

CONSULTING ENGINEERS	Engineering Calculation Sheet Consulting Engineers					Job No.	Sheet No.		Rev.	
						jXXX	1			
						Member/Location				
Job Title	Structure, Member Design - Precast Concrete Frame, Beam					Drg. Ref.				
Structure, Member Design - PC Frame, Beam and Slab						Made by	XX	Date	2/10/2026	
									Chd.	
Material Properties										
Characteristic strength of concrete (insitu topping), $f_{cu,top} / f_{ck,top} f_{c,to}$						45	▼	35	▼	N/mm ²
Characteristic strength of concrete (precast beam), $f_{cu} / f_{ck} f_c'$						55	▼	45	▼	N/mm ²
Note require $f_{cu} \geq 40\text{N/mm}^2$ (pre-T) cl.4.1.8.1 BS8110, $\leq 105\text{N/mm}^2$ HSC;										OK
Characteristic strength of concrete at transfer (precast beam), f_{ci} / f_{ci}'						45	▼	35	▼	N/mm ²
Note require $f_{ci} \geq 25\text{N/mm}^2$ cl.4.1.8.1 BS8110;										OK
Yield strength of longitudinal steel, f_y						Higher	▼	500	▼	N/mm ²
Yield strength of shear link steel, f_{yv}						Higher	▼	500	▼	N/mm ²
Elastic modulus of reinforcement steel, E_s								200,000		N/mm ²
Type of concrete and density, ρ_c						Normal Weight 25kN/m3	▼	25		kN/m ³
ULS Load Combination Factors								EC2	▼	
DL+SDL [G] and LL [Q] factors for ULS, k_G and k_Q						1.35		1.50		
British Standards [BS8110-1:1997]						1.40		1.60		cl.2.4.3.1.1
Indian Standards [IS456:2000]						1.50		1.50		T.18
European Standards [EN1990:2002]						1.35		1.50		cl.6.4.3.2
American Standards [ASCE7-16]						1.20		1.60		cl.2.3.1
Australian Standards [AS1170.0:2002]						1.20		1.50		cl.4.2
Section Dimensions and Reinforcement [Stage 1: Precast Simply-Supported]										
Slab insitu depth, $insitu_{slab}$								100	mm	$f_{cu,top} / f_{ck,top}$
Beam (precast) width, $b_{w1} = b_w$								850	mm	f_{cu}
Beam (precast) depth, h_1								900	mm	
Cover, $cover_{bot1}$ (usually 35 (C35) or 30 (C40) internal; 40 external)								25	mm	
Sag steel reinforcement diameter, ϕ_{t1}								25	▼	mm
Sag steel reinforcement number, n_{t1}								21		
Sag steel area provided, $A_{s,sag,prov} = n_{t1} \cdot \pi \cdot \phi_{t1}^2 / 4$								10308	mm ²	
Number of layers of sag steel, $n_{layers,sag1}$								3	layer(s)	
Spacer for sag steel, $s_{r,sag1} (\geq \text{MAX}(\phi_{t1}, 25\text{mm}))$								32	mm	OK
Shear link diameter, ϕ_{link}								16	▼	mm
Number of shear links in a cross section, i.e. number of legs, n_{leg}								6		
Area provided by all shear links in a cross-section, $A_{sv,prov} = \pi \cdot \phi_{link}^2 / 4 \cdot n_{leg}$								1206	mm ²	
Pitch of shear links, S								250	mm	
Eff. depth to sag steel, $d_{sag1} = h_1 - cover_{bot1} - \phi_{link} - [\phi_{t1} + (n_{layers,sag1} - 1)(\phi_{t1} + s_{r,sag1})]$								790	mm	
Eff. depth to sag prestress strands, d_{ps1}								683	mm	
Section Dimensions [Stage 2: Insitu]										
Beam (comp.) width, $b_{w2} = b_w$								850	mm	
Beam insitu depth, $insitu_{beam}$								300	mm	
Beam (comp.) depth, $h_2 = h_1 + insitu_{beam}$								1200	mm	
Cover, $cover_{top2}$ (usually 35 (C35) or 30 (C40) internal; 40 external)								25	mm	
Add cover to hog steel (due to transverse steel layer(s)), $cover_{add,t}$								35	mm	
Hog steel reinforcement diameter, ϕ_{t2}								32	▼	mm
Hog steel reinforcement number, n_{t2}								9		
Hog steel area provided, $A_{s,hog,prov} = n_{t2} \cdot \pi \cdot \phi_{t2}^2 / 4$								7238	mm ²	eq
Number of layers of hog steel, $n_{layers,hog2}$								2	layer(s)	
Spacer for hog steel, $s_{r,hog2} (\geq \text{MAX}(\phi_{t2}, 25\text{mm}))$								32	mm	OK
										strand
Eff. depth to sag steel, $d_{sag2} = h_2 - cover_{bot1} - \phi_{link} - [\phi_{t1} + (n_{layers,sag1} - 1)(\phi_{t1} + s_{r,sag1})]$								1090	mm	
Eff. depth to hog steel, $d_{hog2} = h_2 - cover_{top2} - \text{MAX}(\phi_{link}, cover_{add,t}) - [\phi_{t2} + (n_{layers,hog2} - 1)(\phi_{t2} + s_{r,hog2})]$								1092	mm	
Eff. depth to sag prestress strands, d_{ps2}								983	mm	

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$\sigma_{cp,st,db} = -P_{st,db}/A_1$ =				-0.4	0.0	0.0	0.0	0.0	-0.4 N/mm ²																																																																																																																																																																																																																																																																																																																																																														
$M_{PT,st,db} = P_{st,db} \cdot e_s$ =				-133	0	0	0	0	-133 kNm																																																																																																																																																																																																																																																																																																																																																														

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CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.	Rev.	
						jXXX	5		
						Member/Location			
Job Title						Structure, Member Design - Precast Concrete Frame, Bea			
Structure, Member Design - PC Frame, Beam and Slab						Drg. Ref.			
