) 1521 < 494	kNm	Invalid		
	Cracked section ($\lvert M_d\rvert \geq M_{0/ct}$) 1521 ≥ 494	kNm	Valid		
Minimum shear strength, v_r = MAX (0.4, $0.4(f_{cu}/40)^{2/3}$), $f_{cu} \leq 80N/mm^2$			0.40	N/mm^2	.3.4.5.3 BC
Check $V_d < 0.5k_{enh}.V_c$ (beam) (minor elements) or $1.0k_{enh}.V_c$ (slab) fo			INVALID	Beam	cl.4.3.8.6
	$k_{enh}.V_c$		670	kN	cl.4.3.8.5,
Check 0.0 (beam) or $1.0k_{enh}.V_c$ (slab) < V_d < $k_{enh}.V_c$+NL for nominal li			N/A		cl.4.3.8.7
	$A_{sv,nom}/S > v_r.b_v/(0.95f_{yv})$, $f_{yv} \leq 460N/mm^2$ i.e. $A_{sv,nom}/S >$		0.46	mm^2/mm	3.8.7, cl.4.
	$k_{enh}.V_c+NL = v_r.b_v d_{cen,d} + k_{enh}.V_c$		843	kN	cl.3.4.5.8,
Check $V_d > k_{enh}.V_c$+NL for design links			VALID		cl.4.3.8.8
	$A_{sv}/S > (V_d-k_{enh}.V_c)/(0.95f_{yv}.d_{max,d})$, $f_{yv} \leq 460N/mm^2$ i.e. $A_{sv}/S >$		1.57	mm^2/mm	cl.3.4.5.8,
	$k_{enh}.V_c+DL = (A_{sv,prov}/S).(0.95f_{yv}).d_{max,d} + k_{enh}.V_c$, $f_{yv} \leq 460N/mn$		1929	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			67%		OK

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Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	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
$f_{cp/pc}[\sigma_{cp}]$	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	
$\rho_{w,var}$	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}	**1.00**	**1.00**	**1.00**	**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	
$k_{enh}.\phi V_{c,var}$	**670**	**724**	**739**	**1024**	**602**	**432**	**300**	kN
$\phi V_{c,var}+k_{enl}$	**843**	**892**	**893**	**1152**	**755**	**602**	**478**	kN
$\phi V_{c,var}+k_{enl}$	**1929**	**1983**	**1998**	**2310**	**1888**	**1718**	**1586**	kN
No Links	**INVALID**	**INVALID**	**INVALID**	**INVALID**	**INVALID**	**INVALID**	**VALID**	
Nom Links	**N/A**	**N/A**	**VALID**	**VALID**	**VALID**	**VALID**	**VALID**	
Des Links	**VALID**	**VALID**	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	
A_{sv}/S >	1.57	0.82	0.46	0.46	0.46	0.46	0.46	mm^2/mm
UT	**67%**	**53%**	**45%**	**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
$f_{cp/pc}[\sigma_{cp}]$	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	
$\rho_{w,var}$	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**	**1.00**	**1.00**	**1.00**	
$V_{cr/ci/uc,var}$	**-300**	**-432**	**-602**	**-916**	**-739**	**-724**	**-670**	kN
Cracked?	Yes	Yes	Yes	No	Yes	Yes	Yes	
$k_{enh}.\phi V_{c,var}$	**-300**	**-432**	**-602**	**-1024**	**-739**	**-724**	**-670**	kN
$\phi V_{c,var}+k_{enl}$	**-478**	**-602**	**-755**	**-1152**	**-893**	**-892**	**-843**	kN
$\phi V_{c,var}+k_{enl}$	**-1586**	**-1718**	**-1888**	**-2310**	**-1998**	**-1983**	**-1929**	kN
No Links	**VALID**	**INVALID**	**INVALID**	**INVALID**	**INVALID**	**INVALID**	**INVALID**	
Nom Links	**VALID**	**VALID**	**VALID**	**VALID**	**VALID**	**N/A**	**N/A**	
Des Links	**N/A**	**N/A**	**N/A**	**N/A**	**N/A**	**VALID**	**VALID**	
A_{sv}/S >	0.46	0.46	0.46	0.46	0.46	0.82	1.57	mm^2/mm
UT	**30%**	**35%**	**40%**	**50%**	**45%**	**53%**	**67%**	**%**
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	**67%**	**OK**

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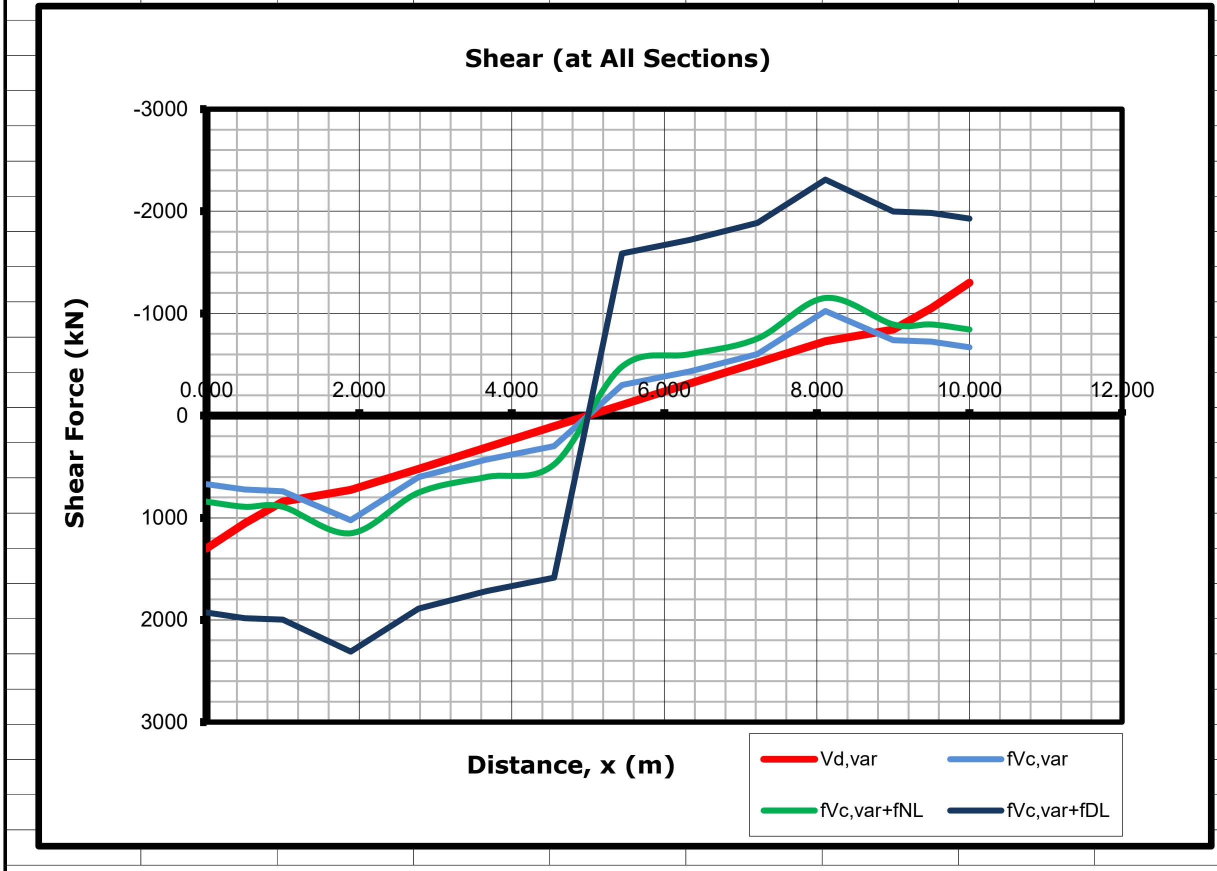

Note an arbitrary shear force sign convention is employed; Note above $V_{d,var} = V_{ULS,E/E,var} + V_{SLS,S/E,var}$;

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BS8110 [PC

Punching Shear at Column Support Rectangular Beam **Note**

Punching Shear at Column Support 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**

Punching Shear at Column Support Add. Parameters Options

Include or exclude secondary effects in punching shear force computation — Include ▼

Include punching shear force reduction within perimeter ? — Include 100% ▼ **Note**

Location for calculation of eff. depth, d_{cen} | d_{rb} | d_{max} ? — At shear perimeter ▼ **Note**

Location for calculation of ecc. of prestress force, e* ? — At shear perimeter ▼ **Note**

Inclusion of the $M_0V_{ult}/|M_{ult}|$ term ? — Include ▼

Punching Shear at Column Support Rectangular Beam

ULS shear force at critical section, V_{ult} = ABS ($V_{ULS,E/E}$ + $V_{SLS,S/E}$)	**N/A**	kN	
ULS bending moment at critical section, M_{ult} = $M_{ULS,E/E,var}$(x=0) + $M_{SLS,S/E,var}$(x=	**N/A**	kNm	

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

Ratio, $V_{ult}/|M_{ult}|$ (usually 5.5/L to 6.0/L for int. columns, cl.6.11.2 TR.43)	N/A	/L	
ULS punching shear into column, V_t = 2V_{ult} (internal), V_{ult} (edge), V_{ult} (corner)	**N/A**	kN	

Note full column tributary punching shear force, V_t = 2V_{ult} *(internal),* V_{ult} *(edge),* V_{ult} *(corner);*

Eff. depth to $A_{s,prov,h}$., d_{rb} — N/A mm — *N/A*

Note d_{rb} = h-cover-MAX(ϕ_{link}, $cover_{add}$)-[ϕ_t+($n_{layers,tens}$-1)(ϕ_t+$s_{r,tens}$)]/2 *[exterior untensioned reinforcemen*

Column Face Perimeter

Eff. depth to A_s, $d_{ps,1}$ = h-$x_{c,(SLS/ULS)}$-e_{var}(x = $l_{h,h/D}$/2 or $l_{h,b}$/2)		N/A	mm	
Eff. depth to centroid of A_s and $A_{s,prov,h}$, $d_{cen,1}$		N/A	mm	*cl.4.3.8.1*
Eff. depth to max of A_s and $A_{s,prov,h}$, $d_{max,1}$		N/A		*cl.4.3.8.1, cl.4.*
Shear force at column face, V_1 = (V_t-$V_{reduced,1}$)	**N/A**	**N/A**	kN	

			Rectangular	*Circular*	
IC|EC|CC:	*$l_{h,b}$.$l_{h,h}$*	*π.$l_{h,D}^2$/4*	*N/A*	*N/A*	m^2

Eff. shear force, $V_{eff,1}$ = (1.15 int., 1.40 edge, 1.50 corner column) . V_1	**N/A**	kN	*cl.3.7.6*
Column face perimeter, u_1	N/A	mm	*cl.3.7.6.1*

			Rectangular	*Circular*	
IC:	*2.($l_{h,b}$+$l_{h,h}$)*	*π.$l_{h,D}$*	*N/A*	*N/A*	mm
EC:	*2$l_{h,b}$+$l_{h,h}$ or 2$l_{h,h}$+$l_{h,b}$*	*3/4(π.$l_{h,D}$)*	*N/A*	*N/A*	mm
CC:	*($l_{h,b}$+$l_{h,h}$)*	*π.$l_{h,D}$/2*	*N/A*	*N/A*	mm

Shear stress at column face perimeter, v_1 = $V_{eff,1}$ / $u_1d_{cen,1}$ (< 0.8$f_{cu}^{0.5}$ & 5N	**N/A**	N/mm^2	*cl.3.7.7.2*
Ultimate shear strength, MIN{0.8$f_{cu}^{0.5}$ & 5N/mm^2}	**N/A**	N/mm^2	*cl.3.7.7.2*
Ultimate shear stress utilisation	**N/A**		**N/A**

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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 **18/08/2025**	Chd.

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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.*
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:	*$(l_{h,b}+1.5d_{cen,2}).(l_{h,h}+3d_{cen,2})$ o*	*$(l_{h,D}+1.5d_{cen,2}).(l_{h,D}$*	*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, $u_2 \leq$ {2L+2L,L+L,L+L,L/2+L/2}				N/A	mm	*cl.3.7.7.6*
			Rectangular	*Circular*		
IC:	*$2.(l_{h,b}+l_{h,h})+12d_{cen,2}$*	*$4l_{h,D}+12d_{cen,2}$*	*N/A*	*N/A*	mm	
EC:	*$2l_{h,b}+l_{h,h}+6d_{cen,2}$ or $2l_{h,h}+l_{h,b}$*	*$3l_{h,D}+6d_{cen,2}$*	*N/A*	*N/A*	mm	
CC:	*$(l_{h,b}+l_{h,h})+3d_{cen,2}$*	*$2l_{h,D}+3d_{cen,2}$*	*N/A*	*N/A*	mm	
Shear stress at column first perimeter, v_2 = $V_{eff,2}$ / $u_2d_{cen,2}$				**N/A**	N/mm^2	**Note**
Width of design strip first shear perimeter, $b_2 \leq b_w$				**N/A**	mm	
			Rectangular	*Circular*		
IC:	*$(l_{h,b} \mid l_{h,h})+3d_{cen,2}$*	*$l_{h,D}+3d_{cen,2}$*	*N/A*	*N/A*	mm	
EC:	*$(l_{h,b} \mid l_{h,h})+1.5\text{-}3d_{cen,2}$*	*$l_{h,D}+1.5\text{-}3d_{cen,2}$*	*N/A*	*N/A*	mm	
CC:	*$(l_{h,b} \mid l_{h,h})+1.5d_{cen,2}$*	*$l_{h,D}+1.5d_{cen,2}$*	*N/A*	*N/A*	mm	
		Hog Steel	Tendons			
$\rho_{w,2}$ = 100.N_T.N_s.A_s/$b_wd_{cen,2}$ + 100.$A_{s,prov,h}$/$b_wd_{cen,2}$		N/A	N/A	N/A	%	**Note**
$v_{c,2}$ = (0.79/1.25)($\rho_{w,2}f_{cu}$/25)$^{1/3}$(400/$d_{cen,2}$)$^{1/4}$, $\rho_{w,2}$<3, f_{cu}<40, (400/$d_{cen,2}$)				**N/A**	N/mm^2	*cl.3.4.5.4*
$V_{co,2}$ = 0.67b_2h√(f_t^2+0.8$f_{cp}f_t$), f_t=0.24√f_{cu}, f_{cp}=KP_0/$A_{(SLS/ULS)}$, $f_{cu}\leq40N/mm^2$				**N/A**	kN	*6.11.2 TR.*
$V_{cr,2}$ = $v_{c,2}b_2d_{cen,2}$ + $M_{0,2}V_{ult}$/|M_{ult}| ≥ 0.1$b_2d_{cen,2}$√f_{cu}, $f_{cu}\leq$40N/mm			**N/A**	**N/A**	kN	*6.11.2 TR.*
	Decompression, $M_{0,2}$=0.8(KP_0/$A_{(SLS/ULS)}$).$Z^*_{t,(SLS/ULS)}$–0.8KP_0*e*			N/A	kNm	
	Z_t for b_2, $Z^*_{t,(SLS/ULS)}$ = $I^*_{(SLS/ULS)}$/$x^*_{c,(SLS/ULS)}$ = (b_2.h^3/12)/(h/2)			N/A	$x10^3cm^3$	
	Prestress force at SLS over b_2 only, KP_0* = KP_0.b_2/b_w			N/A	kN	
	Ecc. of prestress force, e*			N/A	mm	
	Note e = $x_{c,(SLS/ULS)}$ + e_{var}(x=@col face to shear perimeter) - [$x^*_{c,(SLS/ULS)}$=h/2];*					
$V_{c,2}$ = {$V_{co,2}$ uncracked, MIN ($V_{co,2}$, $V_{cr,2}$) cracked}		N/A	N/A	**N/A**	kN	*6.11.2 TR.*
$V_{c,2}$/$b_2d_{cen,2}$				**N/A**	N/mm^2	*cl.4.3.8.1*
	Case v_2 < $V_{c,2}$/$b_2d_{cen,2}$			**N/A**	**N/A**	*cl.3.7.7.6*
	No links required.					
	Case $V_{c,2}$/$b_2d_{cen,2}$ < v_2 < 1.6$V_{c,2}$/$b_2d_{cen,2}$		**N/A**	**N/A**	**N/A**	*cl.3.7.7.5*
				N/A	N/mm^2	
	$\boxed{\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}}$	*$f_{yv}\leq460N/mm^2$*	N/A	>= N/A	mm^2	
		$d=d_{cen,2}$				
	Note $\boxed{\Sigma A_{sv}\sin\alpha \geq}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
	Case 1.6$V_{c,2}$/$b_2d_{cen,2}$ < v_2 < 2.0$V_{c,2}$/b_2d		**N/A**	**N/A**	**N/A**	*cl.3.7.7.5*
				N/A	N/mm^2	
	$\boxed{\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v-v_c)ud}{0.95f_{yv}}}$	*$460N/mm^2$*	N/A	>= N/A	mm^2	
		$d=d_{max,2}$				
	Note $\boxed{\Sigma A_{sv}\sin\alpha \geq}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
	Case v_2 > 2.0$V_{c,2}$/$b_2d_{cen,2}$		**N/A**	**N/A**		*cl.3.7.7.5*
First shear perimeter shear utilisation		**N/A**		**N/A**		**N/A**

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		Member/Location	
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Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025** Chd.

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Second Shear Perimeter	$@2.25d_{cen,}$ **N/A** to	$@0.75d_{cen,}$	**N/A**	mm	*cl.3.7.7.6*
Eff. depth to A_s, $d_{ps,3} = h\text{-}x_{c,(SLS/ULS)}\text{-}e_{var}(x=@\text{shear perimeter})$		**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.*
Shear force at second shear perimeter, $V_3 = (V_t\text{-}V_{reduced,3})$		**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: $(l_{h,b}+2.25d_{cen,3}).(l_{h,h}+4.5d_{cen,}$	$(l_{h,D}+2.25d_{cen,3}).(l_{h,}$	*N/A*	*N/A*	m^2	
CC: $(l_{h,b}+2.25d_{cen,3}).(l_{h,h}+2.25d_{cen}$	$(l_{h,D}+2.25d_{cen,3})^2$	*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, $u_3 \leq \{2L+2L,L+L,L+L,L/2+L/2\}$			N/A	mm	*cl.3.7.7.6*
		Rectangular	*Circular*		
IC: $2.(l_{h,b}+l_{h,h})+18d_{cen,3}$	$4l_{h,D}+18d_{cen,3}$	*N/A*	*N/A*	mm	
EC: $2l_{h,b}+l_{h,h}+9d_{cen,3}$ or $2l_{h,h}+l_{h,b}$	$3l_{h,D}+9d_{cen,3}$	*N/A*	*N/A*	mm	
CC: $(l_{h,b}+l_{h,h})+4.5d_{cen,3}$	$2l_{h,D}+4.5d_{cen,3}$	*N/A*	*N/A*	mm	
Shear stress at column second perimeter, $v_3 = V_{eff,3} / u_3d_{cen,3}$			**N/A**	N/mm^2	**Note**
Width of design strip second shear perimeter, $b_3 \leq b_w$			**N/A**	mm	
		Rectangular	*Circular*		
IC: $(l_{h,b} \mid l_{h,h})+4.5d_{cen,3}$	$l_{h,D}+4.5d_{cen,3}$	*N/A*	*N/A*	mm	
EC: $(l_{h,b} \mid l_{h,h})+2.25\text{-}4.5d_{cen,3}$	$l_{h,D}+2.25\text{-}4.5d_{cen,3}$	*N/A*	*N/A*	mm	
CC: $(l_{h,b} \mid l_{h,h})+2.25d_{cen,3}$	$l_{h,D}+2.25d_{cen,3}$	*N/A*	*N/A*	mm	
	Hog Steel	Tendons			
$\rho_{w,3} = 100.N_T.N_s.A_s/b_wd_{cen,3} + 100.A_{s,prov,h}/b_wd_{cen,3}$	N/A	N/A	N/A	%	**Note**
$v_{c,3} = (0.79/1.25)(\rho_{w,3}f_{cu}/25)^{1/3}(400/d_{cen,3})^{1/4}$, $\rho_{w,3}<3$, $f_{cu}<40$, $(400/d_{cen,3})$			**N/A**	N/mm^2	*cl.3.4.5.4*
$V_{co,3} = 0.67b_3h\sqrt{}(f_t^2+0.8f_{cp}f_t)$, $f_t=0.24\sqrt{}f_{cu}$, $f_{cp}=KP_0/A_{(SLS/ULS)}$, $f_{cu}\leq 40N/mm^2$			**N/A**	kN	*6.11.2 TR.*
$V_{cr,3} = v_{c,3}b_3d_{cen,3} + M_{0,3}V_{ult}/\lvert M_{ult}\rvert \geq 0.1b_3d_{cen,3}\sqrt{}f_{cu}$, $f_{cu}\leq 40N/mm$		**N/A**	**N/A**	kN	*6.11.2 TR.*
Decompression, $M_{0,3}=0.8(KP_0/A_{(SLS/ULS)}).Z^*_{t,(SLS/ULS)}-0.8KP_0{}^*e^*$			N/A	kNm	
Z_t for b_3, $Z^*_{t,(SLS/ULS)} = I^*_{(SLS/ULS)}/x^*_{c,(SLS/ULS)} = (b_3.h^3/12)/(h/2)$			N/A	$x10^3cm^3$	
Prestress force at SLS over b_3 only, $KP_0{}^* = KP_0.b_3/b_w$			N/A	kN	
Ecc. of prestress force, e^*			N/A	mm	
Note $e^* = x_{c,(SLS/ULS)} + e_{var}$ *(x=@col face to shear perimeter) - [*$x^*_{c,(SLS/ULS)}$ *=h/2];*					
$V_{c,3} = \{V_{co,3}$ uncracked, MIN $(V_{co,3}, V_{cr,3})$ cracked$\}$	N/A	N/A	**N/A**	kN	*6.11.2 TR.*
$V_{c,3}/b_3d_{cen,3}$			**N/A**	N/mm^2	*cl.4.3.8.1*
Case $v_3 < V_{c,3}/b_3d_{cen,3}$			**N/A**	**N/A**	*cl.3.7.7.6*
No links required.					
Case $V_{c,3}/b_3d_{cen,3} < v_3 < 1.6V_{c,3}/b_3d_{cen,}$		**N/A**	**N/A**	**N/A**	*cl.3.7.7.5*
			N/A	N/mm^2	
$\boxed{\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}}$ $f_{yv} \leq 460N/mm^2$	N/A	>=	N/A	mm^2	
$d=d_{cen,3}$					
Note $\boxed{\Sigma A_{sv}\sin\alpha \geq}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
Case $1.6V_{c,3}/b_3d_{cen,3} < v_3 < 2.0V_{c,3}/b_3d$		**N/A**	**N/A**	**N/A**	*cl.3.7.7.5*
			N/A	N/mm^2	
$\boxed{\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v-v_c)ud}{0.95f_{yv}}}$ $460N/mm^2$	N/A	>=	N/A	mm^2	
$d=d_{max,3}$					
Note $\boxed{\Sigma A_{sv}\sin\alpha \geq}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
Case $v_3 > 2.0V_{c,3}/b_3d_{cen,3}$		**N/A**	**N/A**		*cl.3.7.7.5*
Second shear perimeter shear utilisation	**N/A**		**N/A**		**N/A**

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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, $d_{ps,4} = h\text{-}x_{c,(SLS/ULS)}\text{-}e_{var}(x\text{=}@shear\ perimeter)$			**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, $V_4 = (V_t\text{-}V_{reduced,4})$			**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 (	$(l_{h,D}+3d_{cen,4}).(l_{h,D}+$	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, $u_4 \leq$ {2L+2L,L+L,L+L,L/2+L/2}				N/A	mm	cl.3.7.7.6
			Rectangular	*Circular*		
IC:	$2.(l_{h,b}+l_{h,h})+24d_{cen,4}$	$4l_{h,D}+24d_{cen,4}$	N/A	N/A	mm	
EC:	$2l_{h,b}+l_{h,h}+12d_{cen,4}$ or $2l_{h,h}+l_{h,l}$	$3l_{h,D}+12d_{cen,4}$	N/A	N/A	mm	
CC:	$(l_{h,b}+l_{h,h})+6d_{cen,4}$	$2l_{h,D}+6d_{cen,4}$	N/A	N/A	mm	
Shear stress at column third perimeter, $v_4 = V_{eff,4} / u_4d_{cen,4}$				**N/A**	N/mm^2	**Note**
Width of design strip third shear perimeter, $b_4 \leq b_w$				**N/A**	mm	
			Rectangular	*Circular*		
IC:	$(l_{h,b}\ \vert\ l_{h,h})+6d_{cen,4}$	$l_{h,D}+6d_{cen,4}$	N/A	N/A	mm	
EC:	$(l_{h,b}\ \vert\ l_{h,h})+3\text{-}6d_{cen,4}$	$l_{h,D}+3\text{-}6d_{cen,4}$	N/A	N/A	mm	
CC:	$(l_{h,b}\ \vert\ l_{h,h})+3d_{cen,4}$	$l_{h,D}+3d_{cen,4}$	N/A	N/A	mm	
		Hog Steel	Tendons			
$\rho_{w,4} = 100.N_T.N_s.A_s/b_wd_{cen,4} + 100.A_{s,prov,h}/b_wd_{cen,4}$		N/A	N/A	N/A	%	**Note**
$v_{c,4} = (0.79/1.25)(\rho_{w,4}f_{cu}/25)^{1/3}(400/d_{cen,4})^{1/4}$, $\rho_{w,4}<3$, $f_{cu}<40$, $(400/d_{cen,4})$				**N/A**	N/mm^2	cl.3.4.5.4
$V_{co,4} = 0.67b_4h\sqrt{}(f_t^2+0.8f_{cp}f_t)$, $f_t=0.24\sqrt{}f_{cu}$, $f_{cp}=KP_0/A_{(SLS/ULS)}$, $f_{cu}\leq 40N/mm^2$				**N/A**	kN	6.11.2 TR.4
$V_{cr,4} = v_{c,4}b_4d_{cen,4} + M_{0,4}V_{ult}/\vert M_{ult}\vert \geq 0.1b_4d_{cen,4}\sqrt{}f_{cu}$, $f_{cu}\leq 40N/mm$			**N/A**	**N/A**	kN	6.11.2 TR.4
	Decompression, $M_{0,4}=0.8(KP_0/A_{(SLS/ULS)}).Z^*_{t,(SLS/ULS)}-0.8KP_0{}^*e^*$			N/A	kNm	
	Z_t for b_4, $Z^*_{t,(SLS/ULS)} = I^*_{(SLS/ULS)}/x^*_{c,(SLS/ULS)} = (b_4.h^3/12)/(h/2)$			N/A	$x10^3cm^3$	
	Prestress force at SLS over b_4 only, $KP_0{}^* = KP_0.b_4/b_w$			N/A	kN	
	Ecc. of prestress force, e^*			N/A	mm	
	Note $e^* = x_{c,(SLS/ULS)} + e_{var}$ *(x=@col face to shear perimeter) - [*$x^*_{c,(SLS/ULS)}$ *=h/2];*					
$V_{c,4}$ = {$V_{co,4}$ uncracked, MIN ($V_{co,4}$, $V_{cr,4}$) cracked}		N/A	N/A	**N/A**	kN	6.11.2 TR.4
$V_{c,4}/b_4d_{cen,4}$				**N/A**	N/mm^2	cl.4.3.8.1
	Case $v_4 < V_{c,4}/b_4d_{cen,4}$			**N/A**	**N/A**	cl.3.7.7.6
	No links required.					
	Case $V_{c,4}/b_4d_{cen,4} < v_4 < 1.6V_{c,4}/b_4d_{cen,4}$		**N/A**	**N/A**	**N/A**	cl.3.7.7.5
				N/A	N/mm^2	
	$\boxed{\Sigma A_{sv}\sin\alpha \geq \frac{(v-v_c)ud}{0.95f_{yv}}}$ $f_{yv} \leq 460N/mm^2$	N/A	>=	N/A	mm^2	
	$d=d_{cen,4}$					
	Note $\boxed{\Sigma A_{sv}\sin\alpha \geq}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
	Case $1.6V_{c,4}/b_4d_{cen,4} < v_4 < 2.0V_{c,4}/b_4d$		**N/A**	**N/A**	**N/A**	cl.3.7.7.5
				N/A	N/mm^2	
	$\boxed{\Sigma A_{sv}\sin\alpha \geq \frac{5(0.7v-v_c)ud}{0.95f_{yv}}}$ $460N/mm^2$	N/A	>=	N/A	mm^2	
	$d=d_{max,4}$					
	Note $\boxed{\Sigma A_{sv}\sin\alpha \geq}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
	Case $v_4 > 2.0V_{c,4}/b_4d_{cen,4}$		**N/A**	**N/A**		cl.3.7.7.5
Third shear perimeter shear utilisation		**N/A**		**N/A**		**N/A**

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Member Design - PC Beam and Slab		Made by XX	Date 18/08/2025		Chd.

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BS8110 [PC

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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:	$(l_{h,b}+3.75d_{cen,5}).(l_{h,h}+7.5d_{cen,5}$	$(l_{h,D}+3.75d_{cen,5}).(l_{h,}$	N/A	N/A	m^2	
CC:	$(l_{h,b}+3.75d_{cen,5}).(l_{h,h}+3.75d_{cen}$	$(l_{h,D}+3.75d_{cen,5})^2$	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, $u_5 \le$ {2L+2L,L+L,L+L,L/2+L/2}				N/A	mm	cl.3.7.7.6
			Rectangular	Circular		
IC:	$2.(l_{h,b}+l_{h,h})+30d_{cen,5}$	$4l_{h,D}+30d_{cen,5}$	N/A	N/A	mm	
EC:	$2l_{h,b}+l_{h,h}+15d_{cen,5}$ or $2l_{h,h}+l_{h,}$	$3l_{h,D}+15d_{cen,5}$	N/A	N/A	mm	
CC:	$(l_{h,b}+l_{h,h})+7.5d_{cen,5}$	$2l_{h,D}+7.5d_{cen,5}$	N/A	N/A	mm	
Shear stress at column fourth perimeter, v_5 = $V_{eff,5}$ / $u_5d_{cen,5}$				N/A	N/mm^2	**Note**
Width of design strip fourth shear perimeter, $b_5 \le b_w$				N/A	mm	
			Rectangular	Circular		
IC:	$(l_{h,b}\ \vert\ l_{h,h})+7.5d_{cen,5}$	$l_{h,D}+7.5d_{cen,5}$	N/A	N/A	mm	
EC:	$(l_{h,b}\ \vert\ l_{h,h})+3.75\text{-}7.5d_{cen,5}$	$l_{h,D}+3.75\text{-}7.5d_{cen,5}$	N/A	N/A	mm	
CC:	$(l_{h,b}\ \vert\ l_{h,h})+3.75d_{cen,5}$	$l_{h,D}+3.75d_{cen,5}$	N/A	N/A	mm	
		Hog Steel	Tendons			
$\rho_{w,5}$ = 100.N_T.N_s.A_s/$b_wd_{cen,5}$ + 100.$A_{s,prov,h}$/$b_wd_{cen,5}$		N/A	N/A	N/A	%	**Note**
$v_{c,5}$ = (0.79/1.25)$(\rho_{w,5}f_{cu}/25)^{1/3}(400/d_{cen,5})^{1/4}$, $\rho_{w,5}$<3, f_{cu}<40, $(400/d_{cen,5})$				N/A	N/mm^2	cl.3.4.5.4
$V_{co,5}$ = $0.67b_5h\sqrt{(f_t^2+0.8f_{cp}f_t)}$, $f_t=0.24\sqrt{f_{cu}}$, $f_{cp}=KP_0/A_{(SLS/ULS)}$, $f_{cu}\le 40N/mm^2$				N/A	kN	6.11.2 TR.4
$V_{cr,5}$ = $v_{c,5}b_5d_{cen,5}$ + $M_{0,5}V_{ult}/\vert M_{ult}\vert \ge 0.1b_5d_{cen,5}\sqrt{f_{cu}}$, $f_{cu}\le 40N/mm$			N/A	N/A	kN	6.11.2 TR.4
	Decompression, $M_{0,5}=0.8(KP_0/A_{(SLS/ULS)}).Z^*_{t,(SLS/ULS)}-0.8KP_0{}^*e^*$			N/A	kNm	
	Z_t for b_5, $Z^*_{t,(SLS/ULS)}$ = $I^*_{(SLS/ULS)}/x^*_{c,(SLS/ULS)}$ = $(b_5.h^3/12)/(h/2)$			N/A	$x10^3cm^3$	
	Prestress force at SLS over b_5 only, $KP_0{}^*$ = $KP_0.b_5/b_w$			N/A	kN	
	Ecc. of prestress force, e*			N/A	mm	
	Note e = $x_{c,(SLS/ULS)}$ + e_{var}(x=@col face to shear perimeter) - [$x^*_{c,(SLS/ULS)}$ =h/2];*					
$V_{c,5}$ = {$V_{co,5}$ uncracked, MIN ($V_{co,5}$, $V_{cr,5}$) cracked}		N/A	N/A	N/A	kN	6.11.2 TR.4
$V_{c,5}/b_5d_{cen,5}$				N/A	N/mm^2	cl.4.3.8.1
	Case v_5 < $V_{c,5}/b_5d_{cen,5}$			N/A	N/A	cl.3.7.7.6
	No links required.					
	Case $V_{c,5}/b_5d_{cen,5}$ < v_5 < $1.6V_{c,5}/b_5d_{cen,5}$		N/A	N/A	N/A	cl.3.7.7.5
				N/A	N/mm^2	
	$\boxed{\Sigma A_{sv}\sin\alpha \ge \frac{(v-v_c)ud}{0.95f_{yv}}}$	$f_{yv}\le 460N/mm^2$	N/A	>=	N/A mm^2	
		$d=d_{cen,5}$				
	Note $\boxed{\Sigma A_{sv}\sin\alpha \ge}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
	Case $1.6V_{c,5}/b_5d_{cen,5}$ < v_5 < $2.0V_{c,5}/b_5d$		N/A	N/A	N/A	cl.3.7.7.5
				N/A	N/mm^2	
	$\boxed{\Sigma A_{sv}\sin\alpha \ge \frac{5(0.7v-v_c)ud}{0.95f_{yv}}}$	$460N/mm^2$	N/A	>=	N/A mm^2	
		$d=d_{max,5}$				
	Note $\boxed{\Sigma A_{sv}\sin\alpha \ge}\boxed{0.4ud/0.95f_{yv}}$			N/A	N/mm^2	
	Case v_5 > $2.0V_{c,5}/b_5d_{cen,5}$		N/A	N/A		cl.3.7.7.5
Fourth shear perimeter shear utilisation		N/A		N/A		N/A

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		jXXX	46	
		Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	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, $\Delta F_d = K_B.(\lvert M_{ULS,E/E}+\lvert$	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			*cl.3.4.4.4*
	$(d - z)/0.40$, for $60 < f_{cu} \leq 75$ N/mm^2	*Note d here refers to d_{cen};*		*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 ~ $M_{ULS}/2 - 0$ 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.			*cl.6.2.4*
	Shear stress distribution factor, K_S	1.33		
	For UDLs, K_S may be taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;			
	Effective width, b_{eff} = MIN(b_w + function (span, section, structure)	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, $\nu f_{cd} \sin\theta_f \cos\theta_f$		**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 α_{cc}=1.0, γ_C=1.5			*cl.3.1.6*
43	Strength reduction factor for concrete cracked in shear, ν	0.533		
43	$\nu = 0,6\left[1 - \frac{f_{ck}}{250}\right]$			*cl.6.2.2*
Longitudinal shear stress limit to prevent crushing utilisation, $(K_S.v_{Ed})/(\nu f_{cd}\sin($		**48%**		**OK**
Longitudinal shear stress limit for no transverse reinforcement, $0.4f_{ctd}$		**0.52**	N/mm^2	*cl.6.2.4*
	Design tensile strength, f_{ctd}	**1.29**	N/mm^2	
43	$f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C$ with α_{ct}=1.0, γ_C=1.5			*cl.3.1.6*
	$f_{ctk;0,05} = 0,7 \times f_{ctm}$	1.94	N/mm^2	*T.3.1*
	$f_{ctm} = 0,30 \times f_{ck}^{(2/3)} \leq C50/60$ — $f_{ctm} = 2,12 \cdot \ln(1+(f_{cm}/10))$ > C50/60	2.77	N/mm^2	*T.3.1*
	$f_{cm} = f_{ck} + 8(MPa)$	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, $(K_S.$		**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 $1.0A_{sf}/s_f$ and $0.5A_{sf}/s_f$ + area required for slab bending; Note K_S factored onto v_{Ed} herein;			*cl.6.2.4*
	Design yield strength of reinforcement, $f_{yd} = f_y / \gamma_S$, γ_S=1.15	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 ($45° \geq \theta_f \geq 26,5°$) $1,0 \leq \cot\theta_f \leq 1,25$ for tension flanges ($45° \geq \theta_f \geq 38,6°$)			*cl.6.2.4*
	Provided transverse reinforcement per unit length, A_e	785	mm^2/m	
Required design transverse reinforcement per unit length utilisation, $(A_{sf}/s_f)/A$		**77%**		**OK**

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Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	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, $V_1 = K_S . \Delta F_d / \Delta x$	**418**	kN/m	
Change of normal force in flange half over Δx, $\Delta F_d = K_B.(\|M_{ULS,E/E}+\|$	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 $f_{cu} \leq 60$ N/mm^2 — *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*

Note d here refers to d_{cen};

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;

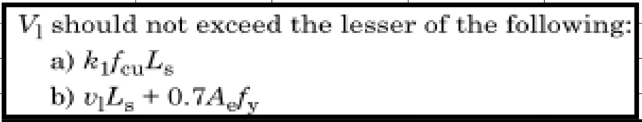
Shear stress distribution factor, K_S	1.33		

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} = MIN(b_w + 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*

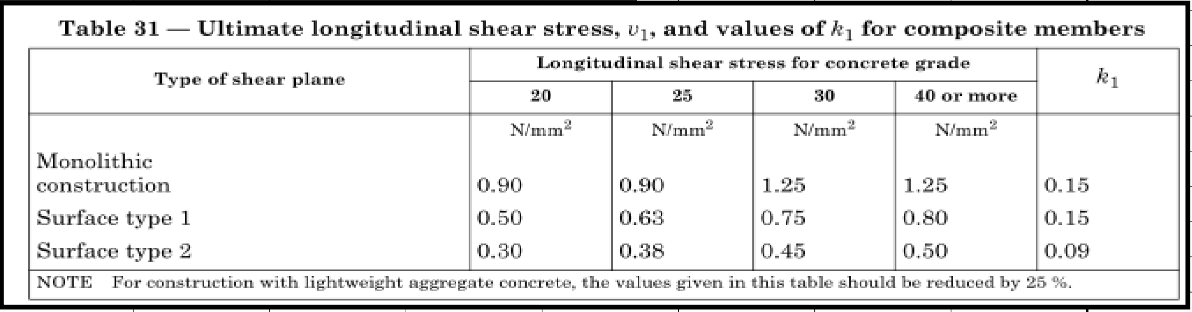


Table 31 — Ultimate longitudinal shear stress, v_1, and values of k_1 for composite members

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

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

Concrete bond constant, k_1	0.15		*T.31*
Ultimate longitudinal shear stress limit, v_1	1.25	N/mm^2	*T.31*
Surface type	Monolithic construction ▼		*T.31*
Length of shear plane, $L_s = h_f$	200	mm	
Provided transverse reinforcement per unit length, A_e	785	mm^2/m	

Note reinforcement provided for coexistent bending effects and shear reinforcement crossing the shear plane, provided to resist vertical shear, may be included provided they are fully anchored; — *cl.7.4.2.3*

Characteristic strength of reinforcement, f_y	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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		Member/Location		
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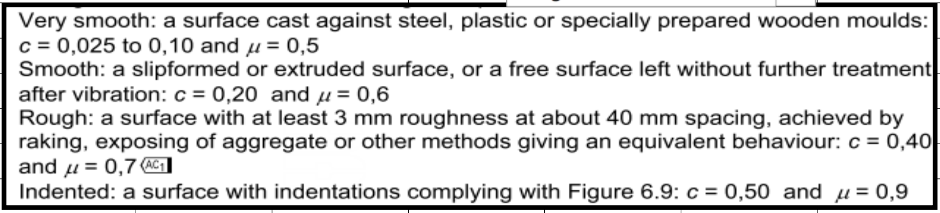
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

			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	*cl.6.2.5*
Ratio, β = 1.0	1.0		*cl.6.2.5*
Transverse shear force, V_{Ed} = ABS ($V_{ULS,E/E}$+$V_{SLS,S/E}$) / 2	651	kN	*cl.6.2.5*
Lever arm, z	0.643	m	*BC2*
Neutral axis, x: x = $(d-z)/0.45$, for $f_{cu} \le 60$ N/mm^2			*cl.3.4.4.4*
x = $(d-z)/0.40$, for $60 < f_{cu} \le 75$ N/mm^2			*cl.3.4.4.4*
x = $(d-z)/0.36$, for $75 < f_{cu} \le 105$ N/mm^2			*cl.3.4.4.4*
Note d here refers to d_{cen};			
Width of the interface, $b_i = b_w$	500	mm	*cl.6.2.5*
Longitudinal shear stress limit, v_{Rdi}	**2.28**	N/mm^2	
$v_{Rdi} = c\, f_{ctd} + \mu\, \sigma_n + \rho\, f_{yd}\, (\mu \sin \alpha + \cos \alpha) \le 0{,}5\, \nu\, f_{cd}$			*cl.6.2.5*
Note c.f_{ctd} = 0.00 if σ_n is negative (tension);			*cl.6.2.5*
Roughness coefficient, c — Rough ▼	0.400		*cl.6.2.5*
Roughness coefficient, μ — Rough ▼	0.7		*cl.6.2.5*



Very smooth: a surface cast against steel, plastic or specially prepared wooden moulds: c = 0,025 to 0,10 and μ = 0,5

Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: c = 0,20 and μ = 0,6

Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: c = 0,40 and μ = 0,7 AC1

Indented: a surface with indentations complying with Figure 6.9: c = 0,50 and μ = 0,9

Design tensile strength, f_{ctd}	**1.29**	N/mm^2	
$f_{ctd} = \alpha_{ct}\, f_{ctk,0.05} / \gamma_C$ with α_{ct}=1.0, γ_C=1.5			*cl.3.1.6*
$f_{ctk;0,05} = 0{,}7 \times f_{ctm}$	1.94	N/mm^2	*T.3.1*
$f_{ctm} = 0{,}30 \times f_{ck}^{(2/3)} \le C50/60$; $f_{ctm} = 2{,}12 \cdot \ln(1+(f_{cm}/10))$ > C50/60	2.77	N/mm^2	*T.3.1*
$f_{cm} = f_{ck} + 8(MPa)$	36	N/mm^2	*T.3.1*
Characteristic cylinder strength of concrete, f_{ck}	28	N/mm^2	*T.3.1*
Characteristic cube strength of concrete, f_{cu}	35	N/mm^2	*T.3.1*
Normal stress across longitudinal shear interface, σ_n = 0	**0.00**	N/mm^2	
Reinforcement ratio, $\rho = A_s / A_i$	**0.006**		*cl.6.2.5*
Area of reinforcement, $A_s = A_{sv,prov}/S$	3142	mm^2/m	
Note that the area of reinforcement crossing the shear interface may include ordinary shear reinforcement with adequate anchorage at both sides of the interface;			*cl.6.2.5*
Area of the joint, $A_i = 1000.b_i$	500000	mm^2/m	
Design yield strength of reinforcement, $f_{yd} = f_{yv} / \gamma_S$, γ_S=1.15	400	N/mm^2	*cl.2.4.2.4*
Angle of reinforcement, α = 90.0°	90.0	degrees	*cl.6.2.5*
Design compressive strength, f_{cd}	19	N/mm^2	
$f_{cd} = \alpha_{cc}\, f_{ck} / \gamma_C$ with α_{cc}=1.0, γ_C=1.5			*cl.3.1.6*
Strength reduction factor for concrete cracked in shear, ν	0.533		
$\nu = 0{,}6\left[1 - \frac{f_{ck}}{250}\right]$			*cl.6.2.2*
Longitudinal shear stress limit utilisation, v_{Edi}/v_{Rdi}	**89%**		**OK**

BS8110

Longitudinal Shear Within Web Rectangular or Flanged Beam (BS8110)

Longitudinal shear stress, $v_h = K_S \cdot \Delta F_c / (b_w.\Delta x)$ **2.27** N/mm^2 cl.5.4.7.2

Change of total compression force over Δx, $\Delta F_c = (|M_{ULS,E/E}+M_{SLS,S/I}$ 2367 kN cl.5.4.7.1

Lever arm, z 0.643 m

Neutral axis, x

$x =$
$(d-z)/0.45, \quad \text{for } f_{cu} \le 60\ N/mm^2$ cl.3.4.4.4
$(d-z)/0.40, \quad \text{for } 60 < f_{cu} \le 75\ N/mm^2$ cl.3.4.4.4
$(d-z)/0.36, \quad \text{for } 75 < f_{cu} \le 105\ N/mm^2$ cl.3.4.4.4

Note d here refers to d_{cen};

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

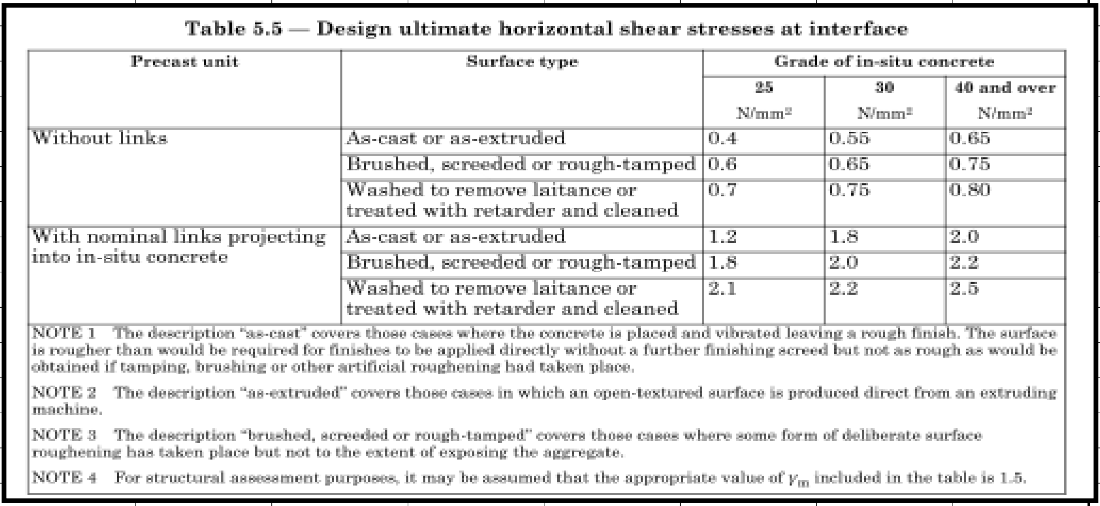


Table 5.5 — Design ultimate horizontal shear stresses at interface

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

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

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

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

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

Longitudinal shear stress limit for no nominal / design vertical reinforcement u **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, $0.15\%b_w/A$ **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**

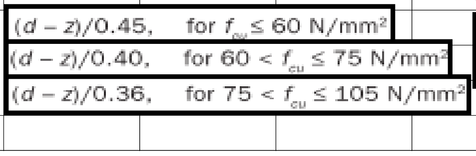
Note UT set to 0% if longitudinal shear stress limit for no design vertical reinforcement UT <= 100%;

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.		Rev.
		jXXX	50		
		Member/Location			
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.			
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.	

BS5400-4

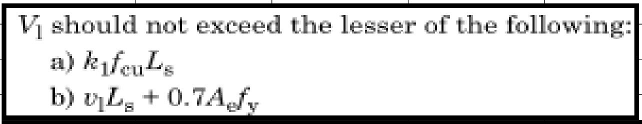
Longitudinal Shear Within Web Rectangular or Flanged Beam (BS5400-4)

Longitudinal shear force per unit length, $V_1 = K_S . \Delta F_c / \Delta x$	**1133**	kN/m	
Change of total compression force over Δx, ΔF_c = (|$M_{ULS,E/E}$+$M_{SLS,S/I}$	2367	kN	
Lever arm, z	0.643	m	
Neutral axis, x x =			*cl.3.4.4.4* *cl.3.4.4.4* *cl.3.4.4.4*


$(d - z)/0.45$, for $f_{cu} \le 60$ N/mm²
$(d - z)/0.40$, for $60 < f_{cu} \le 75$ N/mm²
$(d - z)/0.36$, for $75 < f_{cu} \le 105$ N/mm²

Note d here refers to d_{cen};

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*
Width (rectangular) or web width (flanged), b_w	500	mm	
Longitudinal shear force limit per unit length, $V_{1,limit}$	**1637**	kN/m	
(a)	2625	kN/m	*cl.7.4.2.3*
(b)	1637	kN/m	*cl.7.4.2.3*


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

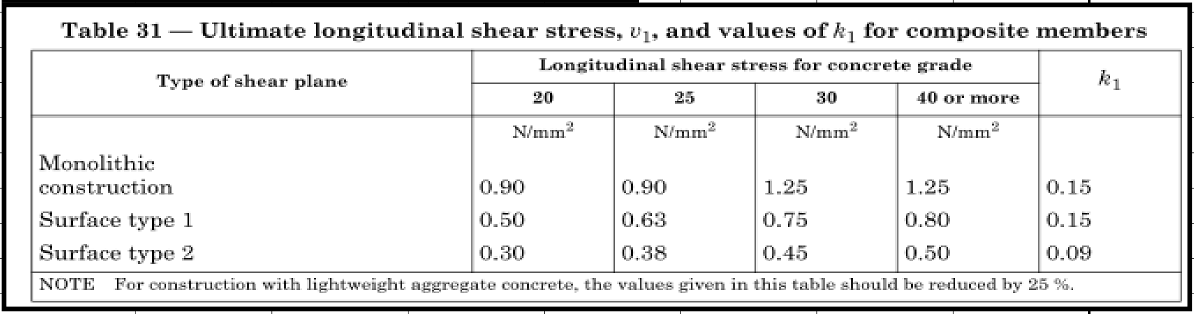


Table 31 — Ultimate longitudinal shear stress, v_1, and values of k_1 for composite members

Type of shear plane	Longitudinal shear stress for concrete grade				k_1
	20	25	30	40 or more	
	N/mm²	N/mm²	N/mm²	N/mm²	
Monolithic construction	0.90	0.90	1.25	1.25	0.15
Surface type 1	0.50	0.63	0.75	0.80	0.15
Surface type 2	0.30	0.38	0.45	0.50	0.09

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

Concrete bond constant, k_1	0.15		*T.31*
Ultimate longitudinal shear stress limit, v_1	1.25	N/mm²	*T.31*
Surface type	Monolithic construction ▼		*T.31*
Length of shear plane, $L_s = b_w$	500	mm	
Provided vertical reinforcement per unit length, A_e	3142	mm²/m	
Note $A_e = A_{sv,prov} / S$;			
Note reinforcement provided for coexistent bending effects and shear reinforcement crossing the shear plane, provided to resist vertical shear, may be included provided they are fully anchored;			*cl.7.4.2.3*
Characteristic strength of reinforcement, f_{yv}	460	N/mm²	
Longitudinal shear force limit per unit length utilisation, $V_1/V_{1,limit}$	**69%**		**OK**
Required nominal vertical reinforcement per unit length, 0.15%L_s	**750**	mm²/m	*cl.7.4.2.3*
Required nominal vertical reinforcement per unit length utilisation, 0.15%L_s/A	**24%**		**OK**

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

BS8110

Step-By-Step Design Procedure

1	*Insert the **Material Properties** and the **Prestress Characteristics and Criteria**.*	Material and Section
2	*Insert the **Section Type Considerations at TLS and (SLS/ULS)**, **Section Dimensions** with **Design Section Hogging Moment** and **Section Properties**.*	
3	*Insert the **TLS, SLS and ULS Load Combination Factors**, **External Loading** and estimate the **Action Effects From Structural Analysis (External Effects)**.*	External Load
4	*Ascertain e_{HOG} and e_{SAG} by choosing a **Physical Tendon Profile** within the limits of the section dimensions.*	Prestress Tendon
5	*Initially exclude **Prestress Force Restraint**.*	
6	*Choose a **Prestress Force at SLS (for Given Eccentricity)**, KP_0 to attain the prestress force required for load balancing at SLS by choosing the **Prestress Reinforcement** and **Prestress Force at TLS** ensuring that the percentage of tensile capacity is say 75% whilst assuming **Prestress Force Losses** of say 10% short term and 20% long term, respectively.*	
7	*Check that the **TLS and SLS Average Precompression** limits are satisfied.*	
8	*Estimate the **Action Effects From Structural Analysis (Equivalent Load, Primary and Secondary Effects)** and **Action Effects From Structural Analysis (External and Equivalent Load Effects)**.*	
9	*Check that the **Allowable Range of Prestress Force at Transfer (for Given Eccentricity) at Design Section** is satisfied.*	TLS / SLS, Detailing and Defl'n Checks
10	*Check that the chosen prestress force at transfer (w. restraint, w.o. ST losses), P_0 is less than the **Maximum Economic Upper Limit to Prestress Force at Transfer at Design Section, $P_{0,ecomax}$**.*	
11	*Check that the **TLS and SLS Top and Bottom Stresses at Design Section** limits are satisfied.*	
12	*Check that the **Magnel Diagram at Design Section** limits for prestress force at transfer, P_0 and the physical eccentricities of prestress tendon(s), e_{HOG} or e_{SAG} are satisfied.*	
13	*Check that the **Allowable Tendon Profile (for Given Prestress Force at Transfer) at All Sections** is satisfied.*	
14	*Check that the **End Block Design** is satisfied.*	
15	*Check that the **Detailing Requirements** are satisfied.*	
16	*Check that the **Deflection Criteria** are satisfied.*	
17	*Check that the **Bending at Design Section** is satisfied.*	ULS Checks
18	*Check that the **Bending at All Sections** are satisfied.*	
19	*Check that the **Shear at Critical and (Shear) Design Section** are satisfied.*	
20	*Check that the **Shear at All Sections** are satisfied.*	
21	*Check that the **Punching Shear at Column Support** are satisfied.*	
22	*Check that the **Longitudinal Shear Between Web and Flange** are satisfied.*	
23	*Check that the **Longitudinal Shear Within Web** is satisfied.*	
24	*Repeat all steps to calculate UTs including **Prestress Force Restraint.***	
25	*Repeat all steps to calculate UTs with **Design Section Sagging Moment.***	

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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*

(c) *prestress force, KP_0 increases*

(d) *prestress tendon(s) eccentricity, e increases*

ULS Moment Capacity increases as: –

$$\boxed{M_u = f_{pb}A_{ps}(d - d_n)}$$

(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, $[f_{pu}A_{ps}]/[f_{cu}bd]$ decreases (T.4.4 BS8110-1)*

$$\boxed{\frac{f_{pe}}{f_{pu}} = \frac{KP_0 / (N_T.N_s.A_s + A_{s,prov}.f_y / f_{pk})}{f_{pk}} \le 0.60}$$

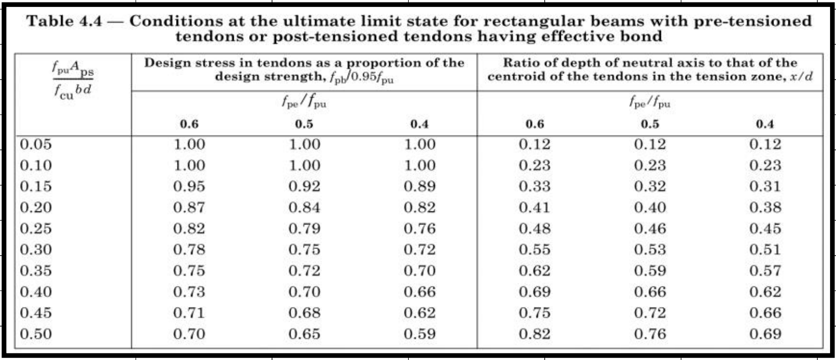

Table 4.4 — Conditions at the ultimate limit state for rectangular beams with pre-tensioned tendons or post-tensioned tendons having effective bond

$\frac{f_{pu}A_{ps}}{f_{cu}bd}$	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

(b) *prestress tendon(s) area, A_{ps} increases*

(c) *section eff. depth, d_{ps} increases*

ULS Shear Capacity increases as: –

$$\boxed{V_{co} = 0.67b_v h\sqrt{(f_t^2 + 0.8f_{cp}f_t)}} \qquad \boxed{V_{cr} = (1 - 0.55\frac{f_{pe}}{f_{pu}})\ v_c b_v d + M_o\frac{V}{M}}$$

(a) *section width, b_w in $b_v = b_w - (2/3\ BD,\ 1\ un\text{-}BD).N_T.D_T$ 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*

$$\boxed{\frac{f_{pe}}{f_{pu}} = \frac{KP_0 / (N_T.N_s.A_s + A_{s,prov}.f_y / f_{pk})}{f_{pk}} \le 0.60}$$

(f) *% of tensile area, $\rho_w = 100(N_T.N_s.A_s + A_{s,prov})/b_w d_d$ 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 $M_0 = 0.8f_{pt}Z_{b/t,(SLS/ULS)}$ increase*

$$\boxed{f_{pt} = \frac{KP_0}{A_{(SLS/ULS)}} \pm \frac{KP_0 e_{var}(x = x_d)}{Z_{b/t,(SLS/ULS)}}}$$

(h) *ratio of applied shear force to bending moment, V/M increases*

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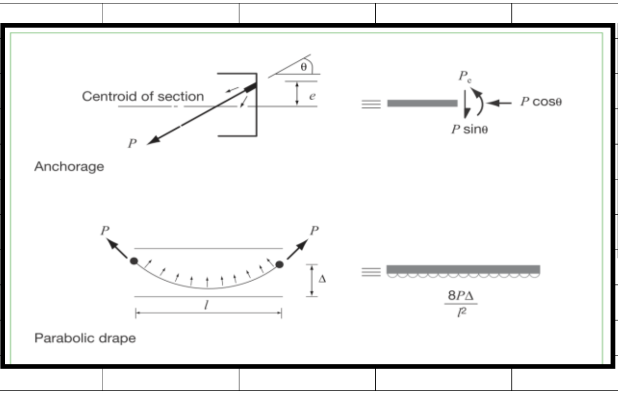
Concepts in Prestressed Concrete

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.4*
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* *Prestressin* *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.*	*IStructE* *Bourne* *Prestressin* *Feb-13*
	Secondary effects also include constant axial and shear forces throughout the span.	*cl.6.9 TR.4*
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.4*



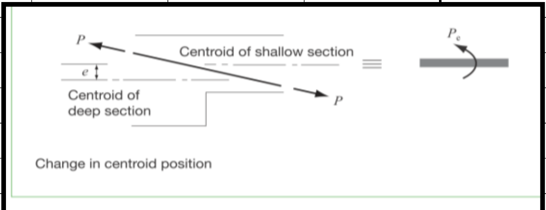
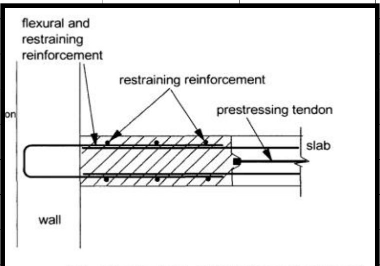


Fig 6.3 Equivalent loads

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*



b) closure strip at junction of wall and slab

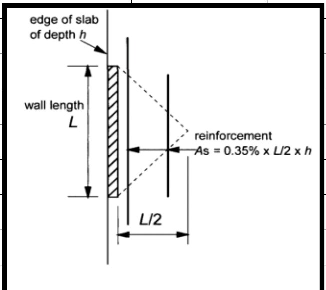


Figure 57: Distribution reinforcement close to restraining wall.