						Made by	XX	Date	2/10/2026
									EC2
Loading and Structural Analysis [Stage 2: Insitu]									
Code of practice for presentation of ULS load combination factors						EC2	▼		
Beam span, L_2						10.000	m		
Tributary width (grid to grid), t_{w2}						10.000	m		
SDL on slab (pressure load), SDL_{slab}						4.00	kPa		
SDL on beam (line load), SDL_{beam}						0.0	kN/m		
SDL on beam (midspan point load), SDL_{point}						0.0	kN		
LL on slab (pressure load), LL_{slab} [with pattern loading]						15.00	kPa		
SLS beam loading (line load), ω_{SLS2}						190.0	kN/m		
Note $\omega_{SLS2} = (SDL_{slab} + LL_{slab}) \cdot t_{w2} + SDL_{beam}$									
ULS beam loading (line load), ω_{ULS2}						279.0	kN/m		
Note $\omega_{ULS2} = (k_G \cdot SDL_{slab} + k_Q \cdot LL_{slab}) \cdot t_{w2} + k_G \cdot SDL_{beam}$									
Beam ULS sag bending moment, $M_{2,sag}$						Continuous Internal Span	▼	2183	kNm
Note $M_{2,sag} = k_{DL SDL,sag} \cdot \omega_{ULS1} \cdot L_2^2$ (if propped only) + $k_{DL SDL,sag} \cdot k_G \cdot [SDL_{slab} \cdot t_{w2} + SDL_{beam}] \cdot L_2^2$									
						$k_{DL SDL,sag}$	$k_{SDL,p,sag}$	$k_{LL,sag}$	
Continuous Internal Span						0.046	0.125	0.086	
Continuous End Span						0.078	0.170	0.100	
Simply-Supported						0.125	0.250	0.125	
Beam SLS sag bending moment, $M_{2,sag,SLS}$								1474	kNm
Note $M_{2,sag,SLS}$ calculated as per $M_{2,sag}$ but without load combination factors.									
Beam bottom SLS stress stage 2 loading (comp. -ve), $\sigma_{b2,SLS}$								4.39	N/mm ²
Note $\sigma_{b2,SLS} = M_{2,sag,SLS} / Z_{b2}$									
Beam bottom SLS stress at end of stage 2 (comp. -ve), $\sigma_{b1,SLS} + \sigma_{b2,SLS}$								2.36	N/mm ²
Beam total ULS sag bending moment, $M_{1,sag} + M_{2,sag}$								3728	kNm
Beam ULS hog bending moment, $M_{2,hog}$						Continuous Internal Span	▼	2946	kNm
Note $M_{2,hog} = k_{DL SDL,hog} \cdot \omega_{ULS1} \cdot L_2^2$ (if propped only) + $k_{DL SDL,hog} \cdot k_G \cdot [SDL_{slab} \cdot t_{w2} + SDL_{beam}] \cdot L_2^2$									
						$k_{DL SDL,hog}$	$k_{SDL,p,hog}$	$k_{LL,hog}$	
Continuous Internal Span						0.083	0.125	0.111	
Continuous End Span						0.105	0.161	0.120	
Simply-Supported						0.040	0.040	0.040	
Load All spans loaded (e.g. dead load) Live load (sequence of loaded spans to give max. bending moment)									
Beam ULS shear force, V_2								1395	kN
Note $V_2 = (0.5-0.6) \cdot \omega_{ULS1} \cdot L_2$ (if propped only) + $(0.5-0.6) \cdot \omega_{ULS2} \cdot L_2 + (0.5-0.6) \cdot (k_G \cdot SDL_{point})$									
Beam total ULS shear force, $V_1 + V_2$								2052	kN

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.	
						jXXX	6			
						Member/Location				
Job Title	Structure, Member Design - Precast Concrete Frame, Bea					Drg. Ref.				
Structure, Member Design - PC Frame, Beam and Slab						Made by	XX	Date	2/10/2026	Chd.
										BS8110
Structural Design [Stage 1: Precast Simply-Supported]										
Bending Design						PT	▼	PT		
Beam ULS sag bending moment, M _{1,sag}								1601	kNm	
Stress coeff., M _{1,sag} /b _{w1} d _{sag1} ²								3.0	N/mm ²	
Stress coeff., K _{sag1} = M _{1,sag} /b _{w1} d _{sag1} ² f _{cu} ≤ 0.156 [f _{cu} ≤60N/mm ²] 0.12						Include	▼	N/A		N/A
Lever arm, z _{sag1} = d _{sag1} ·[0.5+(0.25-K _{sag1} /0.9) ^{0.5}] ≤ 0.95d _{sag1}								N/A	mm	cl.3.4.4.4
Sag steel req., A _{s,sag1} = M _{1,sag} /((0.95f _y ·z _{sag1}), f _y ≤460N/mm ²								N/A	mm ²	cl.3.4.4.4
Sag steel prov., A _{s,sag,prov}								N/A	mm ²	
Sag steel prov. utilisation, A _{s,sag1} / A _{s,sag,prov}								N/A		N/A
Hog tensile stress limit utilisation, [σ _{t1,TLS}] / [σ _{t1,limit}]						1.13		37%		OK
	Tensile stress limit, σ _{t1,limit} = 0.45√f _{ci}							3.02	N/mm ²	cl.4.3.5.2
Sag prestress moment cap., φM _{u1,sag} = f _{pb} ·A _{ps} ·(d _{ps1} -0.45x)								2878	kNm	cl.4.3.7.3
Ratio, f _{pb} /0.95f _{pu} = f _{pb} /0.95f _{pu}								0.87		T.4.4
Ratio, x/d = x/d _{ps1} ≤ 0.50						Include	▼	0.41		OK
	Ratio, f _{pe} /f _{pu} = (P _{st} /A _{ps})/f _{pu} ≤ 0.60							0.60		
	Ratio, [f _{pu} A _{ps}]/[f _{cu} bd] = [f _{pu} A _{ps}]/[f _{cu} b _{w1} d _{ps1}]							0.20		cl.4.3.7.3
Sag prestress moment cap. utilisation, M _{1,sag} / φM _{u1,sag}								56%		OK

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						Member/Location					
Job Title		Structure, Member Design - Precast Concrete Frame, Bea				Drg. Ref.					