6	*Long-span insitu beams on bearings need to be designed to cater for the transfer 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*

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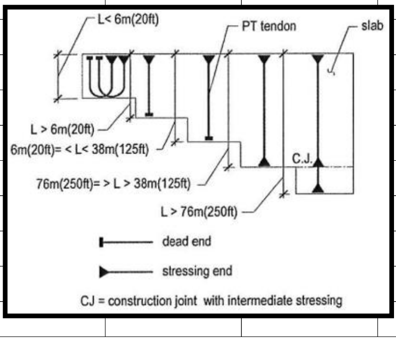
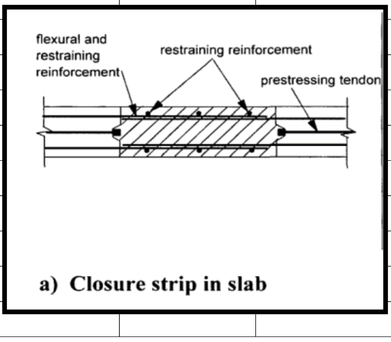
8 ***Beams curved on plan*** *are susceptible to torsion from prestressing as the tendon in the beam will apply an eccentric radial force about the beam's centroid, giving rise to torsional moments.* IEM Mar-15

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

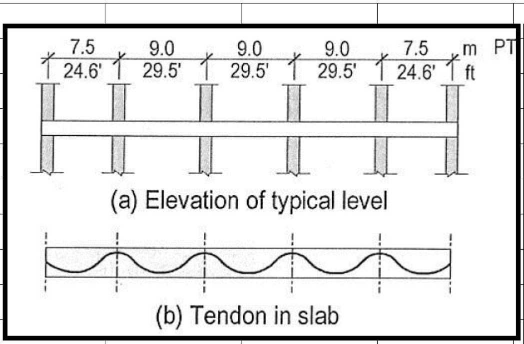
10 *The* ***extent of pours*** *is usually dictated by the limit to the length of tendons. With bonded tendons, friction losses usually restrict the length of single end stressed tendons to 25m, and double end stressed to 50m. The lower friction values for unbonded tendons extend these values to 35m 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

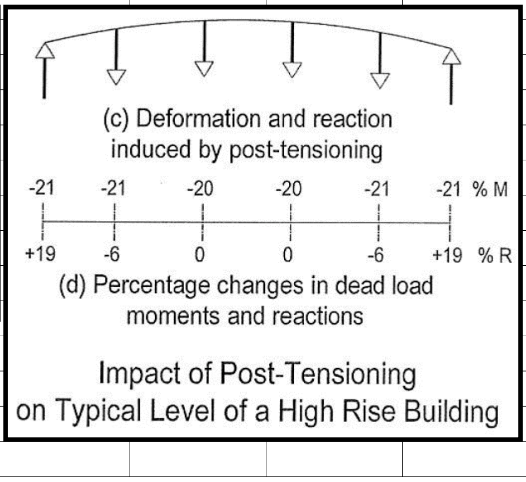




a) Closure strip in slab

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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Concepts in Prestressed Concrete in Flat Slabs

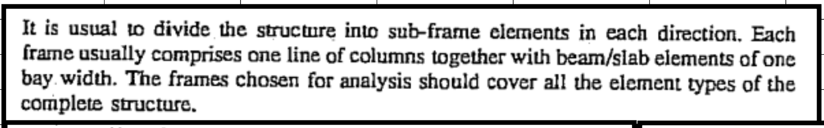
1 ***Flat slab criteria*** *include: -* *cl.2.4.1 TR.4*

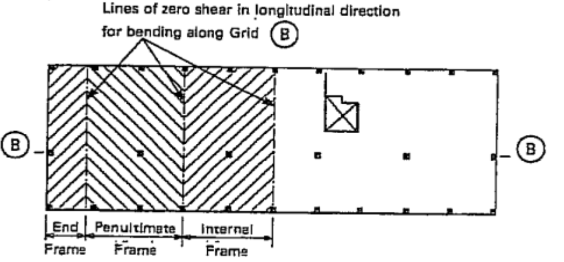
(a) precompression should be applied in two orthogonal directions

(b) aspect ratio of any panel should not be greater than 2.0

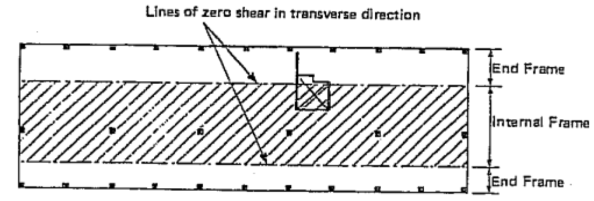
(c) the ratio of stiffness of the slab in two orthogonal directions should not exceed 10.0

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*

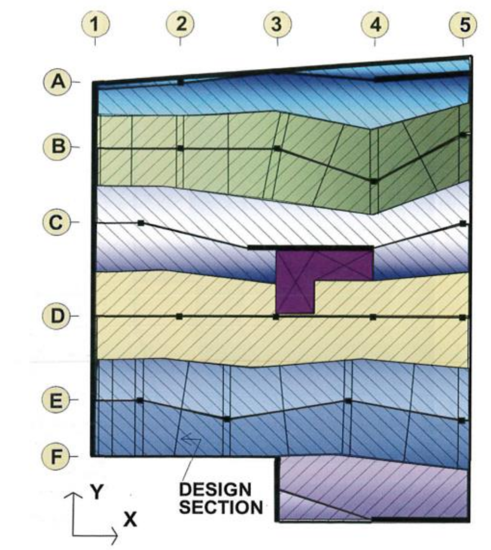

It is usual to divide the structure into sub-frame elements in each direction. Each frame usually comprises one line of columns together with beam/slab elements of one bay width. The frames chosen for analysis should cover all the element types of the complete structure.

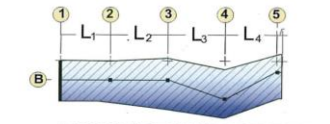


(a) Equivalent frame widths in transverse direction

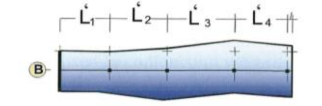


(b) Equivalent frame widths in longitudinal direction

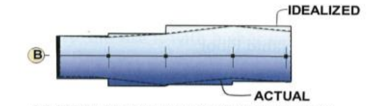




(a) DESIGN STRIP IN PROTOTYPE



(b) STRAIGHTENED DESIGN STRIP



(c) IDEALIZED TRIBUTARY FOR DESIGN

FIGURE 3.5.2E-1 Extraction and Idealization of a Design Strip (P656)

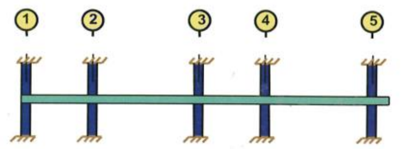


FIGURE 3.5.2E-2 Elevation of Design Strip for Analysis (P657)

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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4 Flat slab with **drop panel** dimensional requirements: -

- width of drop panel ≥ shorter span / 3 cl.3.7.1.5 BS8110, T
- depth of drop panel (excluding slab) ≥ 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 ≥ span / 5 T.1 TR.43
- width of slab band ≥ 3 x slab thickness Aalami, 201
- width of slab band ≈ 0.4 x design strip width to maximise tendon drape SELF
- depth of slab band (excluding slab) ≥ 3/4 x slab thickness SELF
- depth of slab band (excluding slab) ≤ slab thickness Aalami, 201

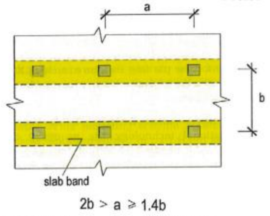
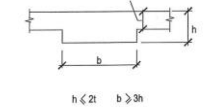


FIGURE 4.6.2-6 Parameters for Efficient Application of Slab Bands



Limiting Dimensions of a Slab-Band

FIGURE 4.6.2-5 Dimensional Parameters of a Slab Band

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 throughout its span irrespectively 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 (and Z) 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].C$_1$**

(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%, $K_{LT|ST}\approx0.8|0.9$ with E
or $(1-1/(1+\phi)/K_{LT}.K_{ST}).(1-\%creep).PT=0.33PT$, $\phi=1.5$, %creep=40%, $K_{LT|ST}\approx0.8|0.9$ with
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].C$_1$, 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, $\delta_{ES,st} = \sigma_{SLS}$	**4.6**	mm

[of the (one) storey in question]

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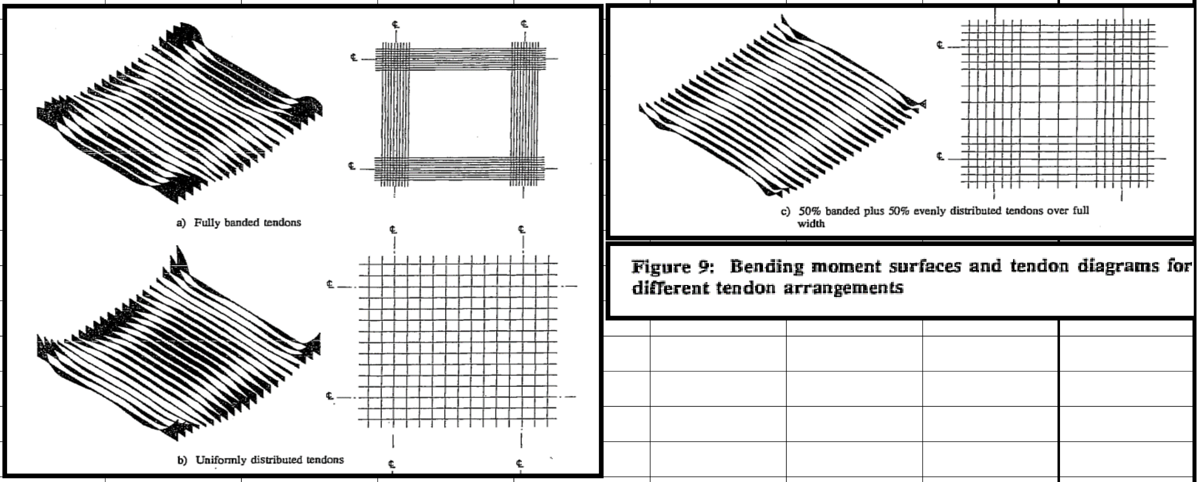
8 *Flat slab design strip* ***integration of hogging effects*** *in commercial* ***2D FE software***

.1 TR.43 *that do* ***not*** *include "rigid max" (i.e. the explicit modelling of the physical column section) should be performed just at the nodal point support (St. Venant's principle considered) and* ***not*** *at the physical column section perimeter face.*

9 *The figure below shows the bending moments derived from the grillage analysis of square panels with differing arrangement of tendons. The balanced load provided by the tendons in each direction is equal to the dead load.* *cl.2.4 TR.4*

4

4



a) Fully banded tendons

b) Uniformly distributed tendons

c) 50% banded plus 50% evenly distributed tendons over full width

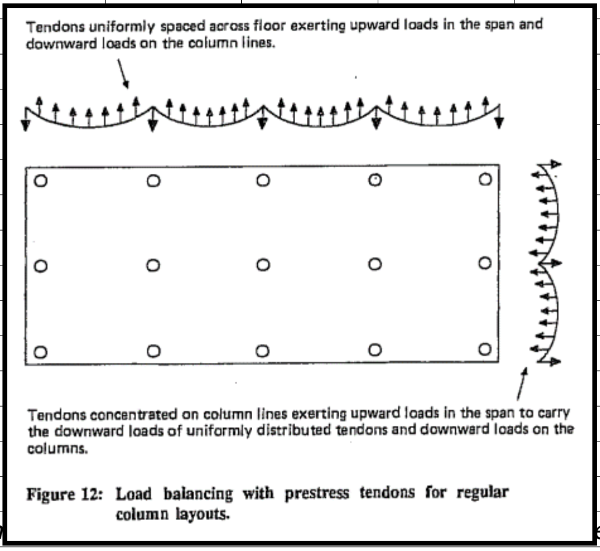
Figure 9: Bending moment surfaces and tendon diagrams for different tendon arrangements

Tests and applications have demonstrated that a post-tensioned flat slab behaves as a flat plate almost regardless of tendon arrangement. As can be seen from the figure, the ***detailed distribution (of tendons) is not critical*** *provided that sufficient tendons pass through the column zone to give adequate protection against punching shear and progressive collapse.*

However, remembering that the downward load of the uniformly distributed tendons occurs over a relatively narrow width under the reverse curvatures and that the only available exterior reaction, the column, is also relatively narrow, it becomes obvious that the ***orthogonal set of tendons should be in narrow strips*** *or bands passing over the columns.*

$=E_{lt} = E_{ck,cp}$

$=E_{lt} = E_{ck,cp}$



Tendons uniformly spaced across floor exerting upward loads in the span and downward loads on the column lines.

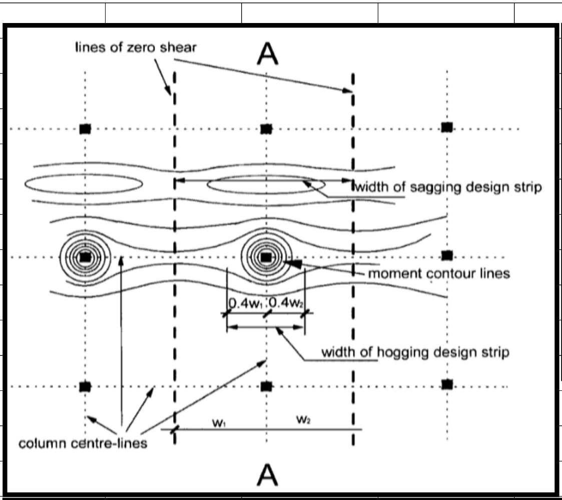
Tendons concentrated on column lines exerting upward loads in the span to carry the downward loads of uniformly distributed tendons and downward loads on the columns.

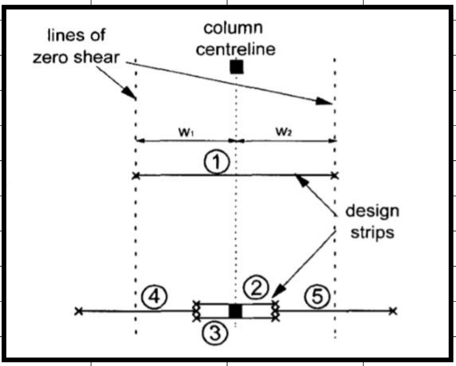
Figure 12: Load balancing with prestress tendons for regular column layouts.

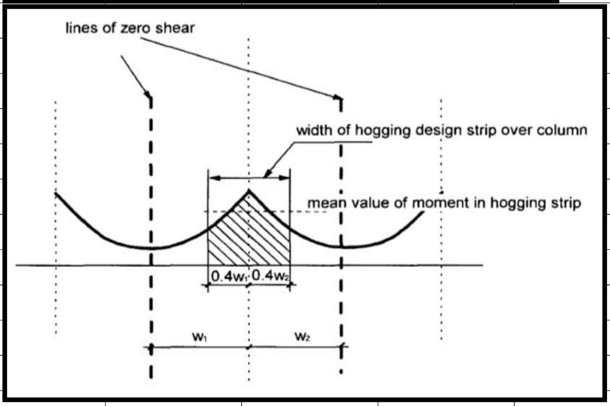
Th *es the* ***most uniform distribution*** *of moments and provides a practical layout of tendons.*

This arrangement gives ***70% of the tendons*** *in the banded zone (of* ***0.4 x panel width****) and remaining* ***30% between the bands*** *(i.e. within* ***0.6 x panel width****).*

10 *Flat slab design strip* ***integration of hogging effects*** *(to suitably model the stress concentrations over the column supports) should be made considering both the full tributary width design strip (FTW-FS-DS) and the column strip tributary width design strip (CSTW-FS-DS). To model the latter effect, a CSTW-FS-DS of* ***40%-50%*** *of the FTW-FS-DS should be checked to a* ***30%*** *higher tensile stress limit criteria. Note that obviously, this CSTW-FS-DS shall also exhibit a corresponding* ***60-50%*** *lower I (and Z) section property to resist a* ***60-80%*** *FTW-FS-DS hogging moment.* *cl.5.7.3 TR.43*







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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: -

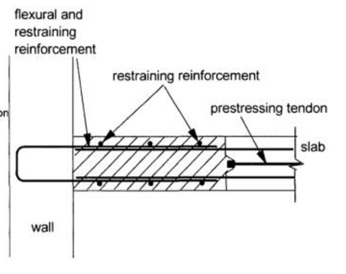
(a) (elastic, creep and shrinkage) **restraint** to axial precompression (inducing tension in the **top** and **bottom** longitudinal reinforcement), reduced with the introduction of a pour strip

(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.G+k_Q.Q+**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, $\Delta\delta_{ES,st}$ = $\Delta\sigma_{SLS}.N_s.h_s/E_{wall/col}$	**45.9**	mm

[of adjacent supports at the storey in question]

(c) **shear force** due to ULS load combination case(s) k_G.G+k_Q.Q+**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

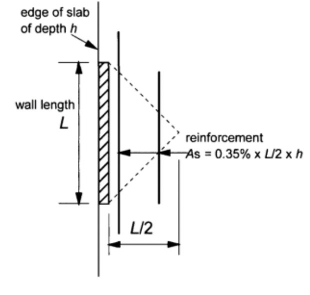


Figure 57: Distribution reinforcement close to restraining wall.

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.G+k_Q.Q+**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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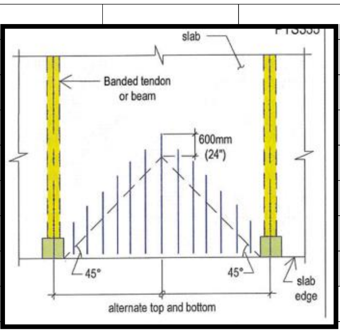
4 *The provision of longitudinal and transverse steel (untensioned reinforcement) between tendon anchorages at* ***flat slab edges*** *as follows: -* cl.6.13 TR.4

(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 x parallel reinforcement, evenly distributed between the anchorages and extending MAX(l_a,$0.7l_a$+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.* cl.6.10.6 TR.

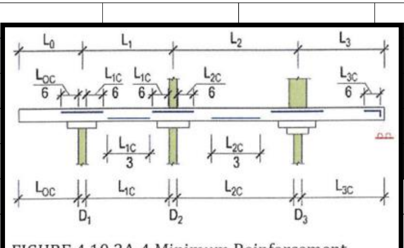
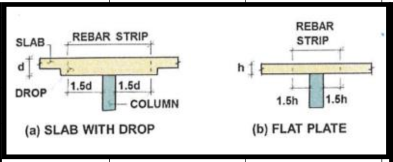


FIGURE 4.10.2A-4 Minimum Reinforcement Lengths and Layout for Common Conditions



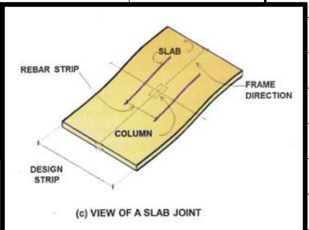


FIGURE 4.10.2A-5 Strip Identification for Placing of Minimum Bonded Reinforcement (P561)

6 ***End block*** *detailing as follows: -*

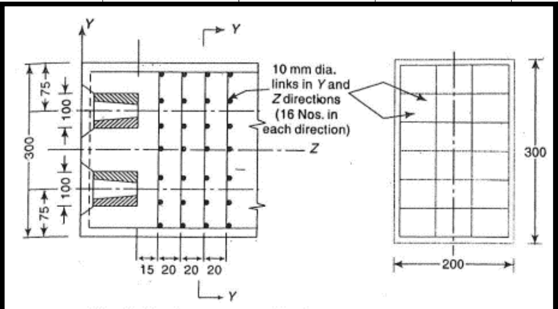


Fig. 10.19 *Arrangement of Anchorage Zone reinforcement*

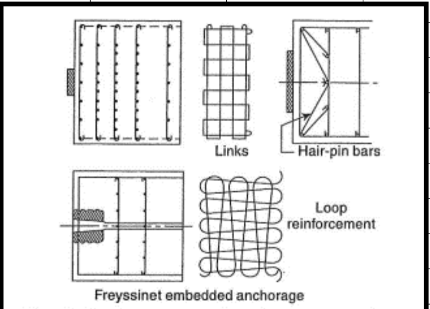


Fig. 10.15 *Arrangement of reinforcement in end blocks*

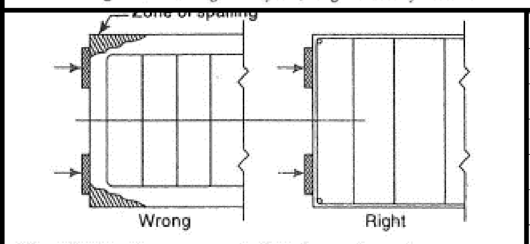


Fig. 10.18 *Arrangement of steel cage in anchorage zone*

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Creep Coefficient, ϕ *cl.7.3*

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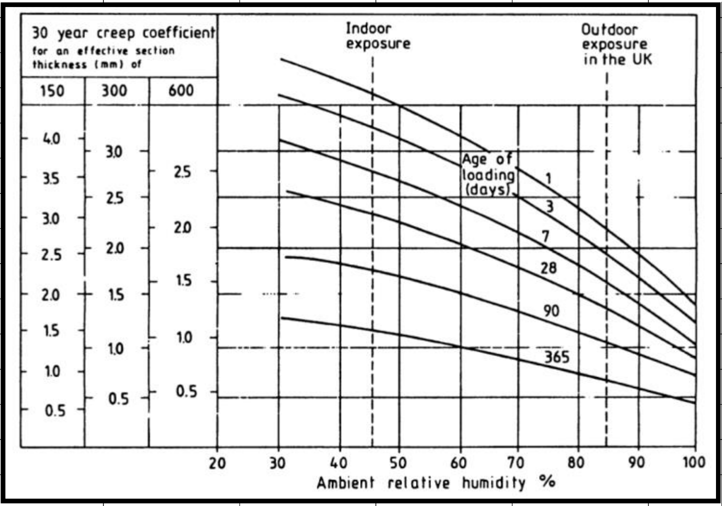

The creep coefficient may be estimated from Figure 7.1. In this Figure, the effective section thickness is defined, for uniform sections, as twice the cross-sectional area divided by the exposed perimeter. If drying is prevented by immersion in water or by sealing, the effective section thickness should 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

RH100%	*ϕ =*	*1.0*	*1/(1+ ϕ) =*	0.50
RH85%	*ϕ =*	*1.5*	*1/(1+ ϕ) =*	0.40

Creep Coefficient, ϕ *cl.24.2.4.1.*

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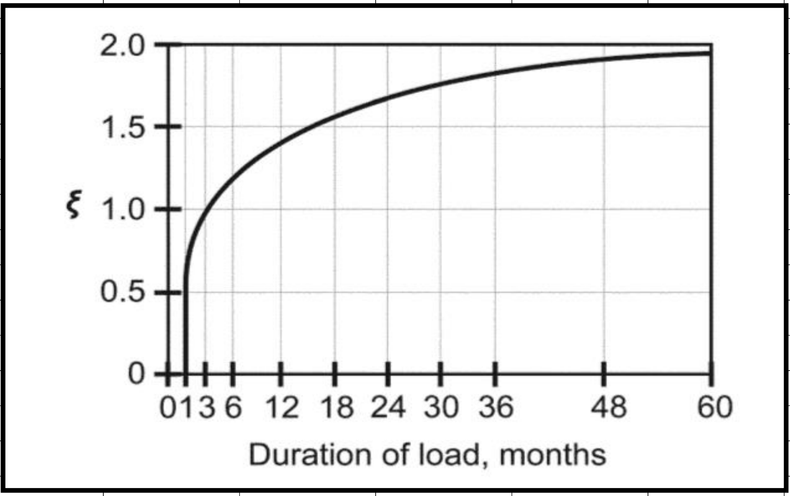


RH100%	*ϕ =*	*2.0*	*1/(1+ ϕ) =*	0.33
RH85%	*ϕ =*	*2.0*	*1/(1+ ϕ) =*	0.33

Creep Coefficient, ϕ *cl.3.1.8.3*

AS3600

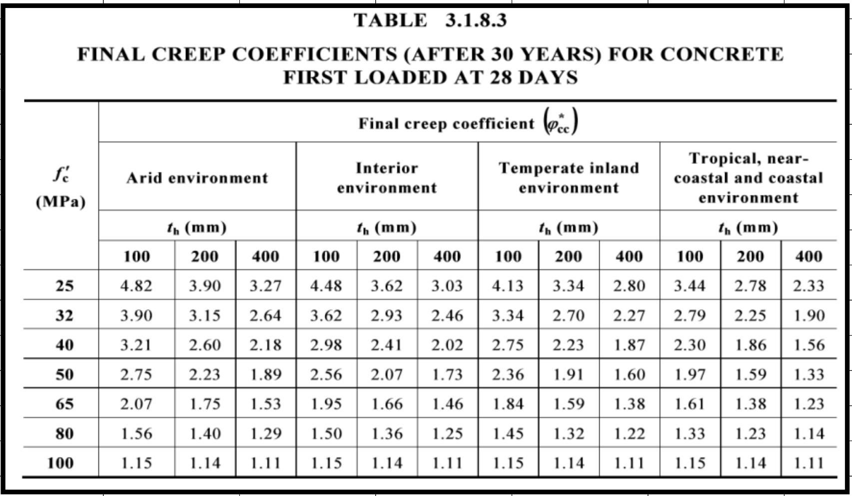


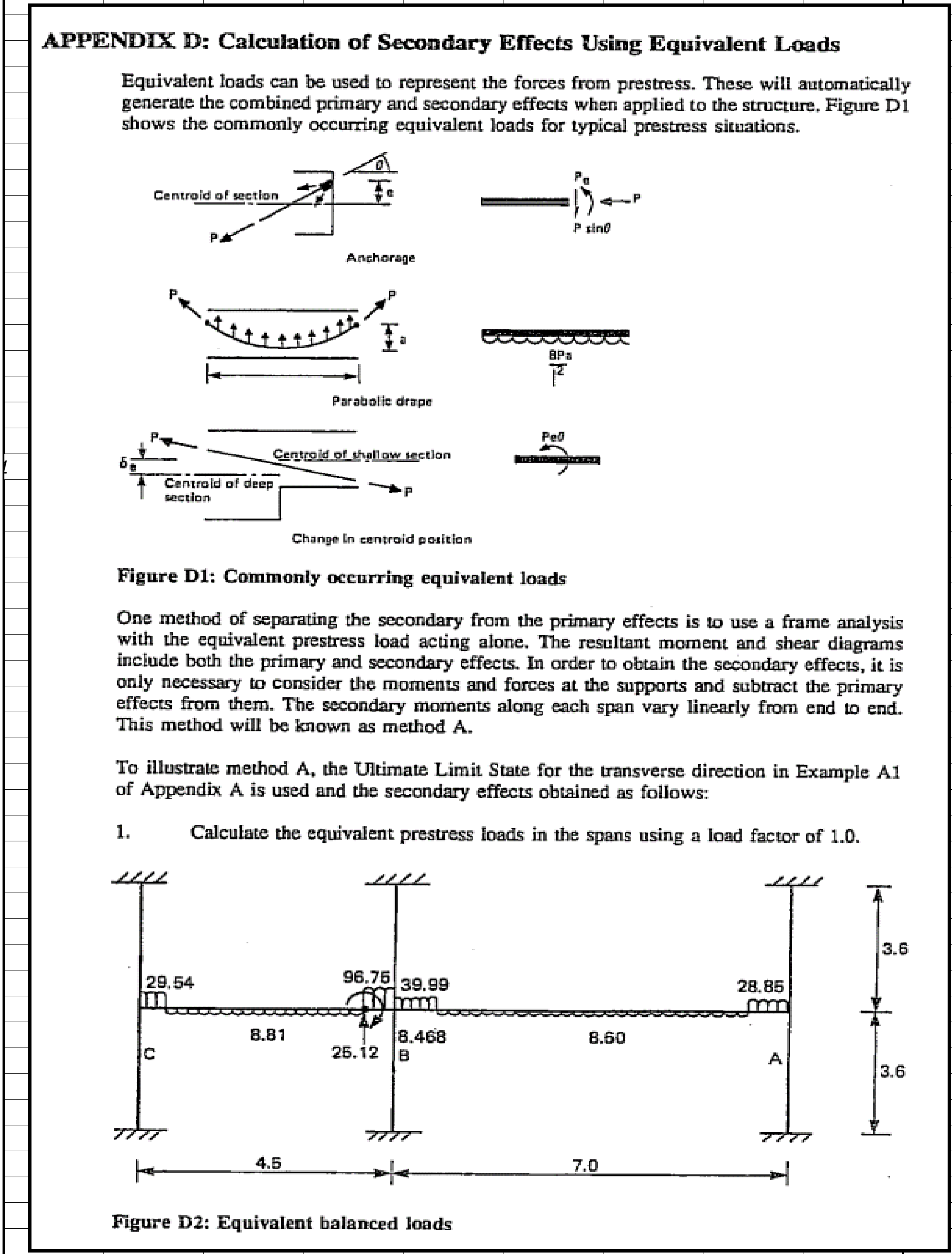
TABLE 3.1.8.3

FINAL CREEP COEFFICIENTS (AFTER 30 YEARS) FOR CONCRETE FIRST LOADED AT 28 DAYS

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

RH100%	*ϕ =*	*1.5*	*1/(1+ ϕ) =*	0.40
RH85%	*ϕ =*	*2.0*	*1/(1+ ϕ) =*	0.33

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

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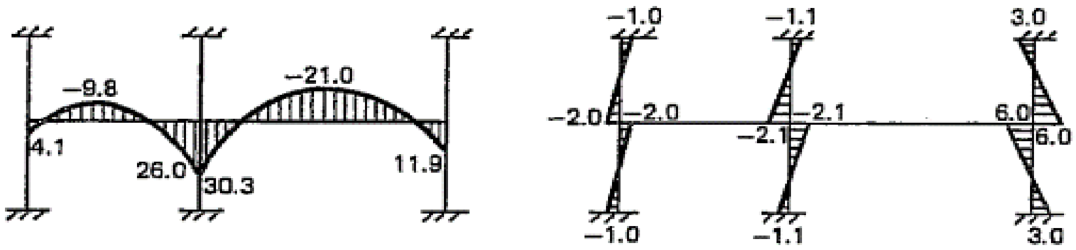
2. Analyse the structure and obtain the bending moment diagram.



a) slab b) columns

Figure D3: Moments due to primary and secondary effects

3. Calculate the primary moments due to prestress (Pe) in the slab at each support. There are no primary moments in the columns.

At support C, Pe = 0
At support B(C), Pe = -172 kNm
At support B(A), Pe = -172 kNm
At support A, Pe = 0

4. Subtract the primary moments from step (2). At this stage it should be noted that the moments and reactions in the columns from the frame analysis are due entirely to secondary effects.

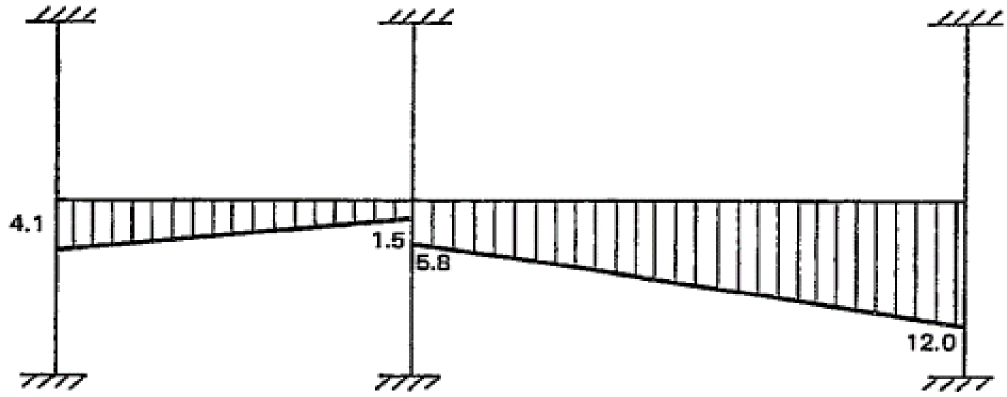


Figure D4: Bending moment diagram due to secondary effects

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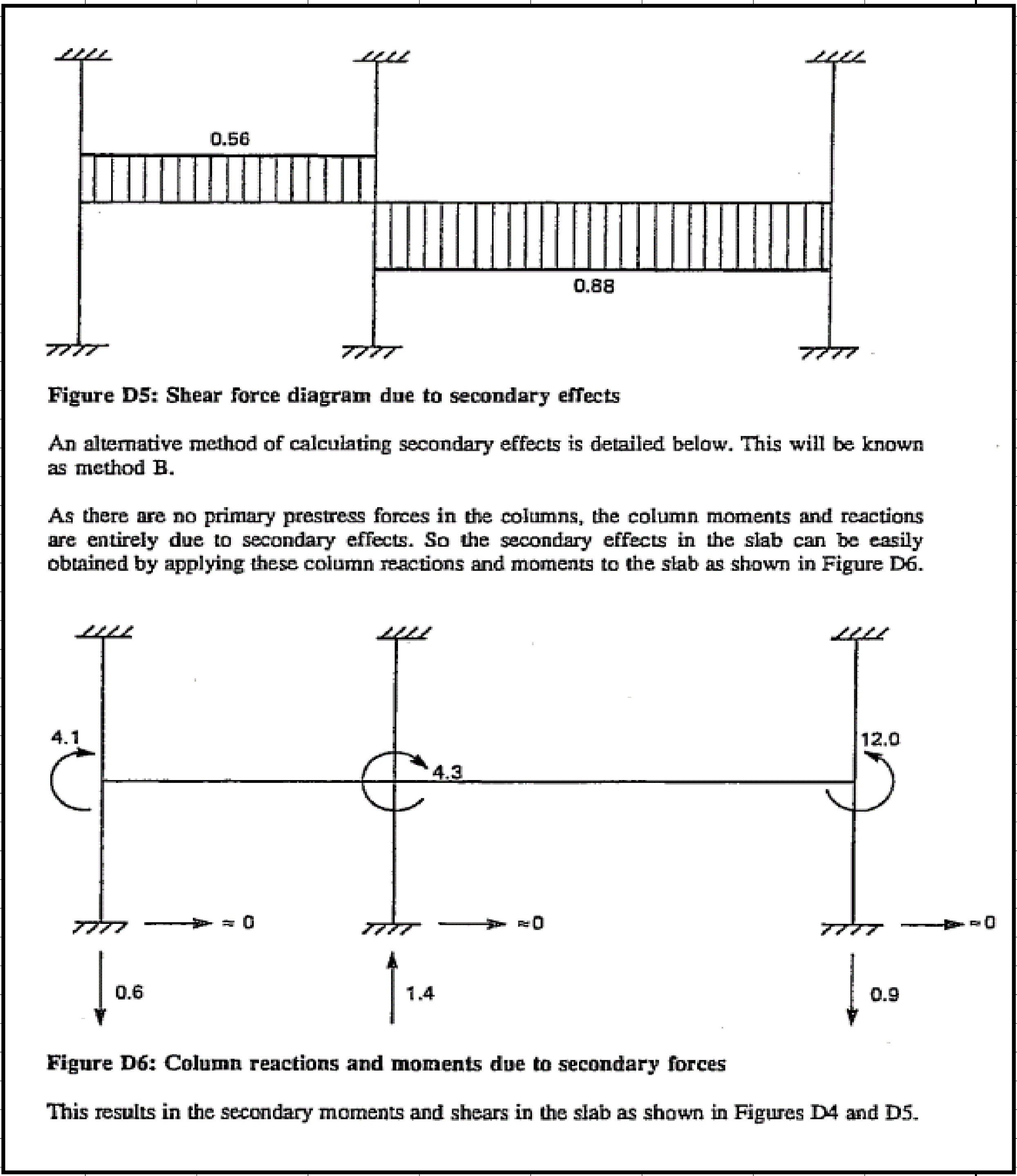

Figure D5: Shear force diagram due to secondary effects

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

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

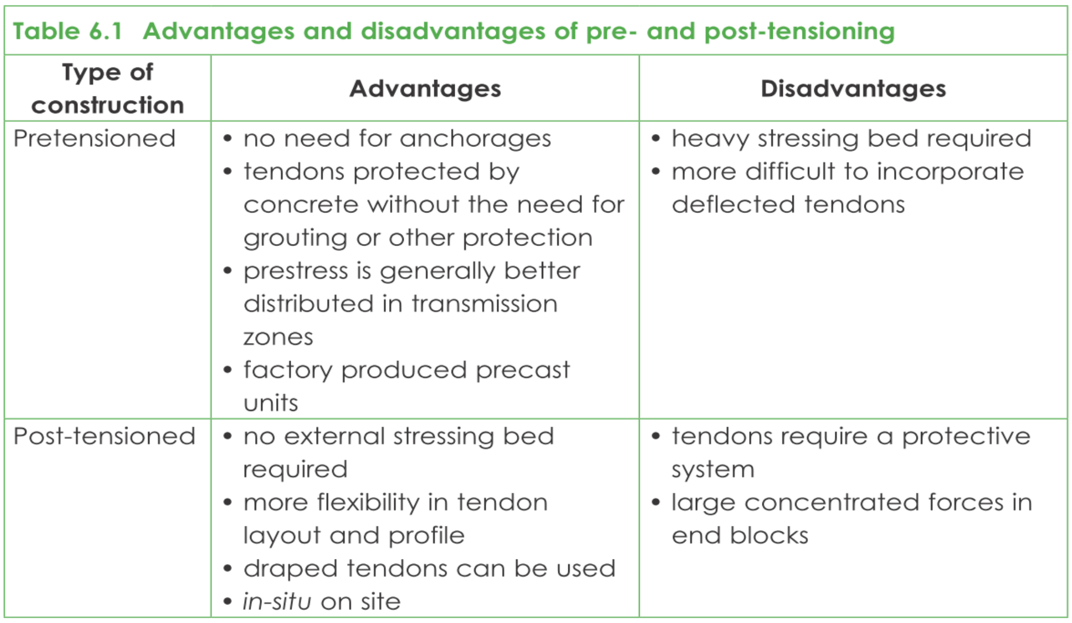
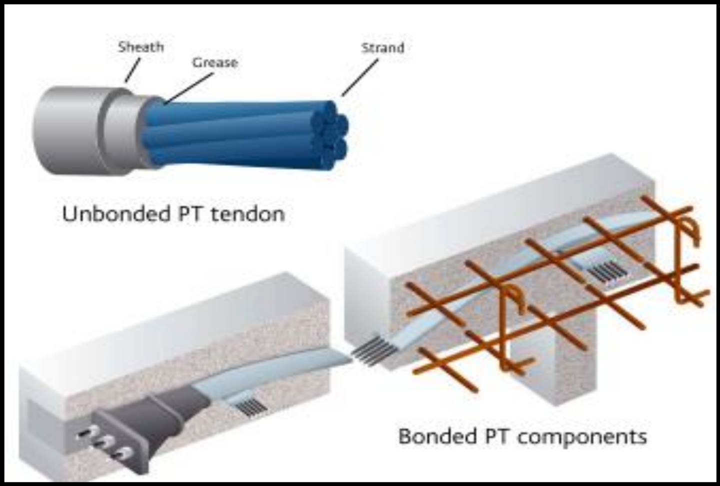
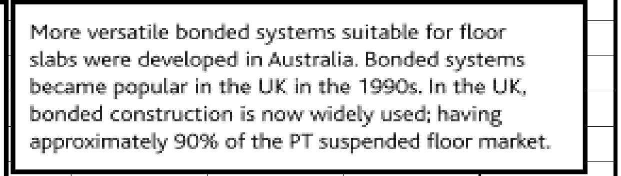


Table 6.1 Advantages and disadvantages of pre- and post-tensioning

Type of construction	Advantages	Disadvantages
Pretensioned	• 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 • *in-situ* on site	• tendons require a protective system • large concentrated forces in end blocks





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 system before pouring concrete.
Courtesy of Freyssinet.

Unbonded system before pouring concrete.
Courtesy of Balvac.

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No. jXXX	Sheet No. 66	Rev.
		Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by **XX**	Date **18/08/2025**	Chd.

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Bonded or Unbonded

Aspect	Bonded	Unbonded
Fire and Corrosion Protection	Greater	Lower
ULS Flexural Strength	Higher	Lower
Grouting, Demolition	Required, Safer	Not Required, Less Safe
Tendon Renewal, Friction Loss	Not Replaceable, Higher	Replaceable, Lower
Speed, Cost	Slower, Dearer	Faster, Cheaper

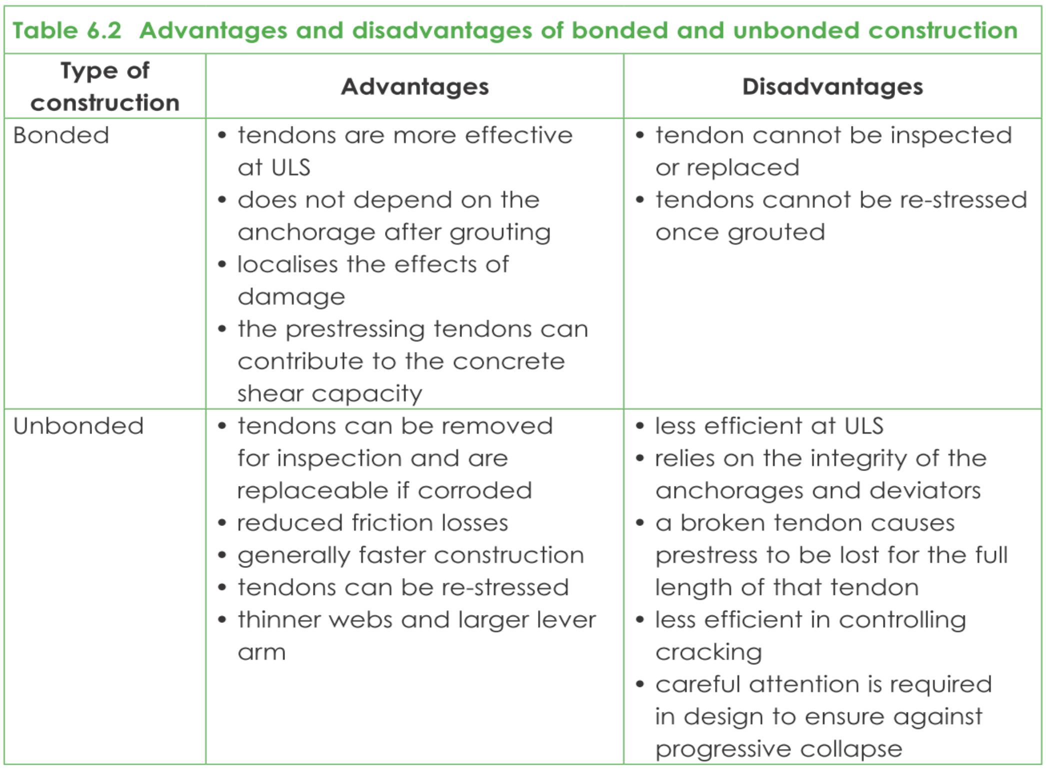

Table 6.2 Advantages and disadvantages of bonded and unbonded construction

Type of construction	Advantages	Disadvantages
Bonded	• tendons are more effective at ULS • does not depend on the anchorage after grouting • localises the effects of damage • the prestressing tendons can contribute to the concrete shear capacity	• tendon cannot be inspected or replaced • tendons cannot be re-stressed once grouted
Unbonded	• tendons can be removed for inspection and are replaceable if corroded • reduced friction losses • generally faster construction • tendons can be re-stressed • thinner webs and larger lever arm	• less efficient at ULS • relies on the integrity of the anchorages and deviators • a broken tendon causes prestress to be lost for the full length of that tendon • less efficient in controlling cracking • careful attention is required in design to ensure against progressive collapse

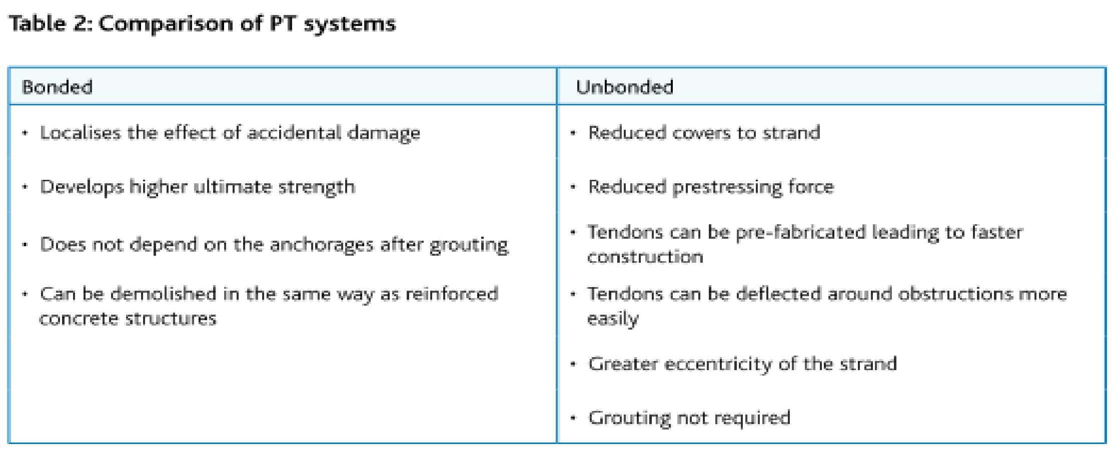

Table 2: Comparison of PT systems

Bonded	Unbonded
• Localises the effect of accidental damage • Develops higher ultimate strength • Does not depend on the anchorages after grouting • Can be demolished in the same way as reinforced concrete structures	• Reduced covers to strand • Reduced prestressing force • Tendons can be pre-fabricated leading to faster construction • Tendons can be deflected around obstructions more easily • Greater eccentricity of the strand • Grouting not required

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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.

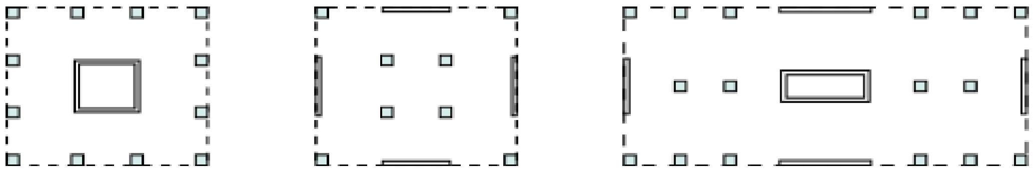
Where the restraining walls are in a favourable arrangement and the floor is in an internal environment, the maximum length of the floor without movement joints can be up to 50m. However, full consideration should be given to the effects of shrinkage due to drying, early thermal effects, elastic shortening 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:

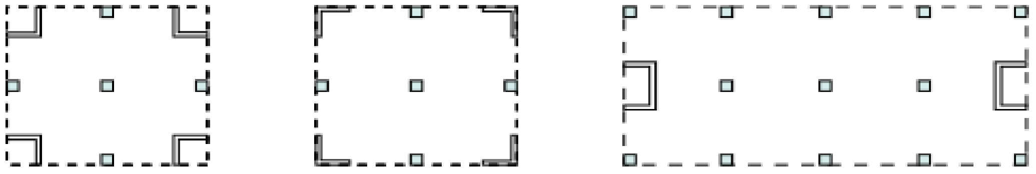
- Using infill strips which are usually cast around 28 days after the remainder of the floor, to allow initial shrinkage to occur (see Figure 10).
- Increasing the quantity of conventional reinforcement, to control the cracking.
- Using temporary release details (see Figure 11).
- Reducing the stiffness 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

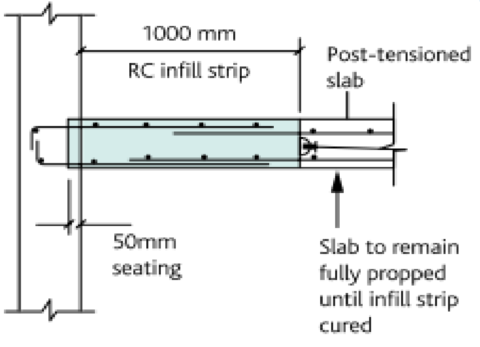
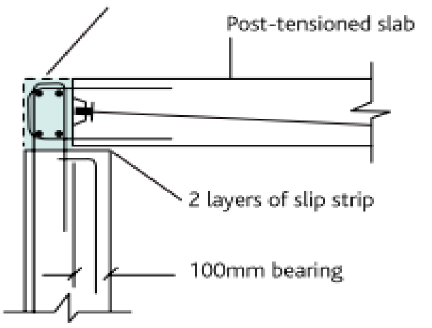


FIGURE 11: TEMPORARY RELEASE DETAIL



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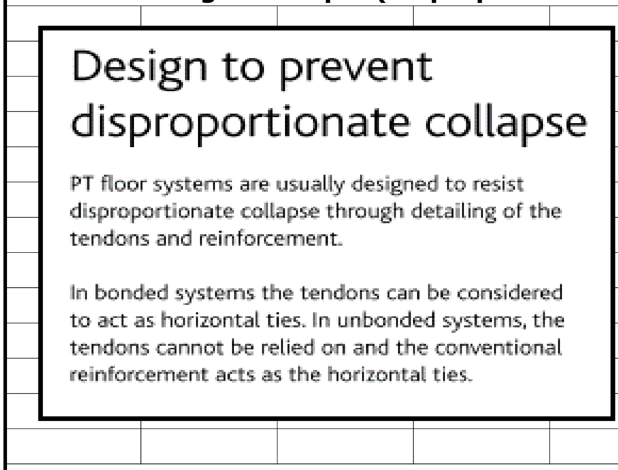
Standard Design Concepts (Disproportionate Collapse Requirements)

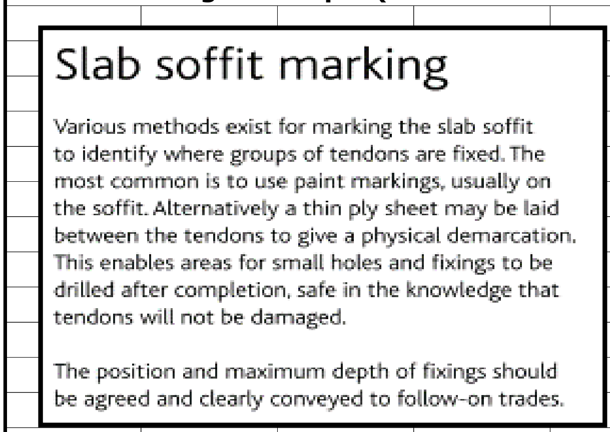


Design to prevent disproportionate collapse

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)

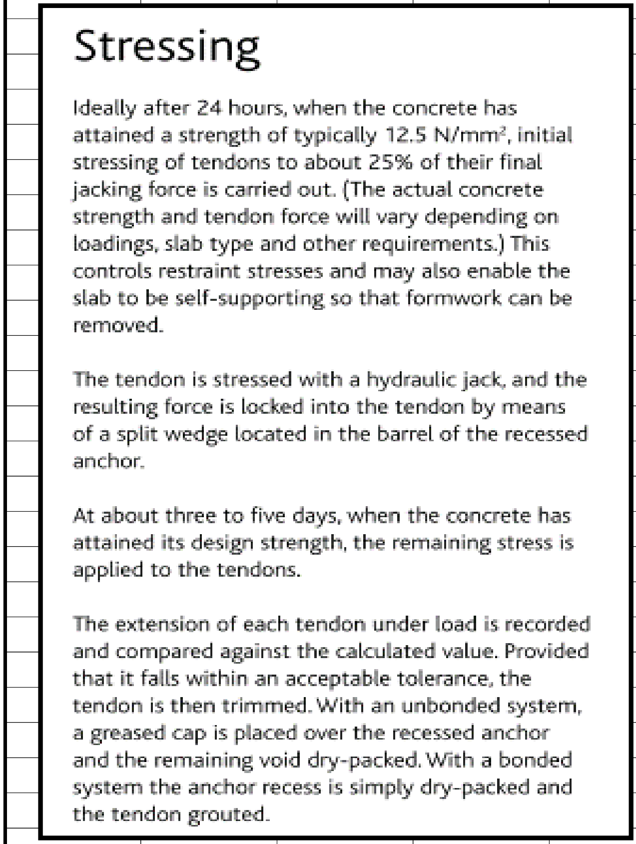


Slab soffit marking

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/mm^2, 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.