Structure, Member Design - PC Frame, Beam and Slab						Made by XX		Date 2/10/2026		Chd.	
											BS8110
Structural Design [Stage 2: Insitu]											
Bending Design						PT	▼	PT			
Beam ULS sag bending moment, M _{2,sag}								2322	kNm		
Stress coeff., M _{2,sag} /b _{w2} d _{sag2} ²								2.3	N/mm ²		
Stress coeff., M _{1,sag} /b _{w1} d _{sag1} ² + M _{2,sag} /b _{w2} d _{sag2} ²								5.3	N/mm ²		
Stress coeff., K _{sag2} = [M _{1,sag} +M _{2,sag}]/b _{w2} d _{sag2} ² f _{cu,top} ≤ 0.156 [f _{cu,top} ≤60N/mm ²] 0.								N/A			N/A
Lever arm, z _{sag2} = d _{sag2} ·[0.5+(0.25-K _{sag2} /0.9) ^{0.5}] ≤ 0.95d _{sag2}								N/A	mm		cl.3.4.4.4
Sag steel req., A _{s,sag2} = [M _{1,sag} +M _{2,sag}]/(0.95f _y ·z _{sag2}), f _y ≤460N/mm ²								N/A	mm ²		cl.3.4.4.4
Sag steel prov., A _{s,sag,prov}								N/A	mm ²		
Sag steel prov. utilisation, A _{s,sag2} / A _{s,sag,prov}								N/A			N/A
Sag tensile stress limit utilisation, [σ _{b1,SLS} +σ _{b2,SLS}] / [σ _{b2,limit}]						2.36		71%			OK
	Tensile stress limit, σ _{b2,limit} = 0.45√f _{cu}							3.34	N/mm ²		cl.4.3.4.3
Sag prestress moment cap., φM _{u2,sag} = f _{pb} ·A _{ps} ·(d _{ps2} -0.45x)								4142	kNm		cl.4.3.7.3
Ratio, f _{pb} /0.95f _{pu} = f _{pb} /0.95f _{pu}								0.87			T.4.4
Ratio, x/d = x/d _{ps2} ≤ 0.50								0.41			OK
	Ratio, f _{pe} /f _{pu} = (P _{it} /A _{ps})/f _{pu} ≤ 0.60							0.60			
	Ratio, [f _{pu} A _{ps}]/[f _{cu,top} bd] = [f _{pu} A _{ps}]/[f _{cu,top} b _{w2} d _{ps2}]							0.17			cl.4.3.7.3
Sag prestress moment cap. utilisation, [M _{1,sag} +M _{2,sag}] / φM _{u2,sag}								95%			OK
Beam ULS hog bending moment, M _{2,hog}								3129	kNm		
Stress coeff., M _{2,hog} /b _{w2} d _{hog2} ²								3.1	N/mm ²		
Stress coeff., K _{hog2} = M _{2,hog} /b _{w2} d _{hog2} ² f _{cu} ≤ 0.156 [f _{cu} ≤60] 0.120 [f _{cu} ≤75] 0.094								0.056			OK
Lever arm, z _{hog2} = d _{hog2} ·[0.5+(0.25-K _{hog2} /0.9) ^{0.5}] ≤ 0.95d _{hog2}								1019	mm		cl.3.4.4.4
Hog steel req., A _{s,hog2} = M _{2,hog} / (0.95f _y ·z _{hog2}), f _y ≤ 460N/mm ²								7026	mm ²		cl.3.4.4.4
Hog steel prov., A _{s,hog,prov}								7238	mm ²		
Hog steel prov. utilisation, A _{s,hog2} / A _{s,hog,prov}								97%			OK
Shear Design								RC			
Note Stage 2 shear design assumes top tension, irrespective of support conditions, check Stage 1 capacity otherwise thus check Stage 1 shear capacity if simply-supported at Stage 2;											
Beam ULS shear force, V ₂						Full Shear at Support ▼		1480	kN		
Ult. shear stress, v _{ult2} = V ₂ /b _{w2} d _{hog2}								1.59	N/mm ²		.3.4.5.2 BC
Ult. shear stress utilisation, [v _{ult1} +v _{ult2}] / (0.8f _{cu} ^{0.5} & {5.0,7.0}N/mm ²)								55%			OK
Design shear stress, v _{d2} = V ₂ /b _{w2} d _{hog2}								1.59	N/mm ²		
Enh. shear strength, 2d _{hog2} /a _v ·v _c (< 0.8f _{cu} ^{0.5} & {5.0,7.0}N/mm ²)						x 1.00		0.76	N/mm ²		cl.3.4.5.8
	Distance, a _v					2.00d	▼	2184	mm		Note
	ρ _w = 100A _{s,hog,prov} /b _{w2} d _{hog2}							0.78	%		cl.3.4.5.4
RC	v _c = (0.79/1.25)(ρ _w f _{cu} /25) ^{1/3} (400/d _{hog2}) ^{1/4} ; ρ _w <3; f _{cu} <80; (400/d _{hog2}) ¹							0.76	N/mm ²		cl.3.4.5.4
Minimum shear strength, v _r = MAX(0.4,0.4(f _{cu} /40) ^{2/3})N/mm ² , f _{cu} ≤80N/mm ²								0.49	N/mm ²		.3.4.5.3 BC
(A _{sv} /S) ₂ > b _{w2} MAX[v _r ,(v _{d2} -2d _{hog2} /a _v ·v _c)]/(0.95f _{yv}), f _{yv} ≤460						0.96	1.63	1.63	mm ² /mm		cl.3.4.5.8,
Tried A _{sv,prov} /S								4.83	mm ² /mm		
V _{cap2} = (A _{sv,prov} /S)·(0.95f _{yv})·d _{hog2} + 2d _{hog2} /a _v ·v _c ·(b _{w2} d _{hog2}), f _{yv} ≤460N/mm ²								3005	kN		cl.3.4.5.8,
Design shear links area utilisation, [(A _{sv} /S) ₁ + (A _{sv} /S) ₂] / (A _{sv,prov} /S)								51%			OK
Design shear resistance utilisation, [V ₁ +V ₂] / V _{cap2}								72%			OK
Deflection Design (Code Based)								RC			
Span, L ₂								N/A	m		
Span, L ₂ / eff. depth ratio, d _{sag2}								N/A			
Basic span / eff. depth ratio criteria = {26.0, Continuous Internal Span						▼		N/A			cl.3.4.6.3
Multiplier C _{1,rect} or flanged				N/A	Multiplier C _{1,L>10m}		Include ▼	N/A			cl.3.4.6.3, cl.3.4
Modification factor for tension C ₂ = 0.55+(477-f _s)/[120(0.9+M/bd ²)] ≤ 2.0								N/A			cl.3.4.6.5
	M/bd ² = [M _{1,sag} +M _{2,sag}]/b _{w2} d _{sag2} ²							N/A	N/mm ²		cl.3.4.6.5
	f _s = [2f _y ·A _{s,sag2} /3A _{s,sag,prov}]·[1/β _b]							N/A	N/mm ²		cl.3.4.6.5
Modified span / eff. depth ratio criteria = C ₁ ·C ₂ ·{26.0,23.0,20.0}								N/A			cl.3.4.6.3, cl.3.4
Deflection utilisation								N/A			N/A

CONSULTING ENGINEERS				Engineering Calculation Sheet Consulting Engineers				Job No.		Sheet No.				Rev.							
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								Member/Location													
Job Title		Structure, Member Design - Precast Concrete Frame, Beam								Drg. Ref.											
Structure, Member Design - PC Frame, Beam and Slab								Made by		XX		Date		2/10/2026		Chd.					
																IS456					
Structural Design [Stage 1: Precast Simply-Supported]																					
Bending Design								PT		▼		PT									
Beam ULS sag bending moment, $M_{1,sag}$												1716		kNm							
Stress coeff., $M_{1,sag}/b_{w1}d_{sag1}^2$												3.2		N/mm ²							
Stress coeff., $K_{sag1} = M_{1,sag}/b_{w1}d_{sag1}^2f_{cu} \leq 0.148 [f_y \leq 250N/mm^2] \mid 0.1$								Include		▼		N/A				N/A					
Lever arm, $z_{sag1} = d_{sag1} \cdot \{1.0 - 0.416[1.2 - ((1.2)^2 - 6.68K_{sag1})^{0.5}]\} \leq 0.95d_{sag1}$												N/A		mm		Note					
Sag steel req., $A_{s,sag1} = M_{1,sag}/(0.87f_y \cdot z_{sag1})$												N/A		mm ²							
Sag steel prov., $A_{s,sag,prov}$												N/A		mm ²							
Sag steel prov. utilisation, $A_{s,sag1} / A_{s,sag,prov}$												N/A				N/A					
Hog tensile stress limit utilisation, $[\sigma_{t1,TLS}] / [\sigma_{t1,limit}]$								1.13				38%				OK					
		Tensile stress limit, $\sigma_{t1,limit} = 3.00N/mm^2$										3.00		N/mm ²		24.2.1 IS13					
Sag prestress moment cap., $\phi M_{u1,sag} = \{F_{t,st,t}, F_{C,c}\} \cdot \{d_{ps1} - 0.42x\}$												2977		kNm		23.1.2 IS13					
		Depth of neutral axis, $x [x/d_{ps1} \leq 0.60]$								Include		▼		323		mm		OK			
Strand tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t} + \epsilon_{b,s,t} = [P_{st}/A_{ps}]/E_p + [(d_{ps1} - x)/x] \cdot \epsilon_{cu}$, $\epsilon_{cu} = 0.0035$												0.0107				cl.23.1.1 IS13					
Strand tensile stress, $\sigma_{t,s,t} = \epsilon_{t,s,t} \cdot E_p$ for $\epsilon_{t,s,t} \leq 1.0 \cdot f_{pu}/1.15/E_p$, else $1.0 \cdot f_{pu}/1.15$												1617		N/mm ²		.23.3 IS134					
Strand tension, $F_{t,s,t} = \sigma_{t,s,t} \cdot A_{ps} \mid$ Conc. comp., $F_{C,c} = 0.36f_{cu} \cdot b_{w1} \cdot x$								5434				5434		kN		23.1.1 IS13					
Sag prestress moment cap. utilisation, $M_{1,sag} / \phi M_{u1,sag}$												58%				OK					
Shear Design								PT		▼		PT									
p;																					
Eff. depth to sag [RC] or [PT], $d_{effsag1} = d_{sag1}$ or d_{ps1}												683		mm							
Beam ULS shear force, V_1				Full Shear at Support				▼				730		kN							
Ult. shear stress, $v_{ult1} = V_1/b_{w1}d_{effsag1}$												1.26		N/mm ²		T.20					
Ult. shear stress utilisation, $v_{ult1} / (0.62f_{cu}^{0.5} \text{ \& } 4.0N/mm^2)$												31%				OK					