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

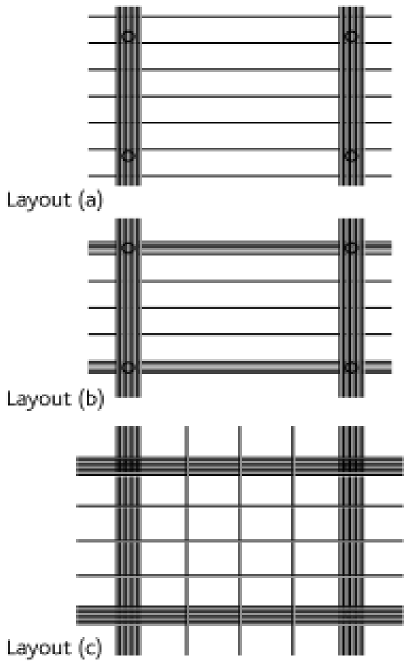


FIGURE 13: DETAILING OF TENDONS AROUND AN OPENING

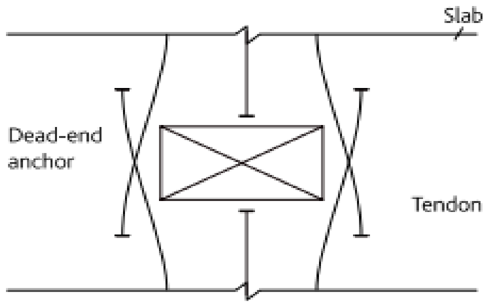
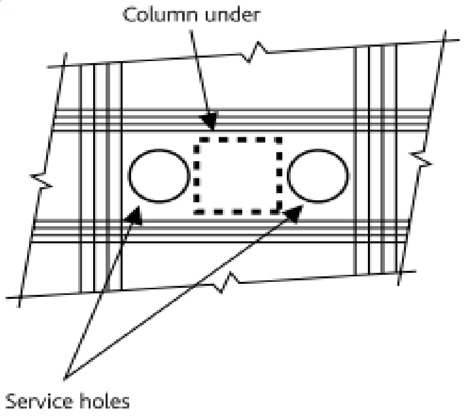
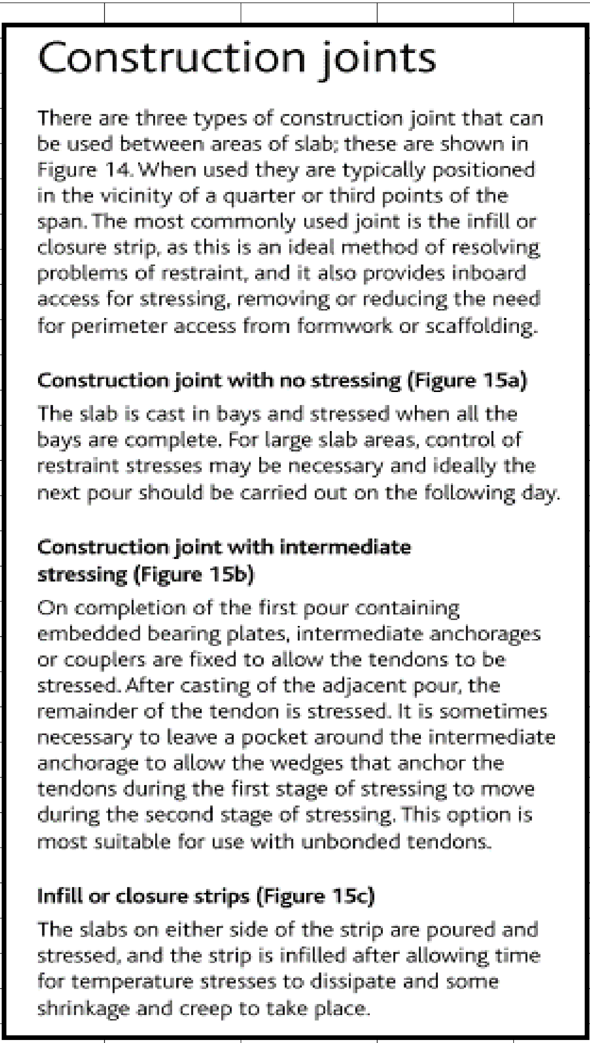


FIGURE 14: LAYOUT OF TENDONS TO ALLOW SERVICES TO BE PLACED CLOSE TO COLUMN FACE



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)

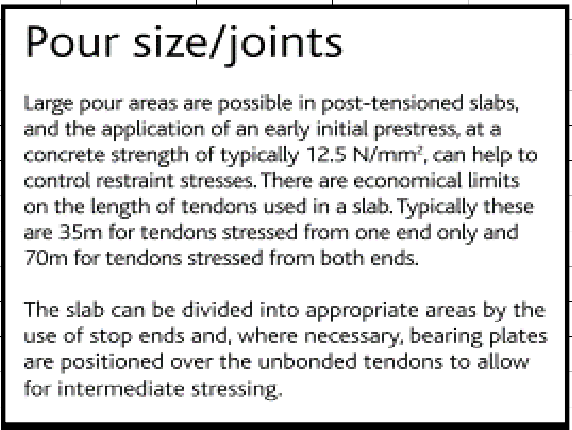
The slab is cast in bays and stressed when all the bays are complete. For large slab areas, control of restraint stresses may be necessary and ideally the next pour should be carried out on the following day.

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.

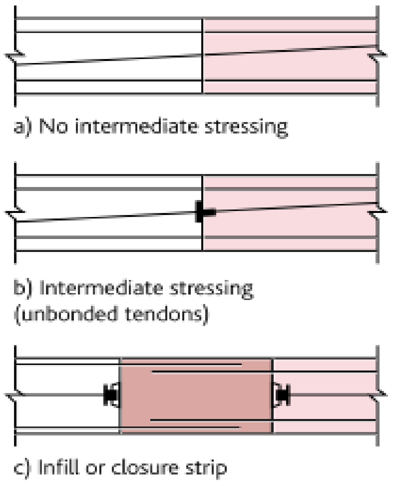


Pour size/joints

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



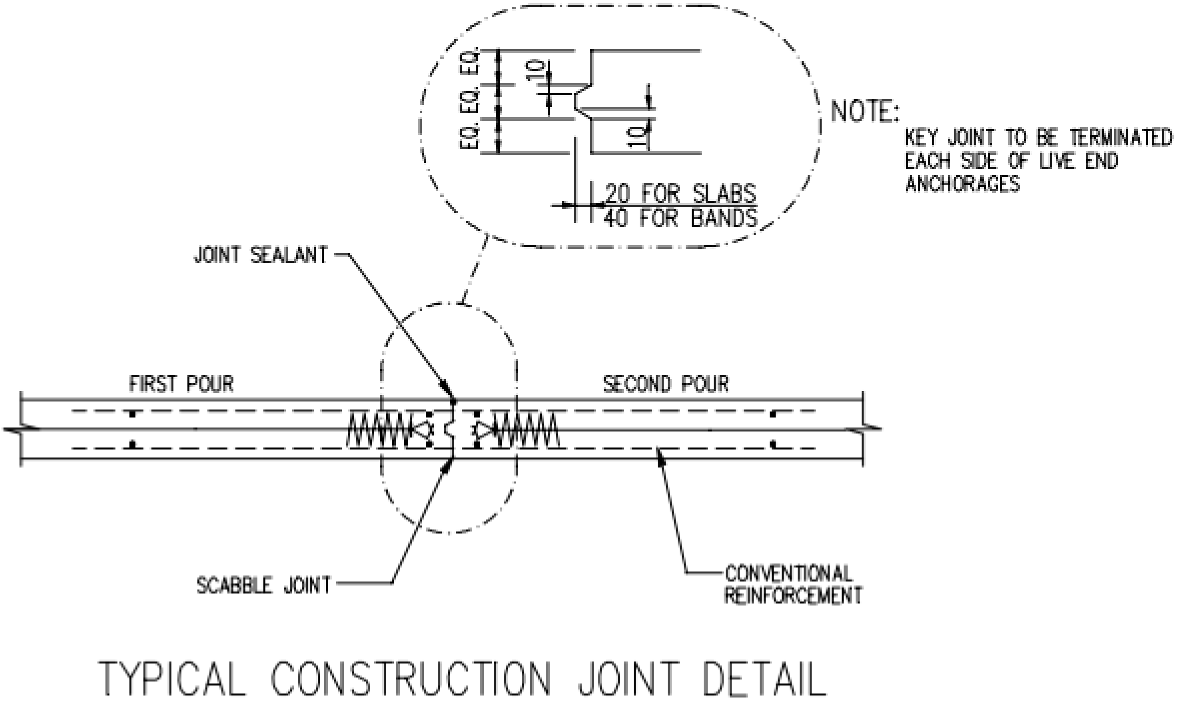


Figure 6 – 'Stitched' construction joint detail

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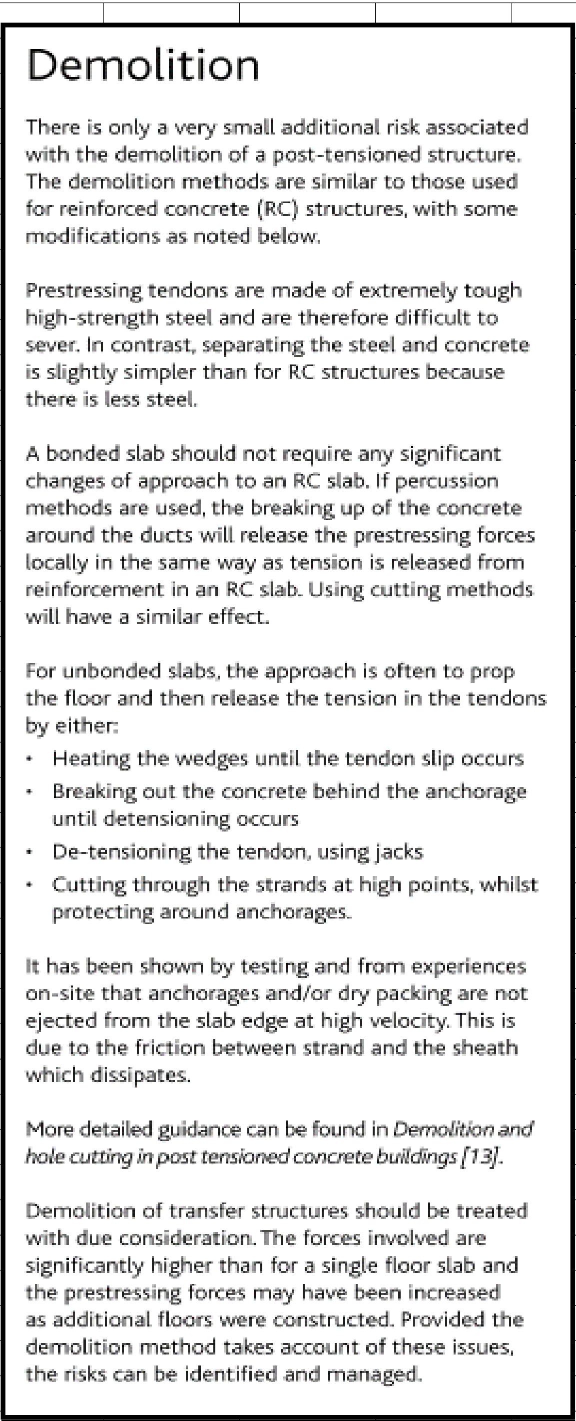
Standard Design Concepts (Demolition and Alterations)



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:

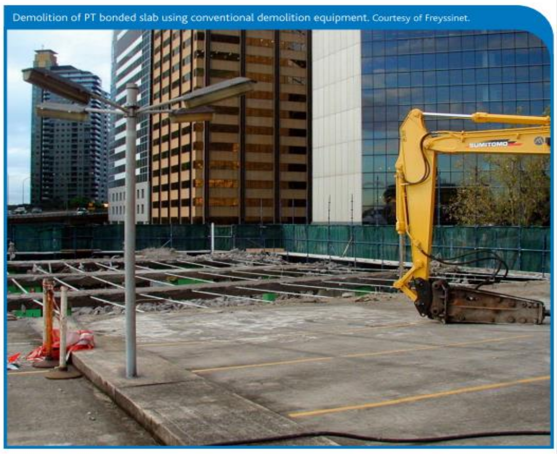
- Heating the wedges until the tendon slip occurs
- Breaking out the concrete behind the anchorage until detensioning occurs
- De-tensioning the tendon, using jacks
- Cutting through the strands at high points, whilst protecting around anchorages.

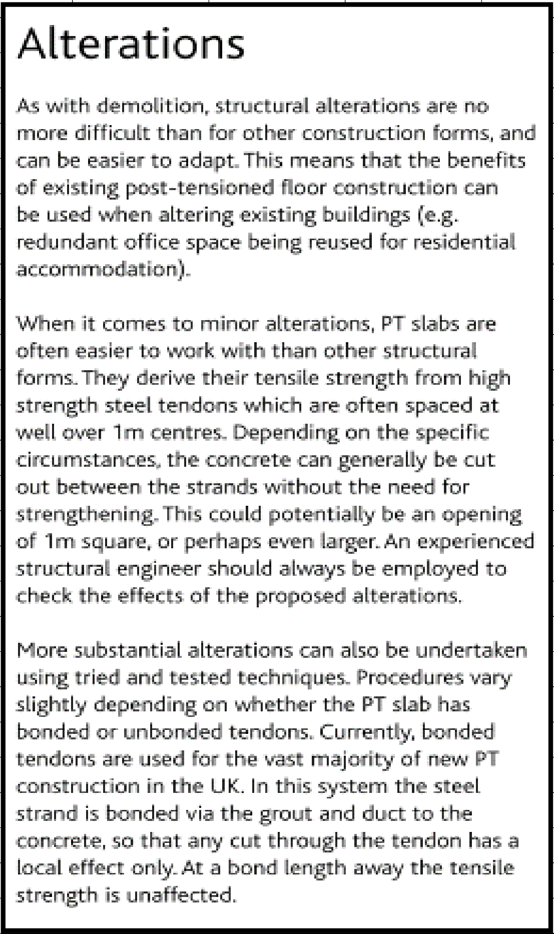
It has been shown by testing and from experiences on-site that anchorages and/or dry packing are not ejected from the slab edge at high velocity. This is due to the friction between strand and the sheath which dissipates.

More detailed guidance can be found in *Demolition and hole cutting in post tensioned concrete buildings [13]*.

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.

Demolition of PT bonded slab using conventional demolition equipment. Courtesy of Freyssinet.



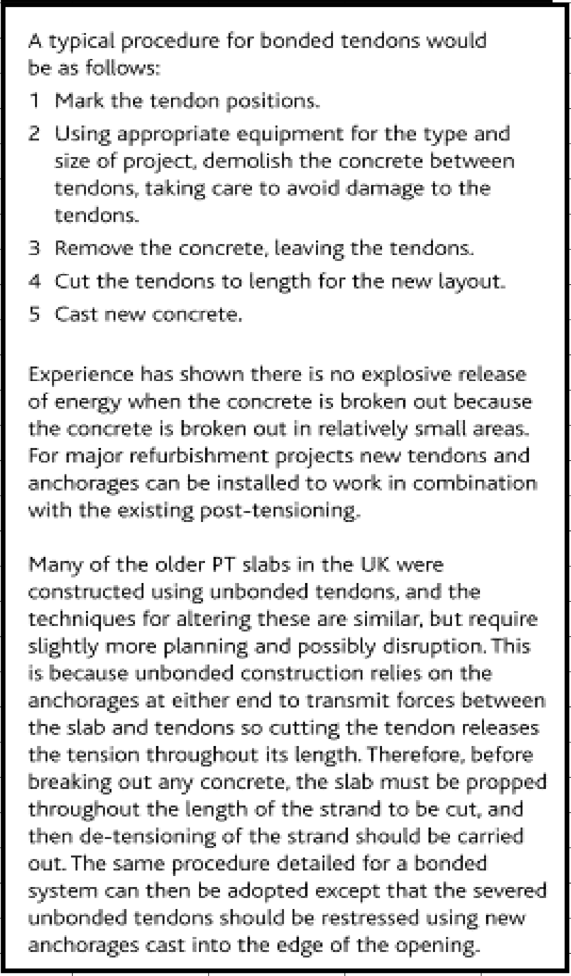


Alterations

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.

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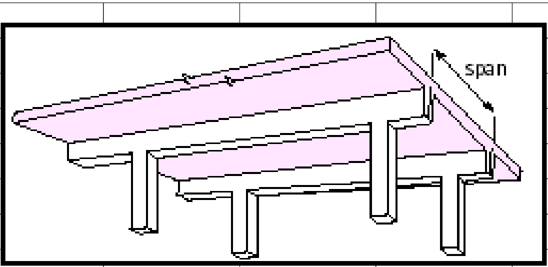
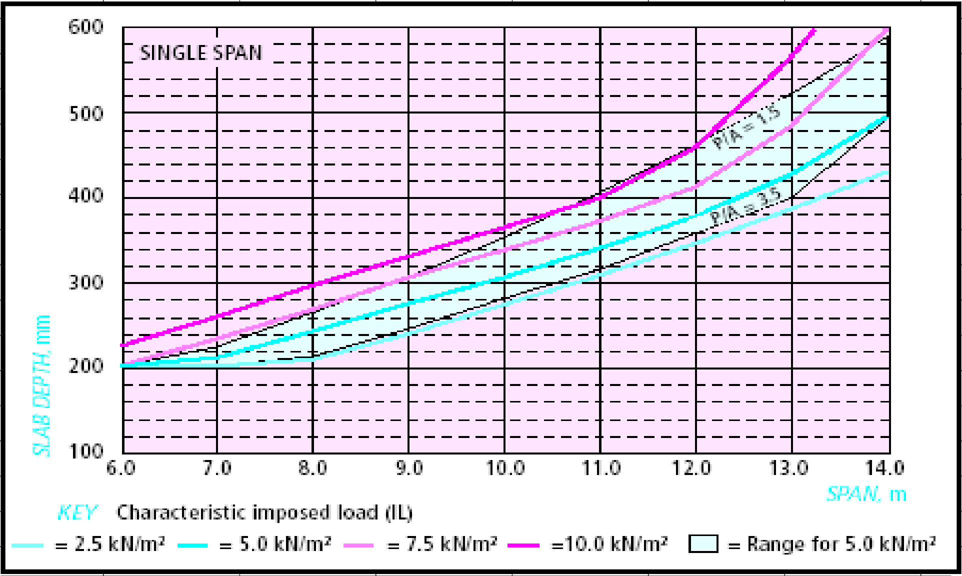
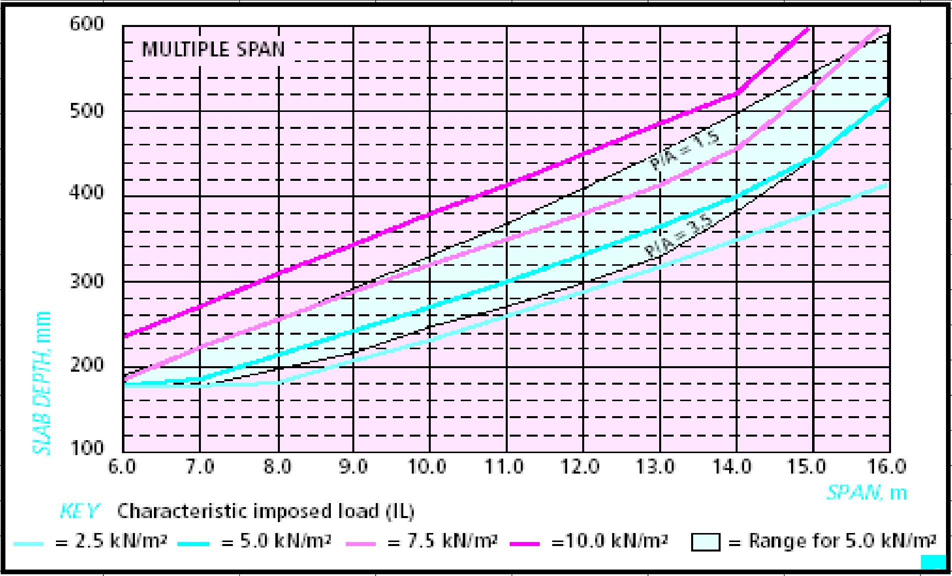
Scheme Design: PT One Way Spanning Solid Slab







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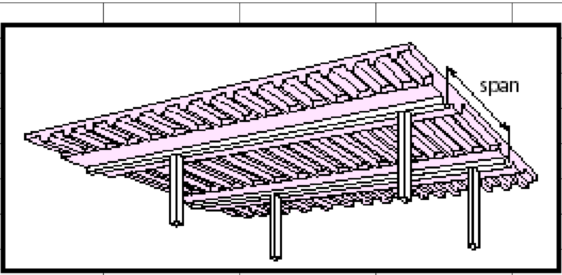
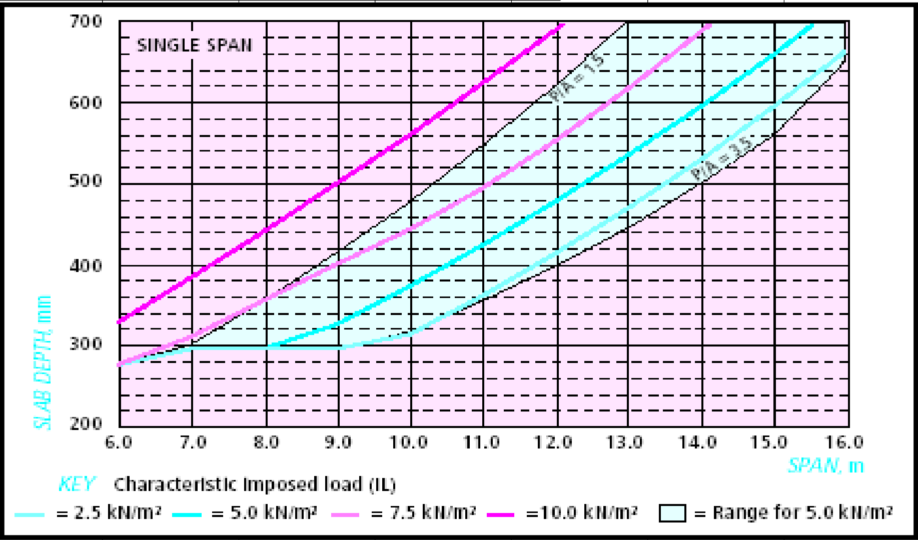
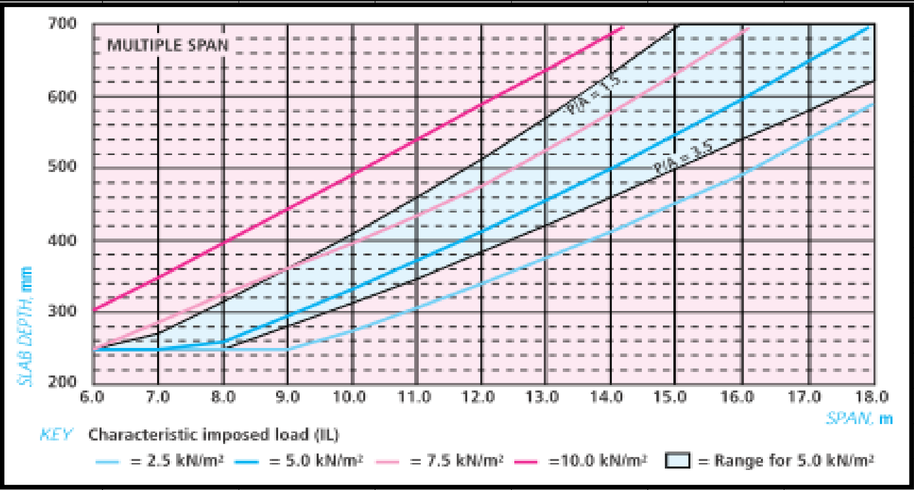
Scheme Design: PT One Way Spanning Ribbed Slab







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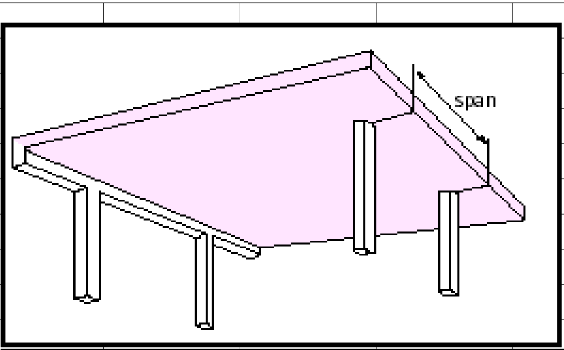
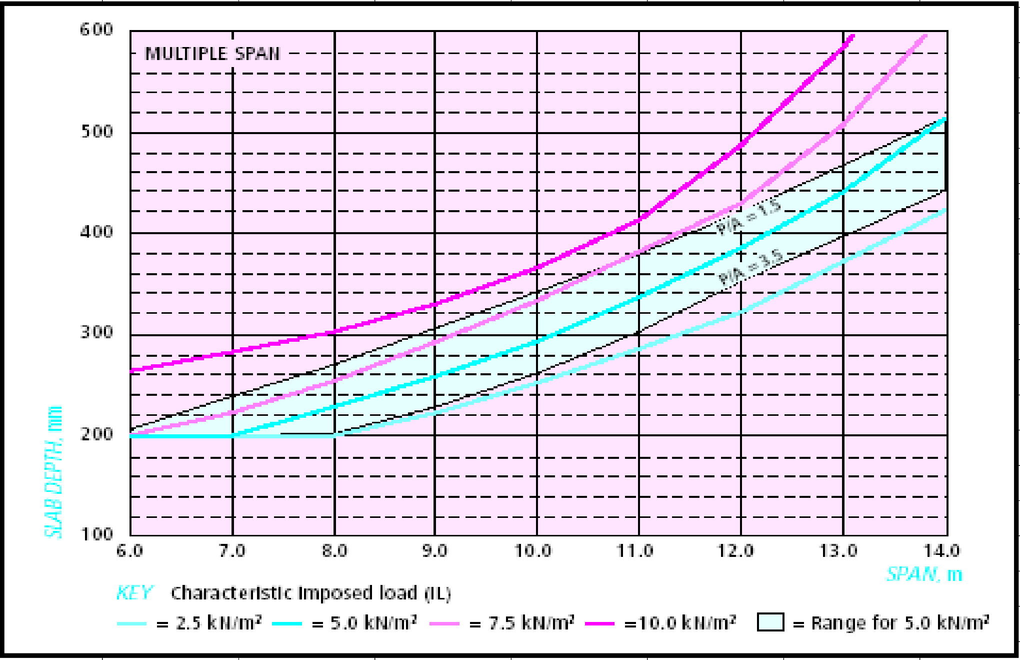
Scheme Design: PT Flat Slab With Edge Beams





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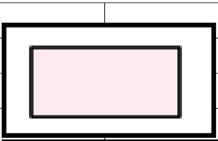
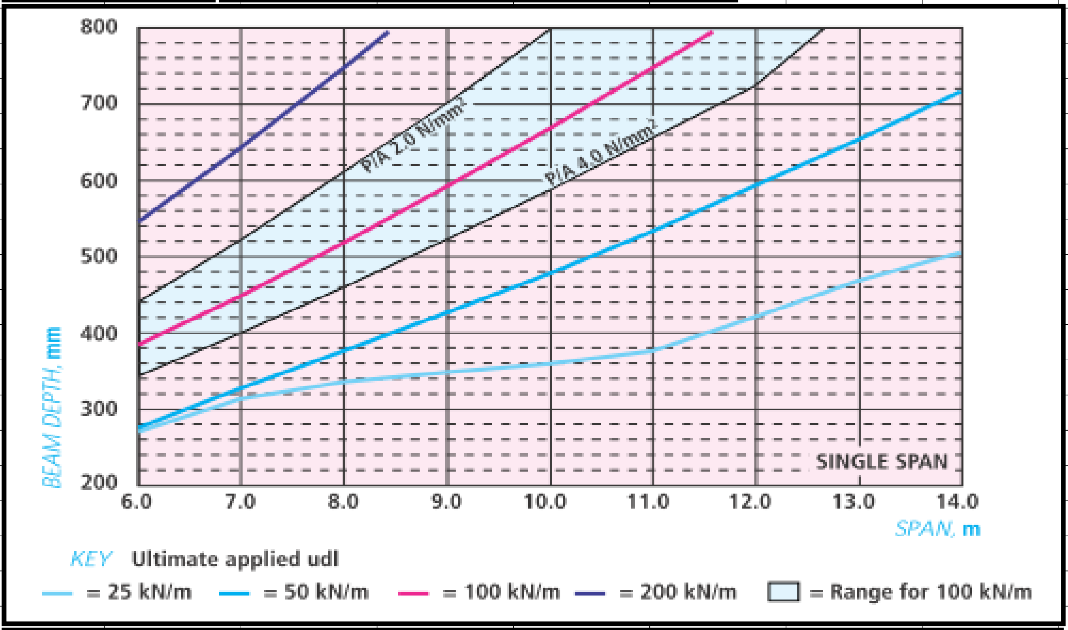
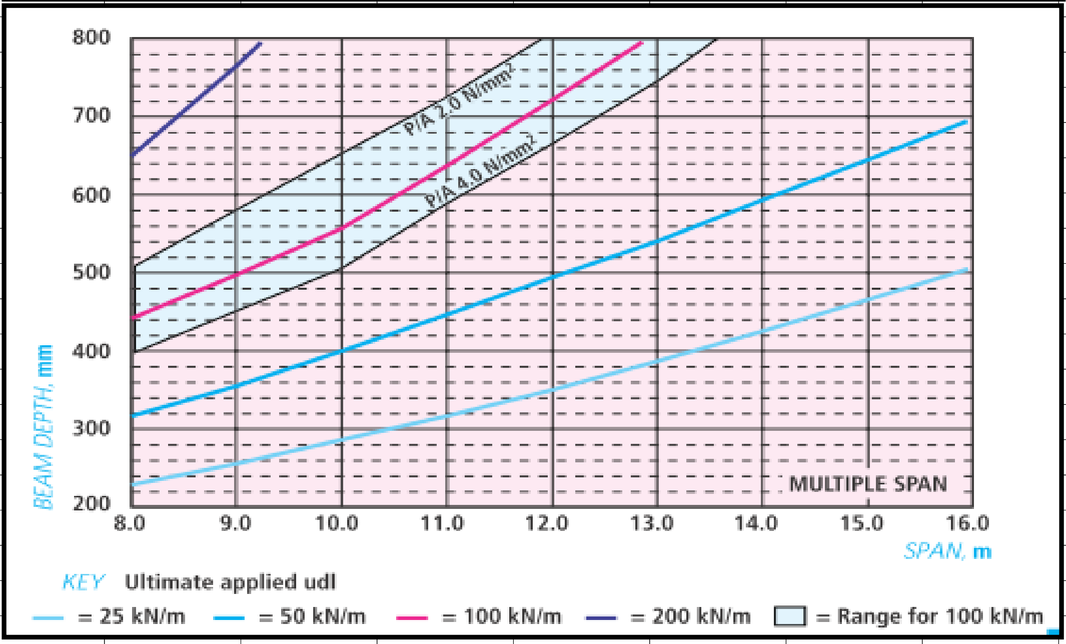
Scheme Design: PT Beam