Design shear stress, $v_{d1} = V_1/b_{w1}d_{effsag1}$												1.26		N/mm ²							
Enh. shear strength, $2d_{effsag1}/a_v \cdot v_c (< 0.62f_{cu}^{0.5} \text{ \& } 4.0N/mm^2)$								$\times 1.00$				0.74		N/mm ²		cl.40.5.1					
		Distance, a_v								2.00d		▼		1367		mm		Note			
		$\rho_w = 100 \{A_{s,sag,prov} [RC] \mid A_{ps} [PT]\} / b_{w1}d_{effsag1}$												0.58		%		cl.40.2.1			
RC		$v_c = 0.85k(0.8f_{cu})^{1/2}[(1+5\beta)^{1/2}-1]/6\beta$; $\beta = 0.8f_{cu}/(6.89\rho_w) \geq 1$; $\rho_w \leq 3$; f_{cu}												0.74		N/mm ²		cl.40.2.1			
PT		$v_c = (1-0.55f_{pe}/f_{pu})0.85(0.8f_{cu})^{1/2}[(1+5\beta)^{1/2}-1]/6\beta \geq 0.1\sqrt{f_{cu}}$; $\beta = 0.8f_{cu}/f_{cu}$																23.4.2 IS13			
Minimum shear strength, $v_r = 0.4N/mm^2$												0.40		N/mm ²		cl.26.5.1.6					
$(A_{sv}/S)_1 > b_{w1} \text{MAX}[v_r, (v_{d1} - 2d_{effsag1}/a_v \cdot v_c)]/(0.87f_{yv})$, $f_{yv} \leq 41$				0.94				1.21				1.21		mm ² /mm		40.4, cl.40.5					
Tried $A_{sv,prov}/S$												4.83		mm ² /mm							
$V_{cap1} = (A_{sv,prov}/S) \cdot (0.87f_{yv}) \cdot d_{effsag1} + 2d_{effsag1}/a_v \cdot v_c \cdot (b_{w1}d_{effsag1})$, $f_{yv} \leq 415N/mm^2$												1621		kN		40.4, cl.40.5					
Design shear links area utilisation, $(A_{sv}/S)_1 / (A_{sv,prov}/S)$												25%				OK					
Design shear resistance utilisation, V_1 / V_{cap1}												45%				OK					
												RC									
Deflection Design (Code Based)																					
Span, L_1												N/A		m							
Span, L_1 / eff. depth ratio, d_{sag1}												N/A									
Basic span / eff. depth ratio criteria = {20.0}												N/A				cl.23.2.1					
Multiplier $C_{1,rect}$ or flanged				N/A				Multiplier $C_{1,L>10m}$				Include		▼		N/A		cl.23.2.1			
Modification factor for tension $C_2 = 1/[0.225 + 0.003f_s + 0.625\log_{10}(\rho_w)] \leq 2.0$												N/A				cl.23.2.1					
		$\rho_w = 100A_{s,sag,prov}/b_{w1}d_{sag1}$										N/A		%		cl.23.2.1					
		$f_s = 0.58f_y \cdot [A_{s,sag1}/A_{s,sag,prov}]$										N/A		N/mm ²		cl.23.2.1					
Modified span / eff. depth ratio criteria = $C_1 \cdot C_2 \cdot \{20.0\}$												N/A				cl.23.2.1					
Deflection utilisation												N/A				N/A					

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Job Title		Structure, Member Design - Precast Concrete Frame, Bea				Drg. Ref.									
Structure, Member Design - PC Frame, Beam and Slab						Made by		XX		Date		2/10/2026		Chd.	
														IS456	
Structural Design [Stage 2: Insitu]															
Bending Design							PT		▼	PT					
Beam ULS sag bending moment, M _{2,sag}										2211	kNm				
Stress coeff., M _{2,sag} /b _{w2} d _{sag2} ²										2.2	N/mm ²				
Stress coeff., M _{1,sag} /b _{w1} d _{sag1} ² + M _{2,sag} /b _{w2} d _{sag2} ²										5.4	N/mm ²				
Stress coeff., K _{sag2} = [M _{1,sag} +M _{2,sag}]/b _{w2} d _{sag2} ² f _{cu,top} ≤ 0.148 [f _y ≤250N/mm ²] 0.13										N/A				N/A	
Lever arm, z _{sag2} = d _{sag2} ·{1.0-0.416[1.2-((1.2) ² -6.68K _{sag2}) ^{0.5}]} ≤ 0.95d _{sag2}										N/A	mm			Note	
Sag steel req., A _{s,sag2} = [M _{1,sag} +M _{2,sag}]/(0.87f _y ·z _{sag2})										N/A	mm ²				
Sag steel prov., A _{s,sag,prov}										N/A	mm ²				
Sag steel prov. utilisation, A _{s,sag2} / A _{s,sag,prov}										N/A				N/A	
Sag tensile stress limit utilisation, [σ _{b1,SLS} +σ _{b2,SLS}] / [σ _{b2,limit}]							2.36			79%				OK	
43	Tensile stress limit, σ _{b2,limit} = 3.00N/mm ²									3.00	N/mm ²			24.2.1 IS13	
Sag prestress moment cap., φM _{u2,sag} = {F _{t,s,tr} ·F _{c,c} }·{d _{ps2} -0.42x}										4443	kNm			23.1.2 IS13	
	Depth of neutral axis, x [x/d _{ps2} ≤ 0.60]									395	mm			OK	
Strand tensile strain, ε _{t,s,t} = ε _{p,s,t} +ε _{b,s,t} = [P _{lt} /A _{ps}]/E _p +[(d _{ps2} -x)/x]·ε _{cu} , ε _{cu} =0.0035										0.0113			cl.23.1.1 IS13		
Strand tensile stress, σ _{t,s,t} = ε _{t,s,t} ·E _p for ε _{t,s,t} ≤ 1.0.f _{pu} /1.15/E _p , else 1.0.f _{pu} /1.15										1617	N/mm ²			23.3 IS134	
Strand tension, F _{t,s,t} = σ _{t,s,t} ·A _{ps} Conc. comp., F _{c,c} = 0.36f _{cu,top} ·b _{w2} ·x							5434			5434	kN			23.1.1 IS13	
Sag prestress moment cap. utilisation, [M _{1,sag} +M _{2,sag}] / φM _{u2,sag}										88%				OK	
Beam ULS hog bending moment, M _{2,hog}										2996	kNm				
Stress coeff., M _{2,hog} /b _{w2} d _{hog2} ²										3.0	N/mm ²				
Stress coeff., K _{hog2} = M _{2,hog} /b _{w2} d _{hog2} ² f _{cu} ≤ 0.148 [f _y ≤250] 0.138 [f _y ≤415] 0.13										0.054				OK	
Lever arm, z _{hog2} = d _{hog2} ·{1.0-0.416[1.2-((1.2) ² -6.68K _{hog2}) ^{0.5}]} ≤ 0.95d _{hog2}										1019	mm			Note	
Hog steel req., A _{s,hog2} = M _{2,hog} / (0.87f _y ·z _{hog2})										6756	mm ²				
Hog steel prov., A _{s,hog,prov}										7238	mm ²				
Hog steel prov. utilisation, A _{s,hog2} / A _{s,hog,prov}										93%				OK	
Shear Design										RC					
Note Stage 2 shear design assumes top tension, irrespective of support conditions, check Stage 1 capacity otherwise															
Beam ULS shear force, V ₂				Full Shear at Support					▼	1425	kN				
Ult. shear stress, v _{ult2} = V ₂ /b _{w2} d _{hog2}										1.54	N/mm ²			T.20	
Ult. shear stress utilisation, [v _{ult1} +v _{ult2}] / (0.62f _{cu} ^{0.5} & 4.0N/mm ²)										70%				OK	
Design shear stress, v _{d2} = V ₂ /b _{w2} d _{hog2}										1.54	N/mm ²				
Enh. shear strength, 2d _{hog2} /a _v ·v _c (< 0.62f _{cu} ^{0.5} & 4.0N/mm ²)							x 1.00			0.61	N/mm ²			cl.40.5.1	
	Distance, a _v							2.00d	▼	2184	mm			Note	
	ρ _w = 100A _{s,hog,prov} /b _{w2} d _{hog2}									0.78	%			cl.40.2.1	
RC	v _c = 0.85k(0.8f _{cu}) ^{1/2} [(1+5β) ^{1/2} -1]/6β; β=0.8f _{cu} /(6.89ρ _w)≥1; ρ _w ≤3; f _{cu}									0.61	N/mm ²			cl.40.2.1	
43															
Minimum shear strength, v _r = 0.4N/mm ²										0.40	N/mm ²			cl.26.5.1.6	
(A _{sv} /S) ₂ > b _{w2} MAX[v _r , (v _{d2} -2d _{hog2} /a _v ·v _c)]/(0.87f _{yv}), f _{yv} ≤415							0.94	2.17		2.17	mm ² /mm			40.4, cl.40.5	
Tried A _{sv,prov} /S										4.83	mm ² /mm				
V _{cap2} = (A _{sv,prov} /S)·(0.87f _{yv})·d _{hog2} + 2d _{hog2} /a _v ·v _c ·(b _{w2} d _{hog2}), f _{yv} ≤415N/mm ²										2471	kN			40.4, cl.40.5	
Design shear links area utilisation, [(A _{sv} /S) ₁ + (A _{sv} /S) ₂] / (A _{sv,prov} /S)										70%				OK	
Design shear resistance utilisation, [V ₁ +V ₂] / V _{cap2}										87%				OK	
Deflection Design (Code Based)										RC					
Span, L ₂										N/A	m				
Span, L ₂ / eff. depth ratio, d _{sag2}										N/A					
Basic span / eff. depth ratio criteria = {26.0, Continuous Internal Span									▼	N/A				cl.23.2.1	
Multiplier C _{1,rect} or flanged					N/A	Multiplier C _{1,L>10m}		Include	▼	N/A				cl.23.2.1	
Modification factor for tension C ₂ = 1/[0.225+0.003f _s +0.625log ₁₀ (ρ _w)] ≤ 2.0										N/A				cl.23.2.1	
	ρ _w = 100A _{s,sag,prov} /b _{w2} d _{sag2}									N/A	%			cl.23.2.1	
	f _s = 0.58f _y ·[A _{s,sag2} /A _{s,sag,prov}]									N/A	N/mm ²			cl.23.2.1	
Modified span / eff. depth ratio criteria = C ₁ ·C ₂ ·{26.0,23.0,20.0}										N/A				cl.23.2.1	
Deflection utilisation										N/A				N/A	

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Structure, Member Design - PC Frame, Beam and Slab						Made by		XX		Date		2/10/2026		Chd.			