Rectangular ***1000 mm*** *wide*





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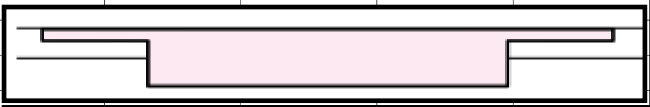
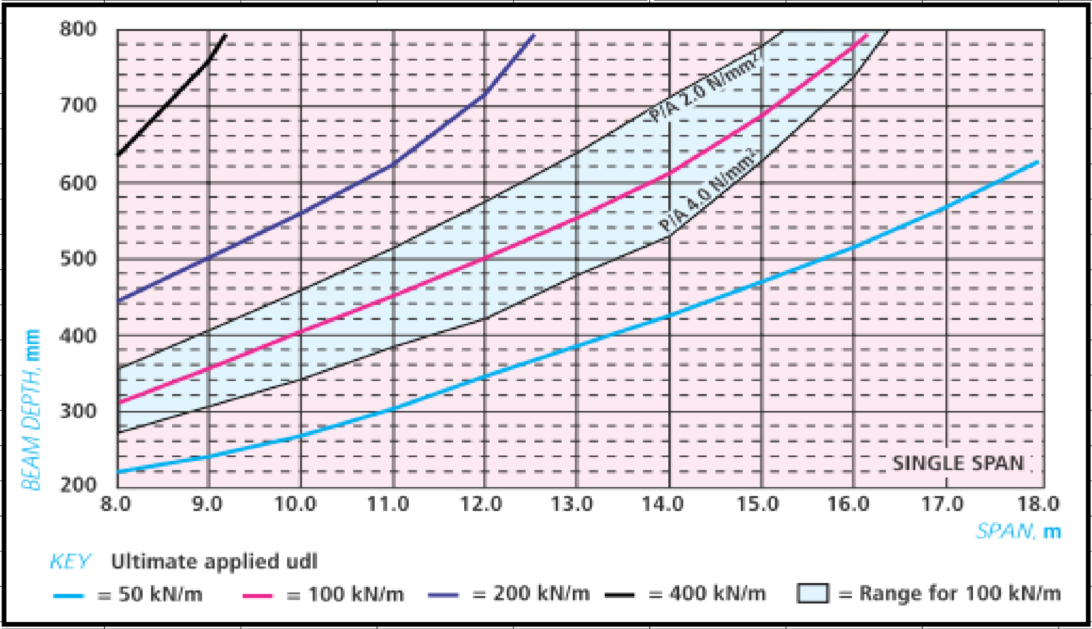
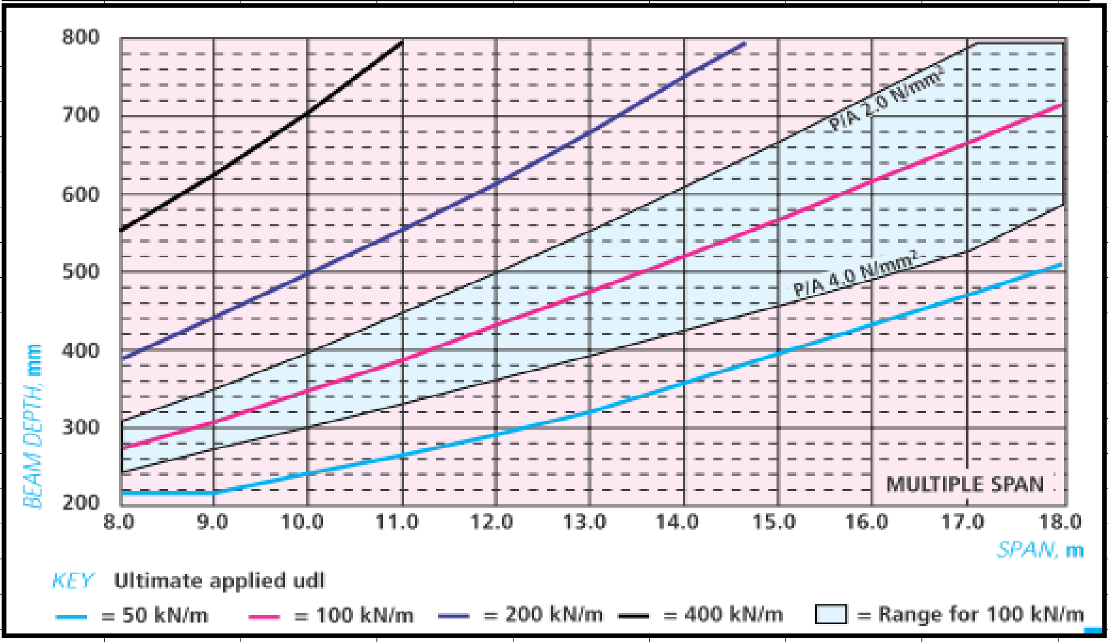


'T' beams 2400 mm wide web





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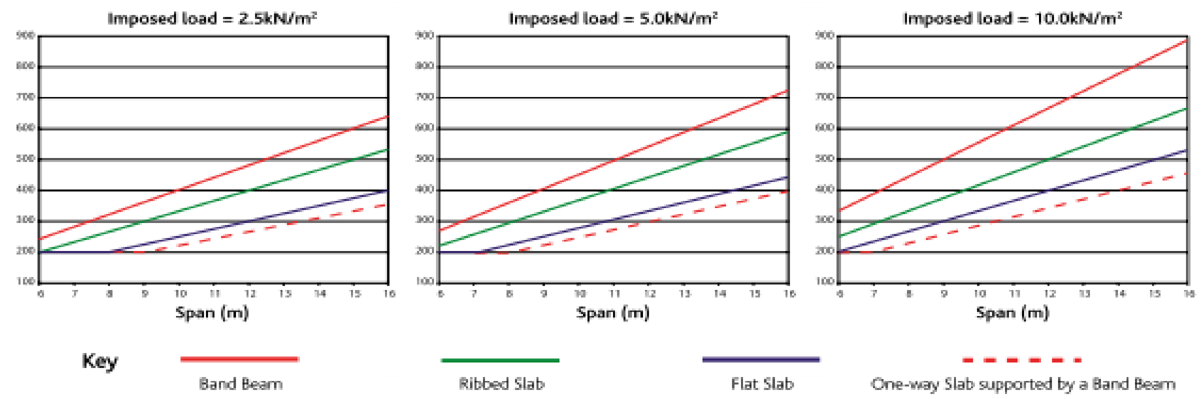
Scheme Design: PT Beam and PT Slab

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

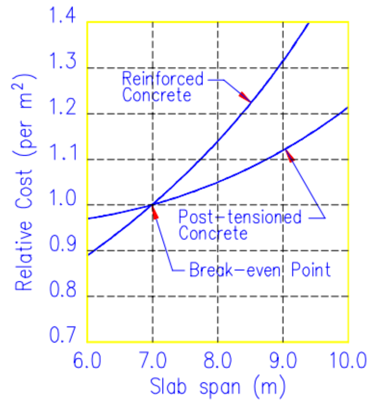
Typically: $P/A \approx 1$ to 2 N/mm²

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

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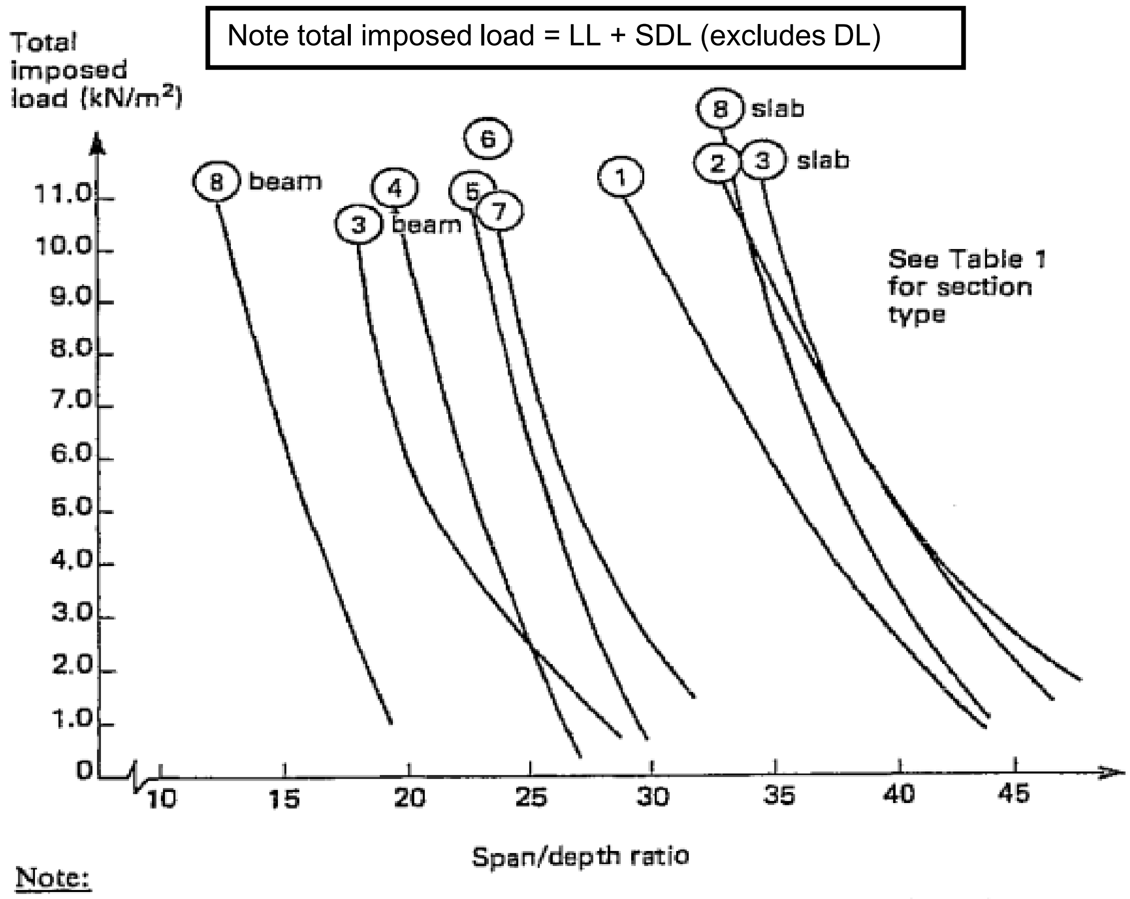
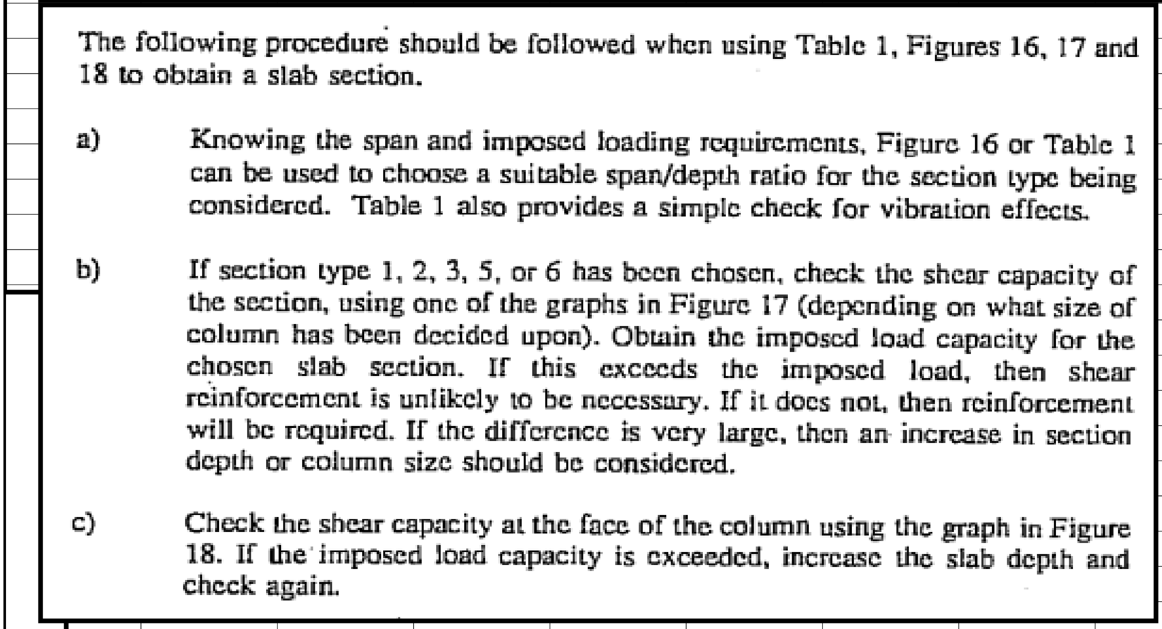


Note:

This chart is derived from the emperical values given in Table 1 for multi-span floors. For single-span floors the depth should be increased by approximately 15%.

Figure 16: Preliminary selection of floor thickness for multi-span floors.



The following procedure should be followed when using Table 1, Figures 16, 17 and 18 to obtain a slab section.

a) Knowing the span and imposed loading requirements, Figure 16 or Table 1 can be used to choose a suitable span/depth ratio for the section type being considered. Table 1 also provides a simple check for vibration effects.

b) If section type 1, 2, 3, 5, or 6 has been chosen, check the shear capacity of the section, using one of the graphs in Figure 17 (depending on what size of column has been decided upon). Obtain the imposed load capacity for the chosen slab section. If this exceeds the imposed load, then shear reinforcement is unlikely to be necessary. If it does not, then reinforcement will be required. If the difference is very large, then an increase in section depth or column size should be considered.

c) Check the shear capacity at the face of the column using the graph in Figure 18. If the imposed load capacity is exceeded, increase the slab depth and check again.

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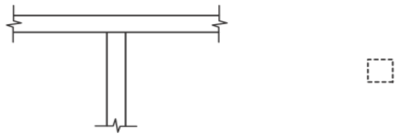
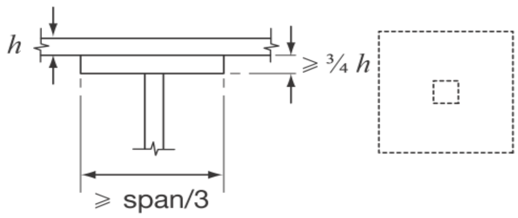
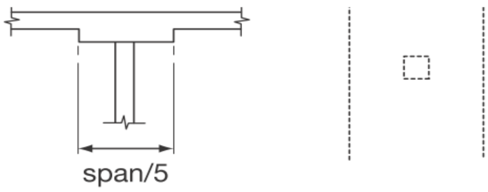
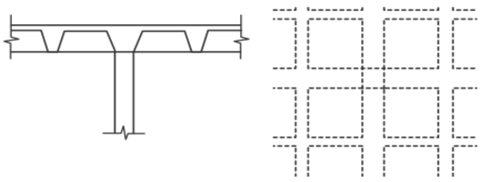
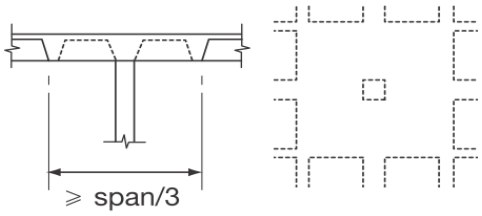
Table 1: Typical span/depth ratios for a variety of section types for multi-span floors.

Section type	Total imposed load (kN/m)	Span/depth ratios 6m ⩽ L ⩽ 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 45 40 35	Beam 25 22 18	A
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

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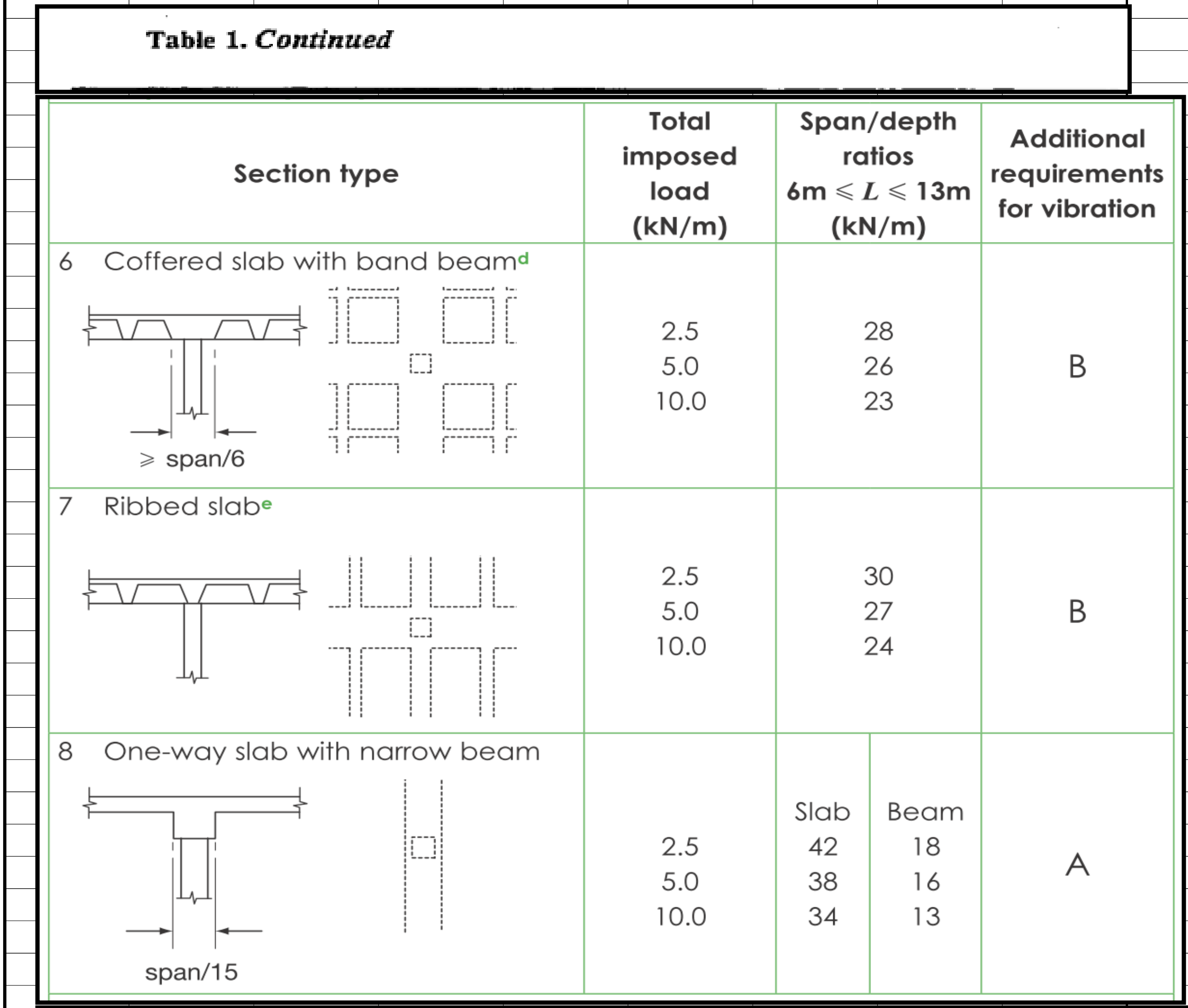
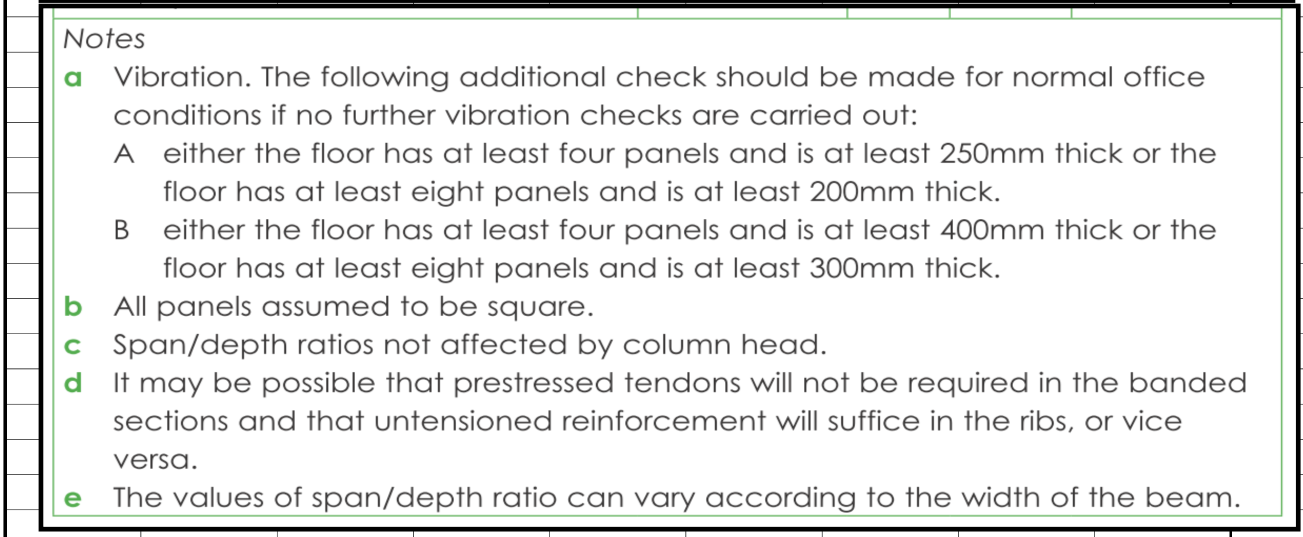


Table 1. *Continued*

Section type	Total imposed load (kN/m)	Span/depth ratios 6m $\leqslant L \leqslant$ 13m (kN/m)		Additional requirements for vibration
6 Coffered slab with band beam[d] ⩾ span/6	2.5 5.0 10.0	28 26 23		B
7 Ribbed slab[e]	2.5 5.0 10.0	30 27 24		B
8 One-way slab with narrow beam span/15	2.5 5.0 10.0	Slab 42 38 34	Beam 18 16 13	A



Notes

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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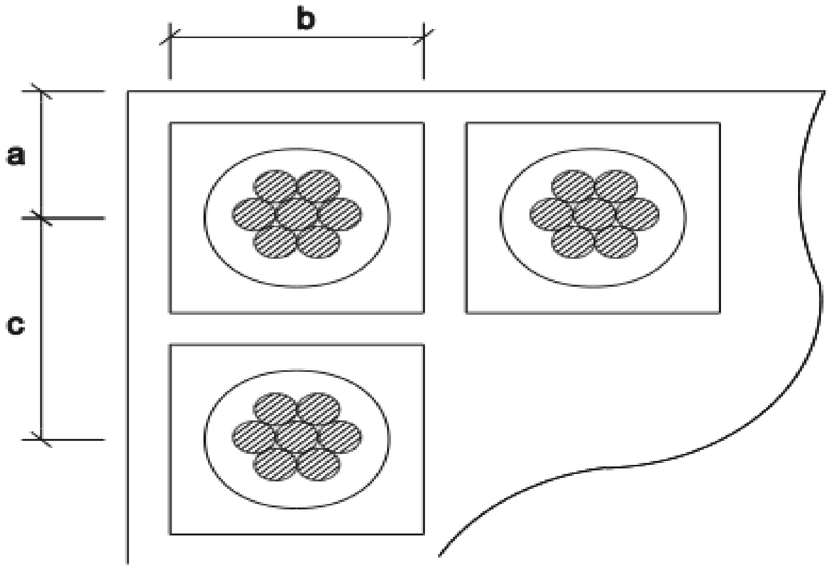
COMMON STRANDS[4]

	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 TENDONS[1]

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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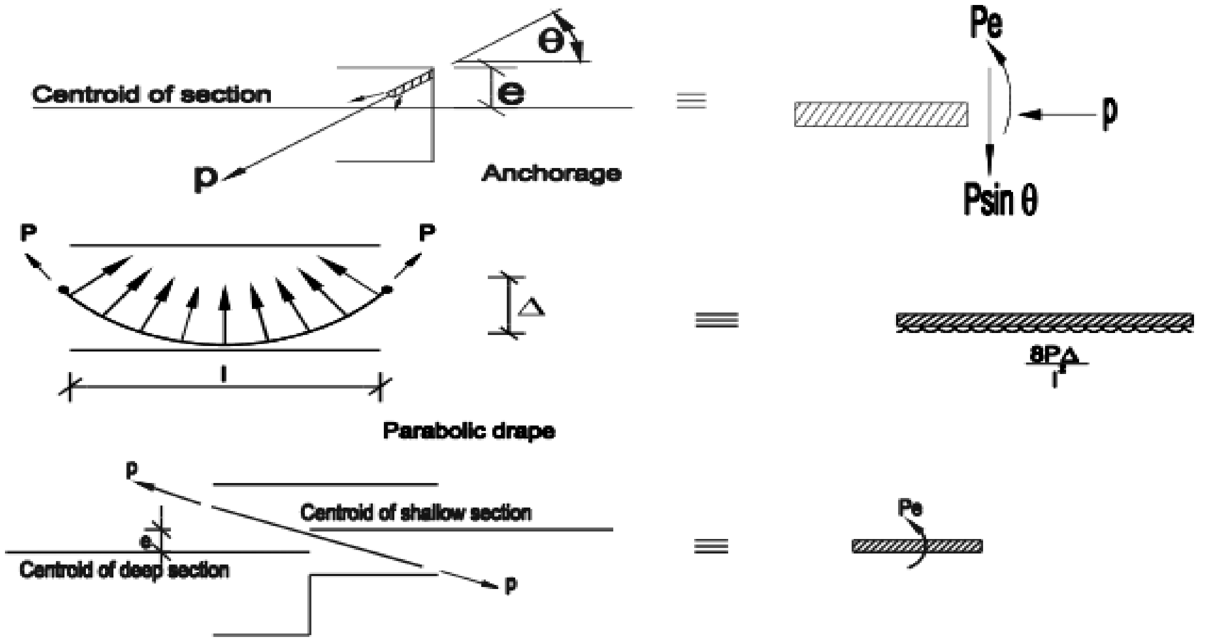
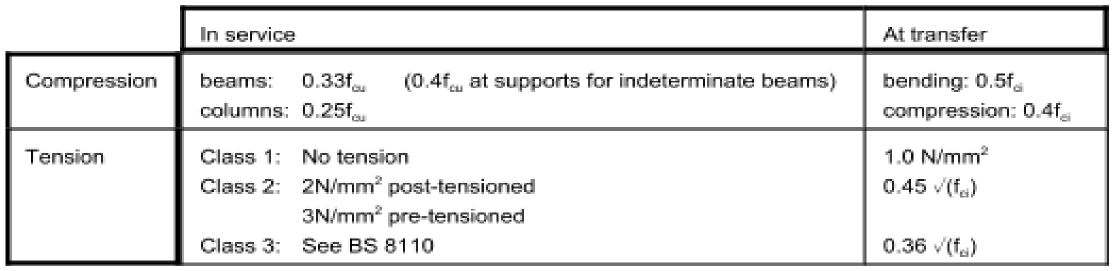
4.3.4 EQUIVALENT LOADS[6]



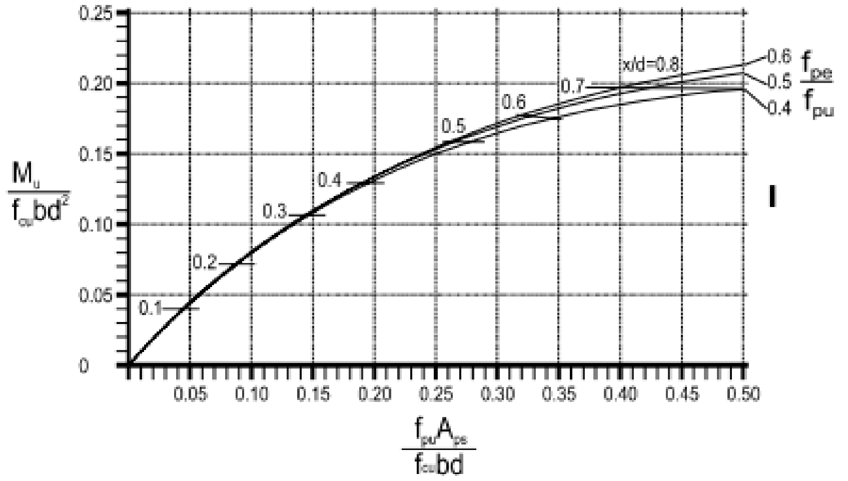
ALLOWABLE STRESSES AT SERVICE LOADS



	In service	At transfer
Compression	beams: $0.33f_{cu}$ ($0.4f_{cu}$ at supports for indeterminate beams) columns: $0.25f_{cu}$	bending: $0.5f_{ci}$ compression: $0.4f_{ci}$
Tension	Class 1: No tension Class 2: 2N/mm² post-tensioned 3N/mm² pre-tensioned Class 3: See BS 8110	1.0 N/mm² $0.45\sqrt{(f_{ci})}$ $0.36\sqrt{(f_{ci})}$

ULTIMATE BENDING STRENGTH[6]

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



SHEAR

Require that $v_u < 0.8 \sqrt{(f_{cu})}$ and $5N/mm^2$

Except that inclined tendons may contribute to a reduced effective shear force on the provided the shear zone is not cracked in bending at M_{ult}.