														EC2			
Structural Design [Stage 1: Precast Simply-Supported]																	
Bending Design							PT		▼	PT							
Beam ULS sag bending moment, $M_{1,sag}$										1544	kNm						
Stress coeff., $M_{1,sag}/b_{w1}d_{sag1}^2$										2.9	N/mm ²						
Stress coeff., $K_{sag1} = M_{1,sag}/b_{w1}d_{sag1}^2f_{ck} \leq 0.167$ [$f_{ck} \leq 50N/mm^2$]						Include		▼		N/A				N/A			
Lever arm, $z_{sag1} = d_{sag1} \cdot [0.5 + (0.25 - K_{sag1}/1.134)^{0.5}] \leq 0.95d_{sag1}$										N/A	mm			4.4.1 Mosle			
Sag steel req., $A_{s,sag1} = M_{1,sag}/(0.87f_y \cdot z_{sag1})$										N/A	mm ²						
Sag steel prov., $A_{s,sag,prov}$										N/A	mm ²						
Sag steel prov. utilisation, $A_{s,sag1} / A_{s,sag,prov}$										N/A				N/A			
Hog tensile stress limit utilisation, $[\sigma_{t1,TLS}] / [\sigma_{t1,limit}]$						1.13		35%						OK			
43		Tensile stress limit, $\sigma_{t1,limit} = 0.72f_{ctm} = 0.72\{f_{ci}' \leq 50:0.3f_c\}$				EC2		3.21		N/mm ²				5.8.2 TR.43			
Sag prestress moment cap., $\phi M_{u1,sag} = \{F_{t,s,t}/F_{c,c}\} \cdot \{d_{ps1} - \lambda \cdot x/2\}$										2790	kNm			3.1.6, cl.3.1			
		Depth of neutral axis, x [$x/d_{ps1} \leq 0.50$]				Include		▼		282		mm			OK		
Strand tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t} + \epsilon_{b,s,t} = [P_{st}/A_{ps}]/E_p + [(d_{ps1} - x)/x] \cdot \epsilon_{cu3}$						Include		▼		0.0118				OK			
Strand tensile stress, $\sigma_{t,s,t} = \epsilon_{t,s,t} \cdot E_p$ for $\epsilon_{t,s,t} \leq k \cdot f_{pu}/\gamma_s/E_p$, else $k \cdot f_{pu}/\gamma_s$										1456	N/mm ²			cl.3.3.6			
Strand tension, $F_{t,s,t} = \sigma_{t,s,t} \cdot A_{ps}$ Conc. comp., $F_{c,c} = \eta \cdot (\alpha_{cc} \cdot f_{ck}/\gamma_c) \cdot b_{w1} \cdot l$						4891		4891		kN				3.1.6, cl.3.1			
Sag prestress moment cap. utilisation, $M_{1,sag} / \phi M_{u1,sag}$										55%				OK			

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Structure, Member Design - PC Frame, Beam and Slab						Made by		XX		Date		2/10/2026		Chd.	
														EC2	
Structural Design [Stage 2: Insitu]															
Bending Design							PT		▼	PT					
Beam ULS sag bending moment, M _{2,sag}										2183	kNm				
Stress coeff., M _{2,sag} /b _{w2} d _{sag2} ²										2.2	N/mm ²				
Stress coeff., M _{1,sag} /b _{w1} d _{sag1} ² + M _{2,sag} /b _{w2} d _{sag2} ²										5.1	N/mm ²				
Stress coeff., K _{sag2} = [M _{1,sag} +M _{2,sag}]/b _{w2} d _{sag2} ² f _{ck,top} ≤ 0.167 [f _{ck,top} ≤50N/mm ²]										N/A				N/A	
Lever arm, z _{sag2} = d _{sag2} ·[0.5+(0.25-K _{sag2} /1.134) ^{0.5}] ≤ 0.95d _{sag2}										N/A	mm			4.4.1 Mosle	
Sag steel req., A _{s,sag2} = [M _{1,sag} +M _{2,sag}]/(0.87f _y ·z _{sag2})										N/A	mm ²				
Sag steel prov., A _{s,sag,prov}										N/A	mm ²				
Sag steel prov. utilisation, A _{s,sag2} / A _{s,sag,prov}										N/A				N/A	
Sag tensile stress limit utilisation, [σ _{b1,SLS} +σ _{b2,SLS}] / [σ _{b2,limit}]							2.36			62%				OK	
-2	Tensile stress limit, σ _{b2,limit} = 0.72f _{ctm} = 0.72{f _{ck} ≤50:0.3f _{ck} }						EC2			3.80	N/mm ²			5.8.2 TR.43	
Sag prestress moment cap., φM _{u2,sag} = {F _{t,s,tr} F _{c,c} }·{d _{ps2} -λ·x/2}										4100	kNm			3.1.6, cl.3.1	
	Depth of neutral axis, x [x/d _{ps2} ≤ 0.50]									363	mm			OK	
Strand tensile strain, ε _{t,s,t} = ε _{p,s,t} +ε _{b,s,t} = [P _{lt} /A _{ps}]/E _p +[(d _{ps2} -x)/x]·ε _{cu3} ≤ 0.02										0.0121				OK	
Strand tensile stress, σ _{t,s,t} = ε _{t,s,t} ·E _p for ε _{t,s,t} ≤ k.f _{pu} /γ _s /E _{p,r} else k.f _{pu} /γ _s										1456	N/mm ²			cl.3.3.6	
Strand tension, F _{t,s,t} = σ _{t,s,t} ·A _{ps} Conc. comp., F _{c,c} = η·(α _{cc} ·f _{ck,top} /γ _c)·b _v							4891			4891	kN			3.1.6, cl.3.1	
Sag prestress moment cap. utilisation, [M _{1,sag} +M _{2,sag}] / φM _{u2,sag}										91%				OK	
Beam ULS hog bending moment, M _{2,hog}										2946	kNm				
Stress coeff., M _{2,hog} /b _{w2} d _{hog2} ²										2.9	N/mm ²				
Stress coeff., K _{hog2} = M _{2,hog} /b _{w2} d _{hog2} ² f _{ck} ≤ 0.167 [f _{ck} ≤50]										0.065				OK	
Lever arm, z _{hog2} = d _{hog2} ·[0.5+(0.25-K _{hog2} /1.134) ^{0.5}] ≤ 0.95d _{hog2}										1026	mm			4.4.1 Mosle	