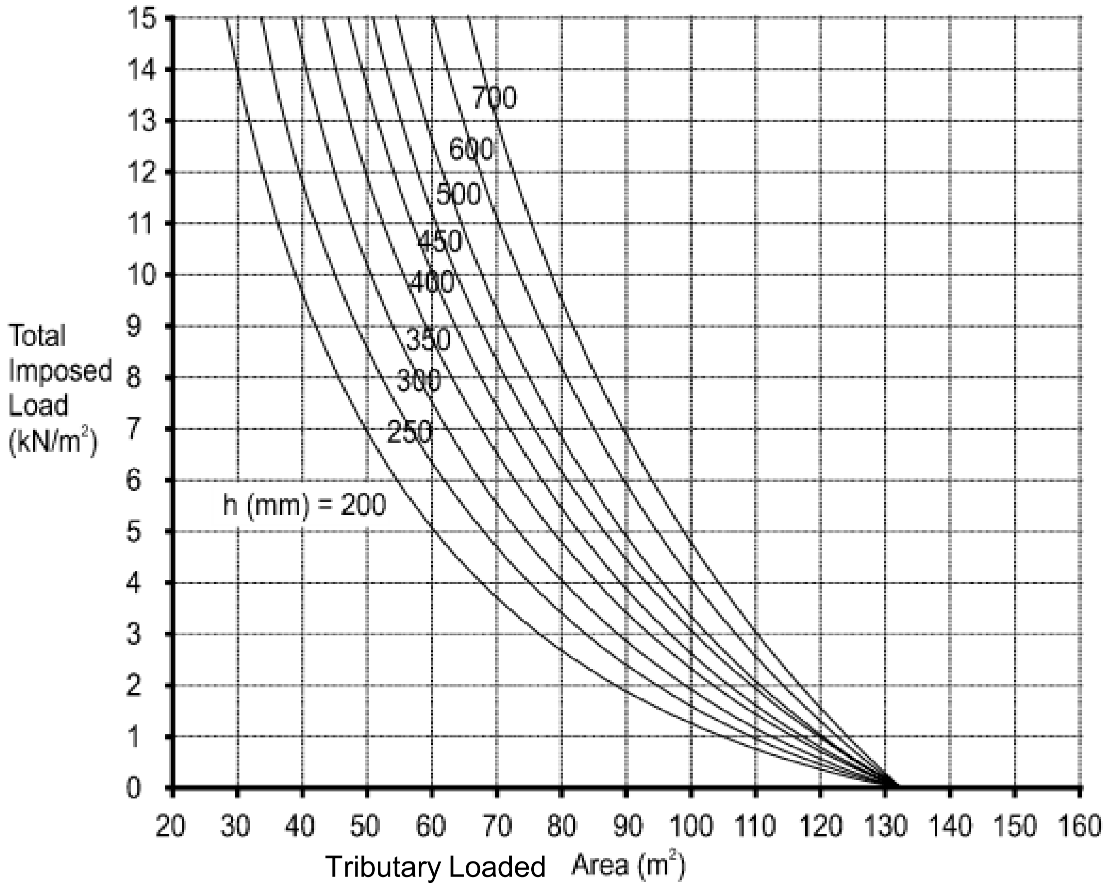
Ultimate shear check at column face

Column (inc. head) 300 x 300

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

Explanation

Figure 18: Ultimate Shear Check at Column Face



Note total imposed load = LL + SDL (excludes DL)
Information to be used in conjunction with the graph:
1. f_{cu} = 40 N/mm²
2. Dead load factor = 1.4
3. Live load factor = 1.6
4. The value of d/h is assumed to be 0.85
5. The ratio of V_{eff}/V is assumed to be 1.15
6. These curves do not take account of elastic distribution effects
7. The maximum shear stress for f_{cu} = 40 N/mm² and more is 5 N/mm².
   For f_{cu} < 40 N/mm² the maximum shear stress is $0.8 \sqrt{f_{cu}}$
   For f_{cu} = 35 N/mm² increase slab depth by a factor of 1.06
   For f_{cu} = 30 N/mm² increase slab depth by a factor of 1.14

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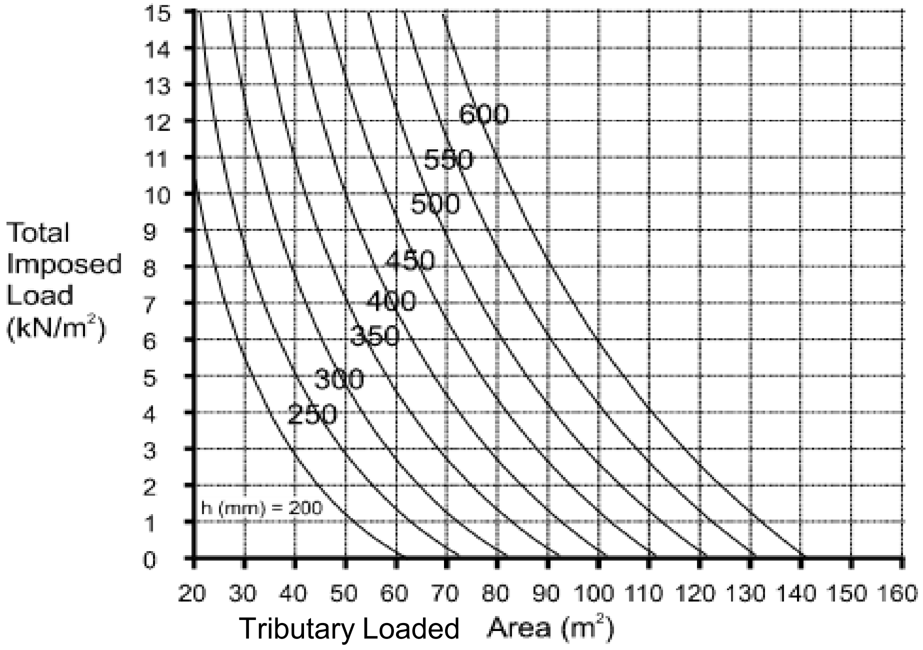
Column 300 x 300
Punching shear check for preliminary design (v_c = 0.75 N/mm²)



Note total imposed load = LL + SDL (excludes DL)

Column 500 x 500
Punching shear check for preliminary design (v_c = 0.75 N/mm²)

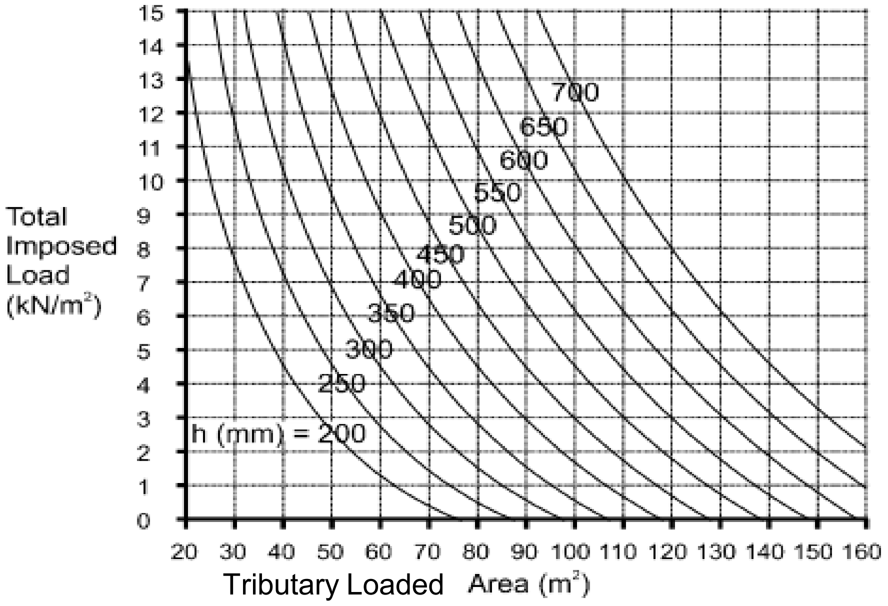


Figure 17: Punching Shear Check at Column Face

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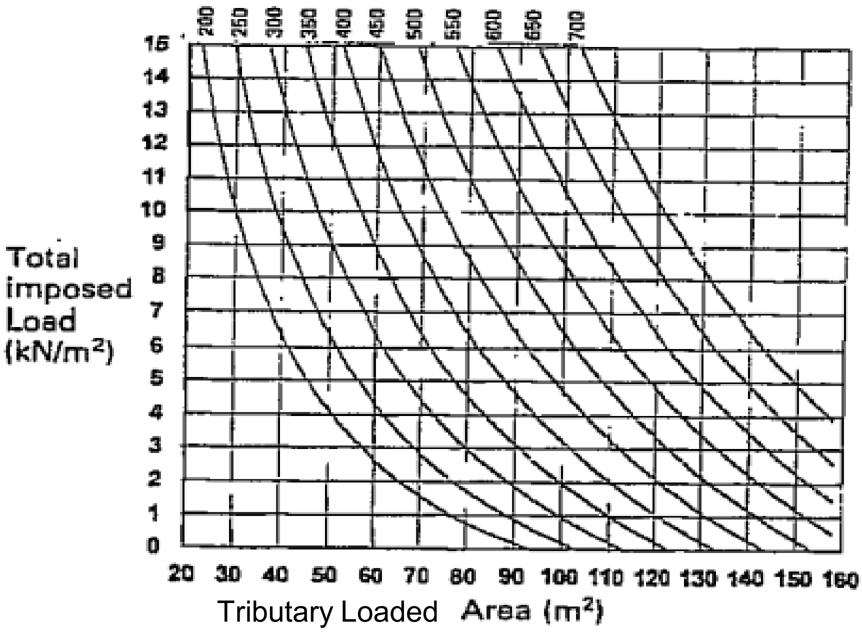

Slab thickness adjacent to columns

vc = 0.75 N/mm^2
Column size including head = 700 x 700 mm

Note total imposed load = LL + SDL (excludes DL)

Figure 17: Preliminary shear check for slab thickness at internal column.

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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Typical Initial Span / Effective Depth Ratios

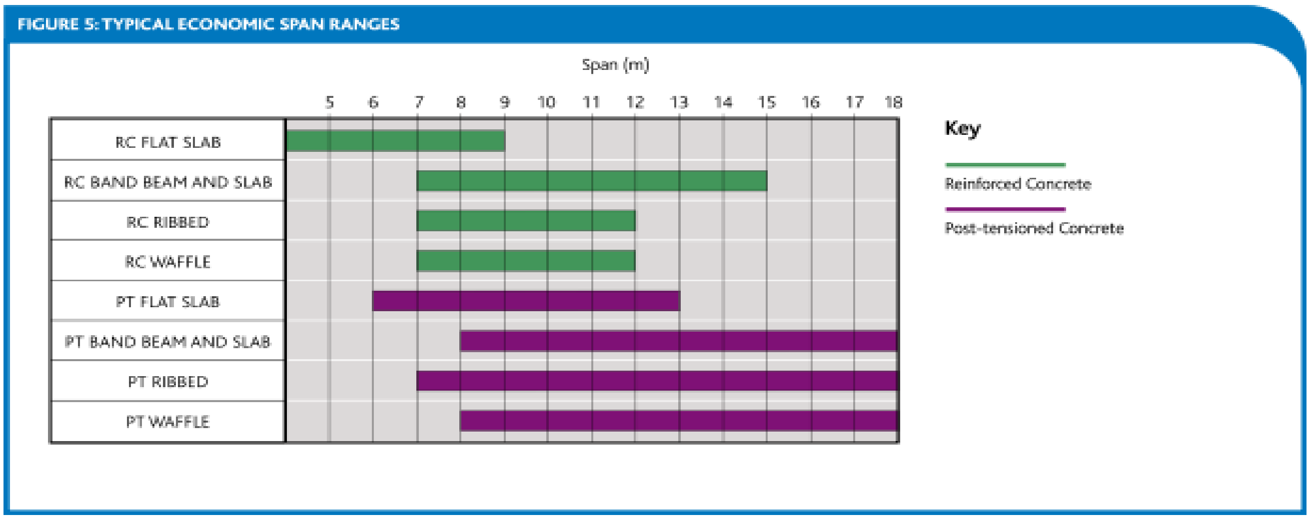
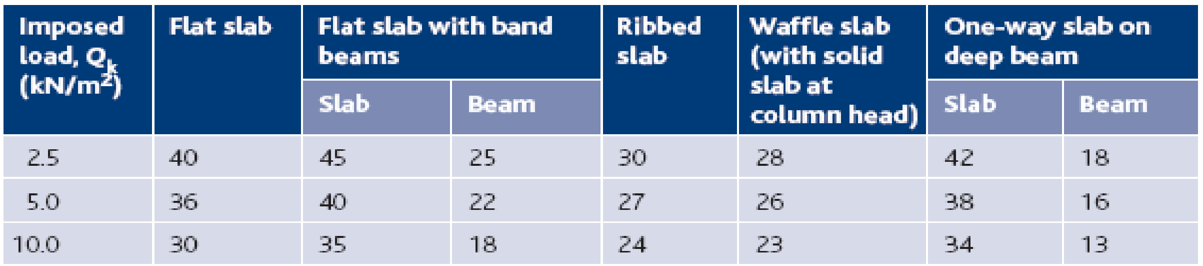


FIGURE 5: TYPICAL ECONOMIC SPAN RANGES

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

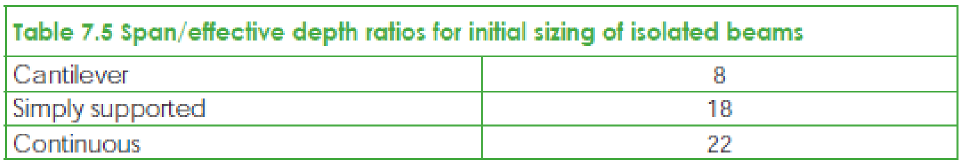


Table 7.5 Span/effective depth ratios for initial sizing of isolated beams

Cantilever	8
Simply supported	18
Continuous	22

TR.43 cl.6.14
Span to Depth Ratios

Solid Slab 42-48
Roof Slab 48-52

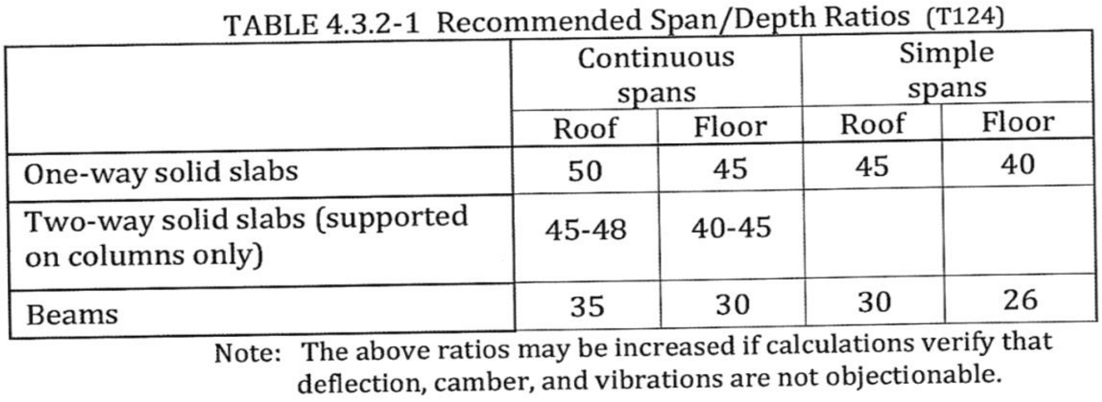


TABLE 4.3.2-1 Recommended Span/Depth Ratios (T124)

	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

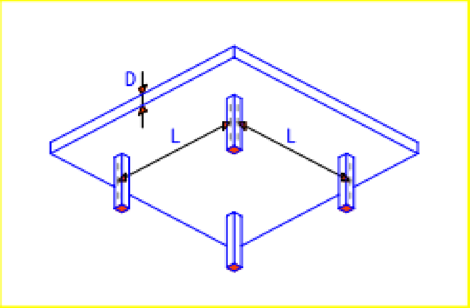
Note: The above ratios may be increased if calculations verify that deflection, camber, and vibrations are not objectionable.

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

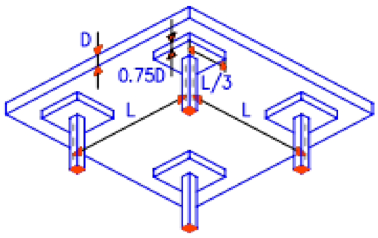
Flat Slab *(figure 10)*

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 5 10	38 35 32
End Span	3 5 10	46 43 40
Internal Span	3 5 10	52 49 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.

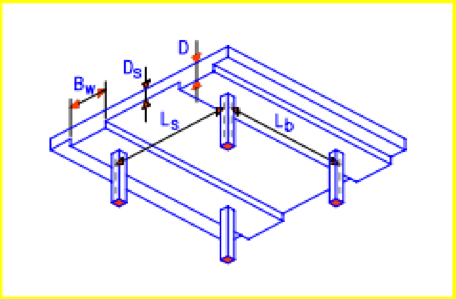
Banded Slab *(figure 11)*

This system is used for structures where spans in one direction are predominant. It is also a very common system due to minimum material costs as well as relatively simple formwork. In most circumstances the width of the band beam is chosen to suit the standard sizes of the formwork. The sides of the band can be either square, or tapered for a more attractive result.

The band beam has a relatively wide, shallow cross section which reduces the overall depth of the floor while permitting longer spans. This concrete section simplifies the formwork and permits services to easily pass under the beams. The post-tensioned tendons are not interwoven leading to fast installation and decreased cycle time.

The band beam system has another advantage which is not widely appreciated. In most circumstances depending on the actual geometry of the cross section the beam can be considered as a two way slab for fire rating and shear design. This enables considerable economies to be achieved in both post-tensioning and reinforcement quantities.

Used	Span predominant in one direction	
Economic Span Range	Band Beam:	8.0 to 15.0 m
	Slab:	6.0 to 10.0 m
Imposed Loads	Up to 15.0 kPa	



		Imposed Load (kPa)	Span/Depth Ratio
Slabs (Ls)	Single Span	3	38
		5	35
		10	32
	End Span	3	46
		5	43
		10	40
	Internal Span	3	52
		5	49
		10	45
Band Beams (Lb)	Single Span	3	20
		5	18
		10	16
	End Span	3	24
		5	22
		10	19
	Internal Span	3	27
		5	25
		10	22

Figure 11. Banded slab. Note that the band width, Bw is generally in the range 0.15Ls to 0.25Ls. The band sides can be square or tapered.

For rectangular grids the band beam and slab solution may be appropriate. This is the system typically used for shopping centres and carparks due to the economic benefit and relative insensitivity to floor height restrictions.

Normally band beams span in the long direction and impose the same constraints on hole placement as would a steel or reinforced concrete beam. However, small hydraulic type penetrations (approximately 150 mm diameter) can usually be accommodated without the need for remedial action.

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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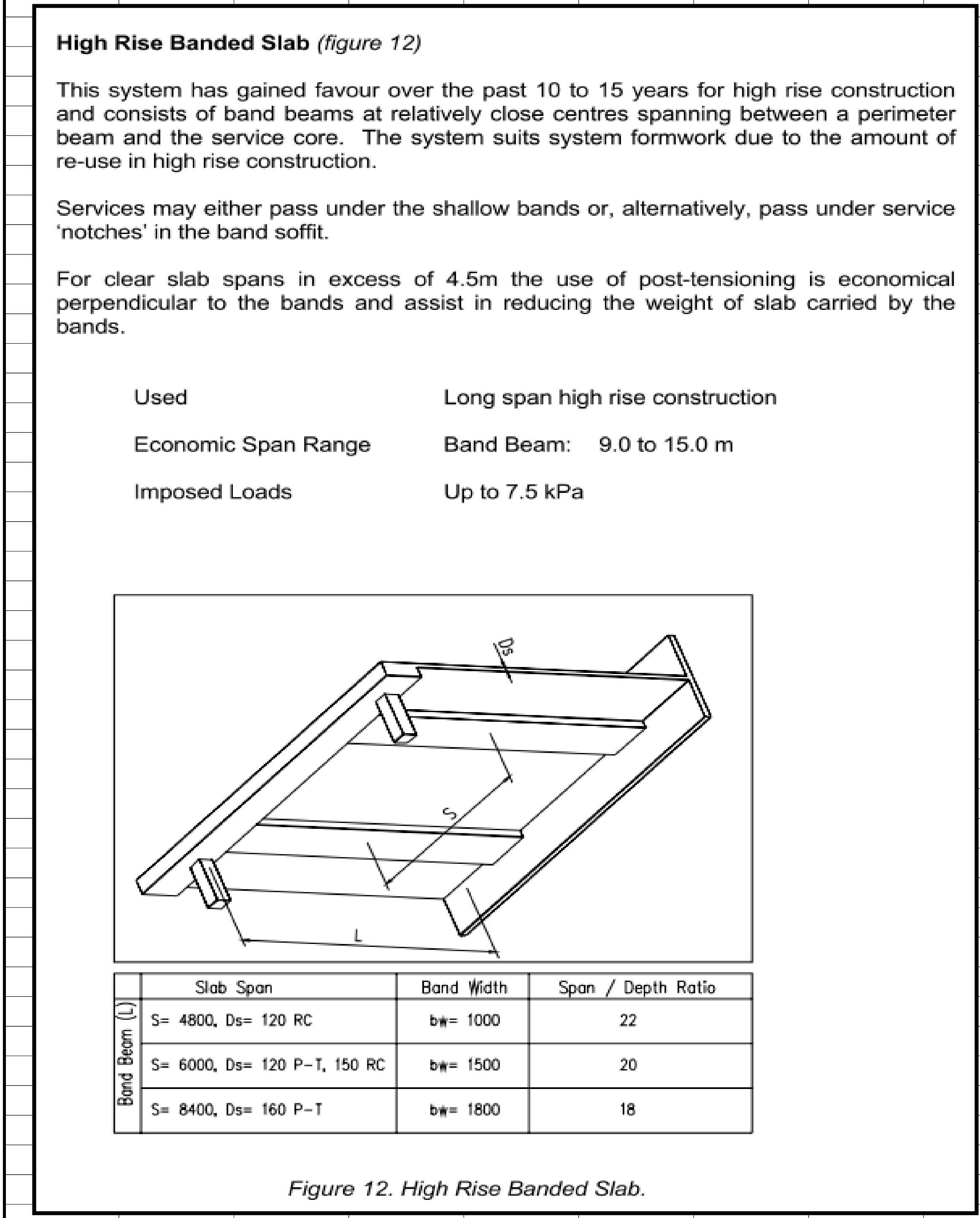

High Rise Banded Slab *(figure 12)*

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
Band Beam (L)	S= 4800, Ds= 120 RC	b_w= 1000	22
	S= 6000, Ds= 120 P-T, 150 RC	b_w= 1500	20
	S= 8400, Ds= 160 P-T	b_w= 1800	18

Figure 12. High Rise Banded Slab.

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Typical Initial Span / Effective Depth Ratios

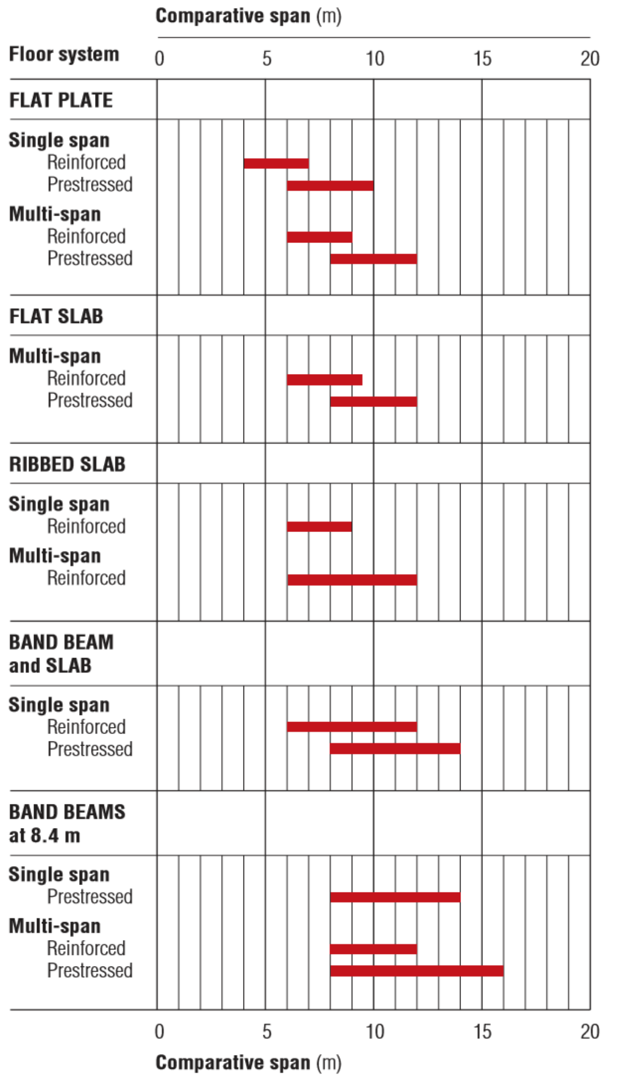

FIGURE 8: Quick selection guide for insitu floors

The span 'L' of a reinforced concrete flat-plate is approximately D x 28 for simply supported, D x 30 for an end span of a continuous system, to D x 32 for internal continuous spans. The economical span of a flat plate can be extended by prestressing to approximately D x 30, D x 37 and D x 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 x 28 for simply supported, D x 32 for an end span and D x 36 for an interior span. Prestressing the slab increases the economical span to D x 35, D x 40 and D x 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 x 20 to D x 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 x 24 to D x 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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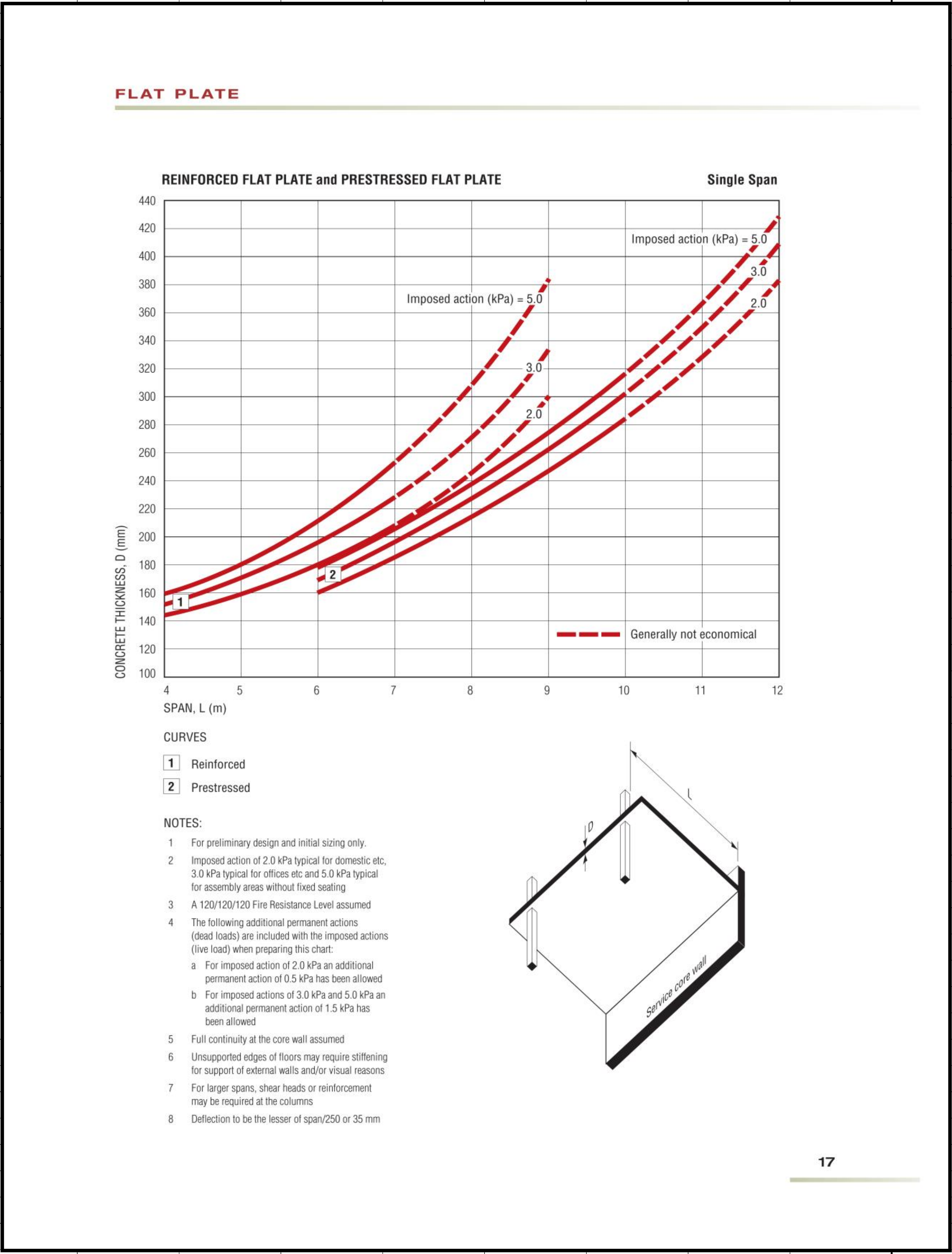


FLAT PLATE

REINFORCED FLAT PLATE and PRESTRESSED FLAT PLATE **Single Span**

CURVES

1 Reinforced

2 Prestressed

NOTES:

1 For preliminary design and initial sizing only.

2 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

3 A 120/120/120 Fire Resistance Level assumed

4 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

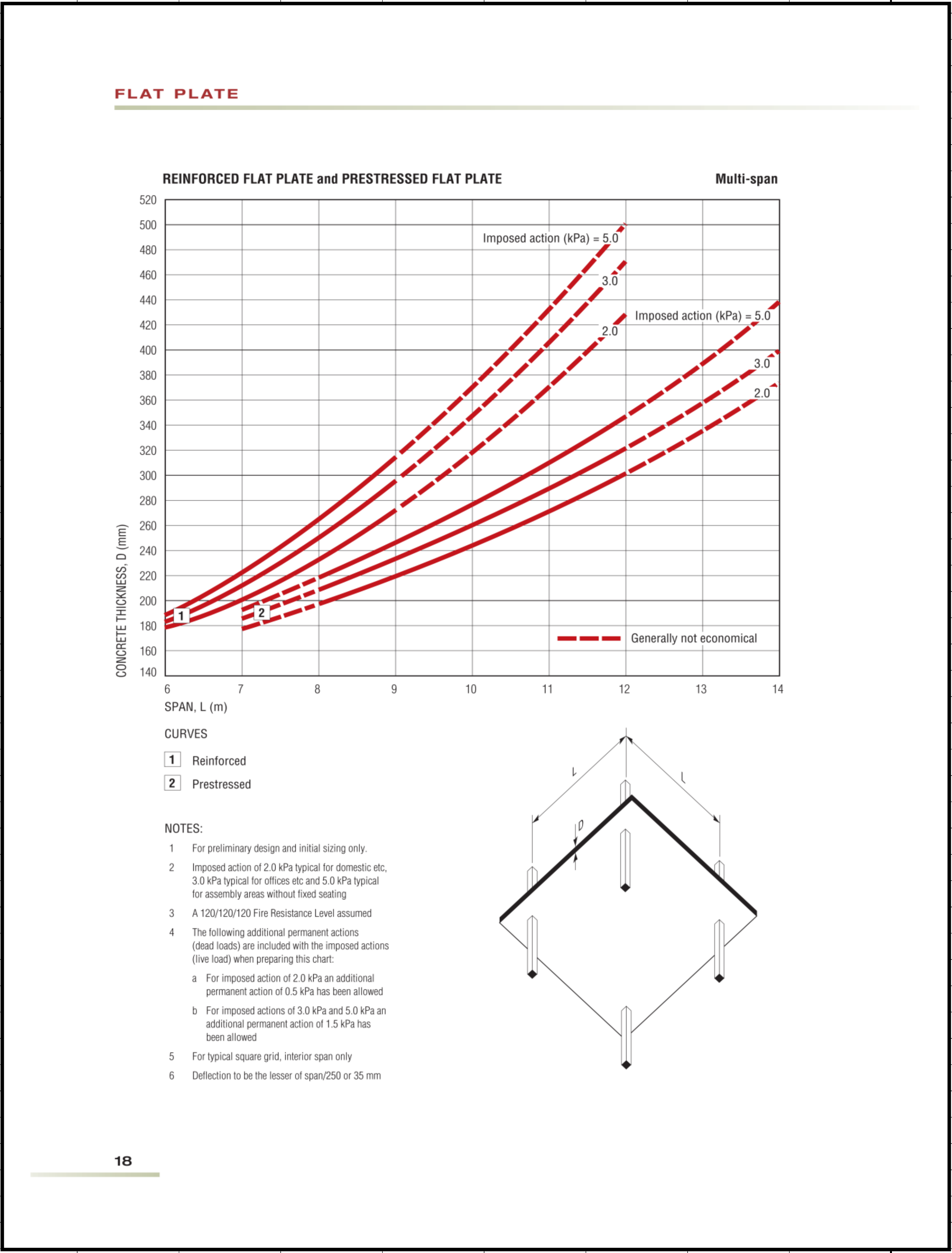
5 Full continuity at the core wall assumed

6 Unsupported edges of floors may require stiffening for support of external walls and/or visual reasons

7 For larger spans, shear heads or reinforcement may be required at the columns

8 Deflection to be the lesser of span/250 or 35 mm

17



FLAT PLATE

REINFORCED FLAT PLATE and PRESTRESSED FLAT PLATE **Multi-span**

CURVES

1 Reinforced

2 Prestressed

NOTES:

1 For preliminary design and initial sizing only.

2 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

3 A 120/120/120 Fire Resistance Level assumed

4 The following additional permanent actions (dead loads) are included with the imposed actions (live load) when preparing this chart:

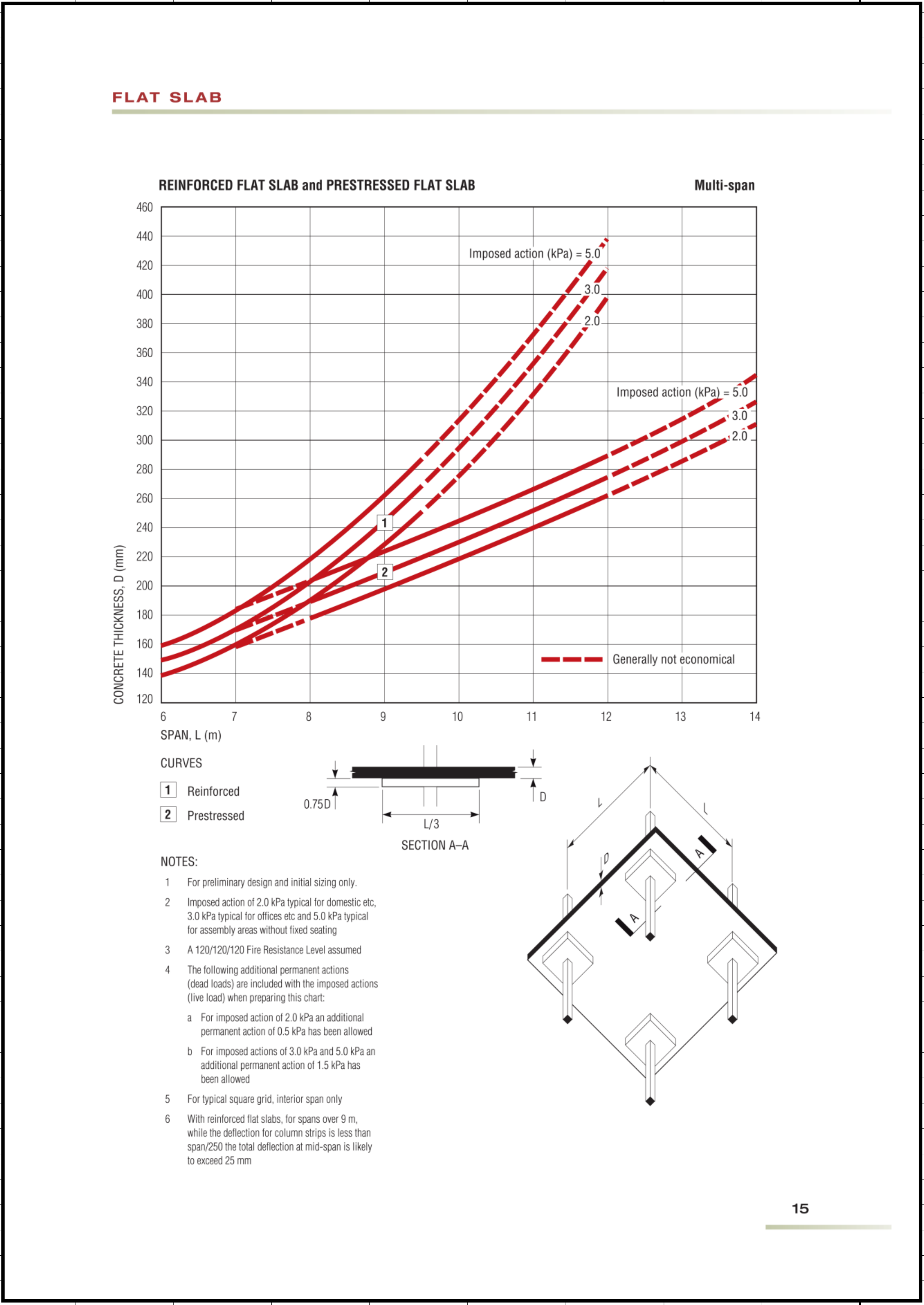
   a For imposed action of 2.0 kPa an additional permanent action of 0.5 kPa has been allowed

   b For imposed actions of 3.0 kPa and 5.0 kPa an additional permanent action of 1.5 kPa has been allowed

5 For typical square grid, interior span only

6 Deflection to be the lesser of span/250 or 35 mm

18



FLAT SLAB

REINFORCED FLAT SLAB and PRESTRESSED FLAT SLAB

Multi-span

CURVES

1 Reinforced

2 Prestressed

NOTES:

1 For preliminary design and initial sizing only.

2 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

3 A 120/120/120 Fire Resistance Level assumed

4 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

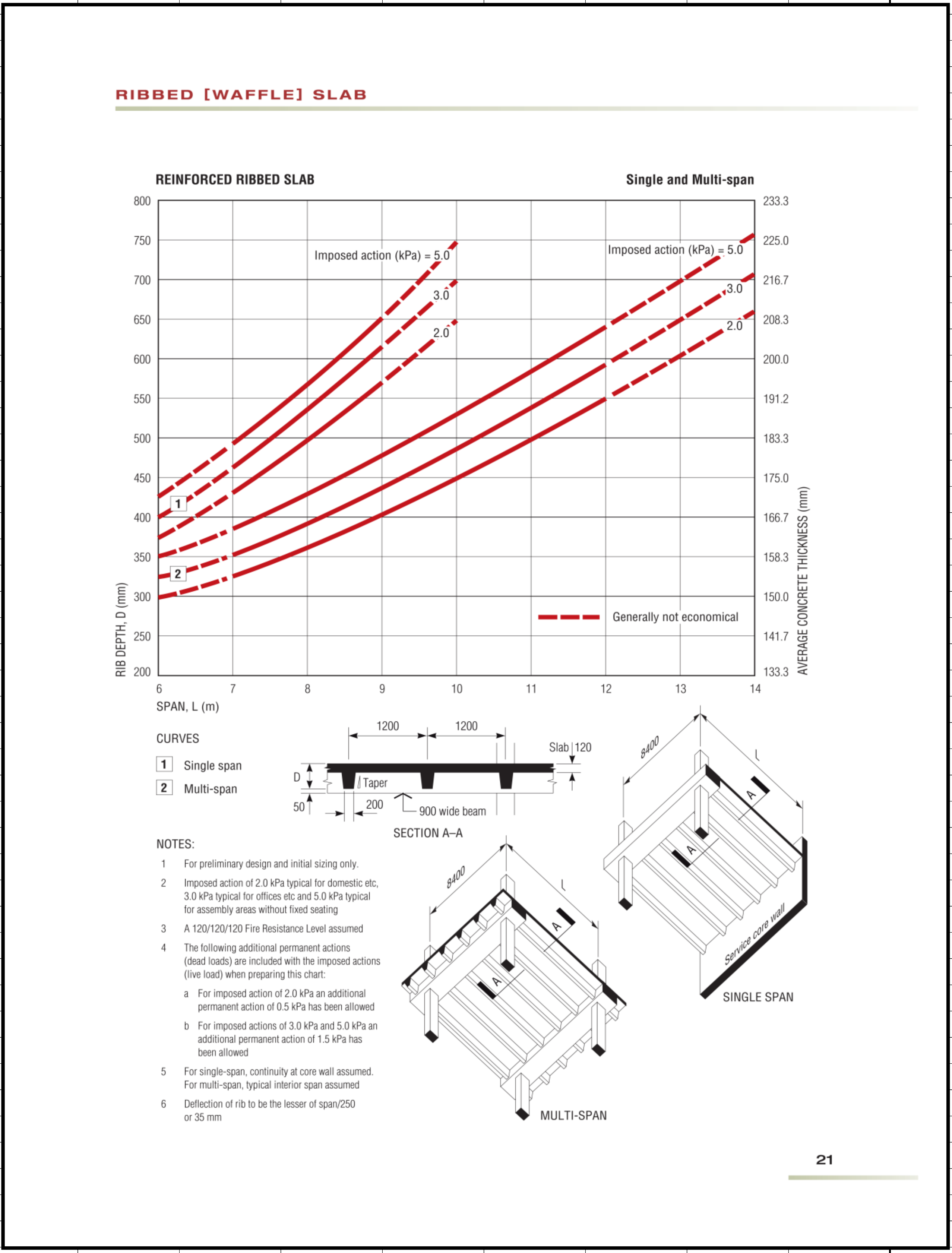
  b For imposed actions of 3.0 kPa and 5.0 kPa an additional permanent action of 1.5 kPa has been allowed

5 For typical square grid, interior span only

6 With reinforced flat slabs, for spans over 9 m, while the deflection for column strips is less than span/250 the total deflection at mid-span is likely to exceed 25 mm

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers	Job No.	Sheet No.	Rev.
		jXXX	95	
		Member/Location		
Job Title	Member Design - Prestressed Concrete Beam and Slab	Drg. Ref.		
Member Design - PC Beam and Slab		Made by XX	Date 18/08/2025	Chd.

BS8110



RIBBED [WAFFLE] SLAB

REINFORCED RIBBED SLAB

Single and Multi-span

RIB DEPTH, D (mm): 800, 750, 700, 650, 600, 550, 500, 450, 400, 350, 300, 250, 200

AVERAGE CONCRETE THICKNESS (mm): 233.3, 225.0, 216.7, 208.3, 200.0, 191.2, 183.3, 175.0, 166.7, 158.3, 150.0, 141.7, 133.3

SPAN, L (m): 6, 7, 8, 9, 10, 11, 12, 13, 14

Imposed action (kPa) = 5.0

3.0

2.0

Imposed action (kPa) = 5.0

3.0

2.0

1

2

Generally not economical

CURVES

1 Single span

2 Multi-span

1200 1200

Slab 120

D

Taper

50

200

900 wide beam

SECTION A–A

8400

L

A

A

Service core wall

SINGLE SPAN

8400

L

A

A

MULTI-SPAN

NOTES:

1 For preliminary design and initial sizing only.

2 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

3 A 120/120/120 Fire Resistance Level assumed

4 The following additional permanent actions (dead loads) are included with the imposed actions (live load) when preparing this chart:

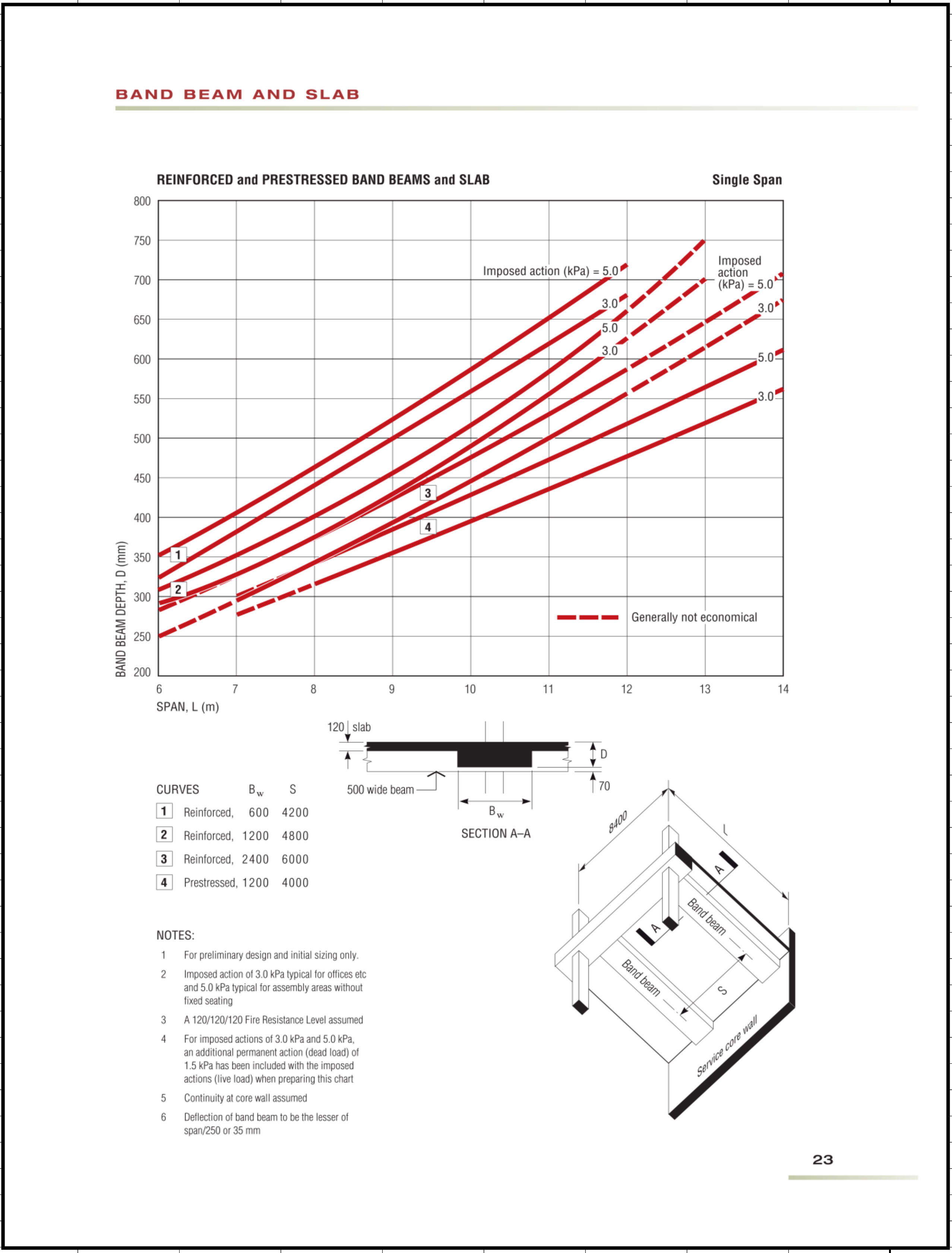
- a For imposed action of 2.0 kPa an additional permanent action of 0.5 kPa has been allowed
- b For imposed actions of 3.0 kPa and 5.0 kPa an additional permanent action of 1.5 kPa has been allowed

5 For single-span, continuity at core wall assumed. For multi-span, typical interior span assumed

6 Deflection of rib to be the lesser of span/250 or 35 mm

21

BS8110



BAND BEAM AND SLAB

REINFORCED and PRESTRESSED BAND BEAMS and SLAB

Single Span

CURVES		B_w	S
1	Reinforced,	600	4200
2	Reinforced,	1200	4800
3	Reinforced,	2400	6000
4	Prestressed,	1200	4000

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

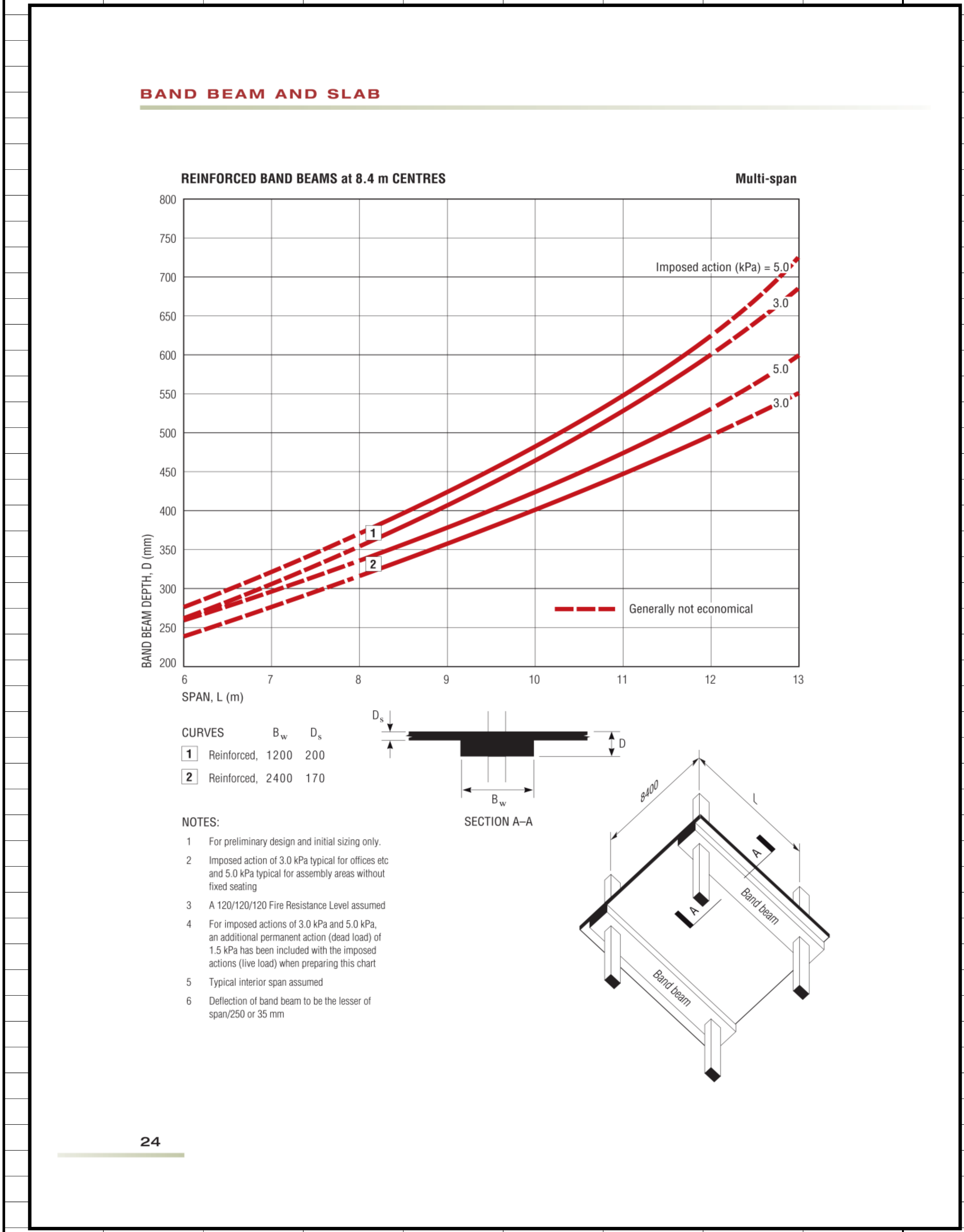
5 Continuity at core wall assumed

6 Deflection of band beam to be the lesser of span/250 or 35 mm

23

CONSULTING ENGINEERS Engineering Calculation Sheet
Consulting Engineers

Job No. jXXX
Sheet No. 97
Rev.
Member/Location
Job Title Member Design - Prestressed Concrete Beam and Slab
Drg. Ref.
Member Design - PC Beam and Slab
Made by **XX** Date **18/08/2025** Chd.
BS8110



BAND BEAM AND SLAB

REINFORCED BAND BEAMS at 8.4 m CENTRES

Multi-span

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

24

CONSULTING ENGINEERS Engineering Calculation Sheet
Consulting Engineers

Job No.	Sheet No.	Rev.
jXXX	98	

Member/Location

Job Title Member Design - Prestressed Concrete Beam and Slab

Drg. Ref.

Member Design - PC Beam and Slab

Made by **XX** Date **18/08/2025** Chd.

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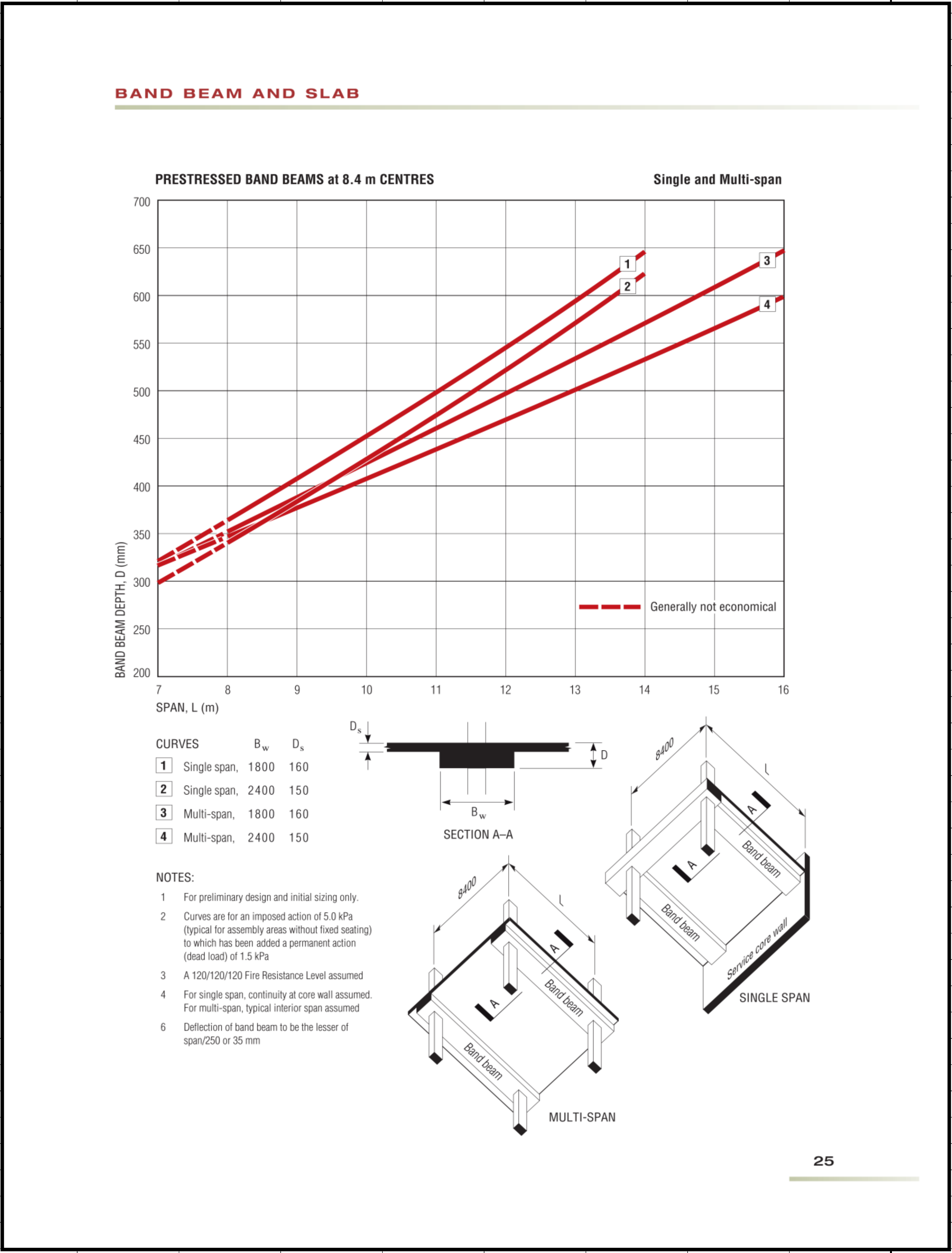

BAND BEAM AND SLAB

PRESTRESSED BAND BEAMS at 8.4 m CENTRES **Single and Multi-span**

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:

1 For preliminary design and initial sizing only.

2 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

3 A 120/120/120 Fire Resistance Level assumed

4 For single span, continuity at core wall assumed. For multi-span, typical interior span assumed

6 Deflection of band beam to be the lesser of span/250 or 35 mm