Hog steel req., A _{s,hog2} = M _{2,hog} / (0.87f _y ·z _{hog2})										6601	mm ²				
Hog steel prov., A _{s,hog,prov}										7238	mm ²				
Hog steel prov. utilisation, A _{s,hog2} / A _{s,hog,prov}										91%				OK	
Shear Design										RC					
Note Stage 2 shear design assumes top tension, irrespective of support conditions, check Stage 1 capacity otherwise															
Beam ULS shear force, V ₂				Full Shear at Support					▼	1395	kN				
Ult. shear stress, v _{ult2} = V ₂ /b _{w2} d _{hog2}										1.50	N/mm ²			3(3), cl.3.1	
Ult. shear stress utilisation, [v _{ult1} +v _{ult2}] / (v _{Rd,max} (θ=45°)=0.18(1-f _{ck} /250)·f _{ck,r} f _{ck} ≤50)										40%				OK	
Design shear stress, v _{d2} = V ₂ /b _{w2} d _{hog2}										1.50	N/mm ²				
Enh. shear strength, 2d _{hog2} /a _v ·v _{Rd,c} (< v _{Rd,max} (θ=45°)=0.18(1-f _{ck} /250)·f _{ck} x 1.00										0.56	N/mm ²			cl.6.2.3(8),	
	Distance, a _v							2.00d	▼	2184	mm			Note	
	ρ _w = 100A _{s,hog,prov} /b _{w2} d _{hog2}									0.78	%			cl.6.2.2(1)	
RC	v _{Rd,c} = C _{Rd,c} ·k(ρ _w f _{ck}) ^{1/3} +k ₁ ·σ _{cp} ≥ v _{min} +k ₁ ·σ _{cp} ; v _{min} =0.035k ^{3/2} f _{ck} ^{0.5} , k ₁ =0														
Minimum shear strength, v _r = 0.08/γ _s ·f _{ck} ^{0.5} N/mm ² , γ _s =1.15, f _{ck} ≤50N/mm ²											0.47	N/mm ²			2(5), cl.3.1
(A _{sv} /S) ₂ > b _{w2} MAX[v _r ,v _{d2} /(0.9.cotθ)]/(2d _{hog2} /a _v)]/(0.87f _{yv}),										0.91	1.32	1.32		mm ² /mm	cl.6.2.3(8),
Tried A _{sv,prov} /S										4.83	mm ² /mm				
V _{cap2} = 2d _{hog2} /a _v ·(A _{sv,prov} /S)·(0.87f _{yv})·0.9d _{hog2} ·cotθ, f _{yv} ≤600N/mm ²							22.0		5106	kN			cl.3.4.5.8,		
Design shear links area utilisation, [(A _{sv} /S) ₁ + (A _{sv} /S) ₂] / (A _{sv,prov} /S)									48%				OK		
Design shear resistance utilisation, [V ₁ +V ₂] / V _{cap2}									40%				OK		
Deflection Design (Code Based)												RC			
Span, L ₂										N/A	m				
Span, L ₂ / eff. depth ratio, d _{sag2}										N/A					
Basic span / eff. depth ratio criteria, {L/d _{eff} } _b Continuous Internal Span								▼	N/A				cl.7.4.2		
Multiplier F _{1,rect} or flanged				N/A		Multiplier F _{2,L>7m}		Include	▼	N/A				cl.7.4.2	
Modification factor for tension F ₃ = 310/σ _s ≤ 1.5										N/A				cl.7.4.2	
	ρ _{req,2} = 100A _{s,sag,2} /b _{w2} d _{sag2}									N/A	%			cl.7.4.2	
	σ _s = 310.(f _y /500)·[A _{s,sag2} /A _{s,sag,prov}]·[1/β _b]									N/A	N/mm ²			cl.7.4.2	
Modified span / eff. depth ratio criteria = F ₁ ·F ₂ ·F ₃ ·{L/d _{eff} } _{basic}										N/A					
Deflection utilisation										N/A				N/A	

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Structural Design [Stage 1: Precast Simply-Supported]																							
Bending Design												PT		▼		PT							
Beam ULS sag bending moment, $M_{1,sag}$														1373		kNm							
Stress coeff., $M_{1,sag}/\phi b_{w1}d_{sag1}^2$, $\phi=0.90$														2.9		N/mm ²		cl.21.2					
Stress coeff., $R_{n,sag1}/f_c' = M_{1,sag}/\phi b_{w1}d_{sag1}^2 f_c'$, $\phi=0.90$														N/A									
Net strain, $\epsilon_{tsag1} = 0.003(d_{tsag1}/c-1) \geq 0.005$, $c=a/\beta_1$, $a=A_{s,sag,prov}.f_y/(C$												Include ▼		N/A				N/A					
Sag steel req., $A_{s,sag1} = 0.85f_c'/f_y.[1-\sqrt{(1-2R_{n,sag1}/((0.85f_c')))}].b_{w1}d_{sag1}$, $f_y \leq 550$ N/mm														N/A		mm ²		, 22.2.2.4.3					
Sag steel prov., $A_{s,sag,prov}$														N/A		mm ²							
Sag steel prov. utilisation, $A_{s,sag1} / A_{s,sag,prov}$														N/A				N/A					
Hog tensile stress limit utilisation, $[\sigma_{t1,TLS}] / [\sigma_{t1,limit}]$												1.13		38%				OK					
-2		Tensile stress limit, $\sigma_{t1,limit} = 0.50\sqrt{f_{ci}'}$												2.96		N/mm ²		cl.24.5.3.2					
Sag prestress moment cap., $\phi M_{u1,sag} = \phi.\{F_{t,s,t}r.F_{C,c}\}.\{d_{ps1}-\beta_1.c/2\}$, $\phi=0$												0.90		3217		kNm		cl.3.1.1, cl.2					
		Depth of neutral axis, c												256		mm							
Strand tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t}+\epsilon_{b,s,t} = [P_{st}/A_{ps}]/E_p+[(d_{ps1}-c)/c].\epsilon_{cu}$, $\epsilon_{cu}=0.003$														0.0118				cl.22.2.2.1					
Strand tensile stress, $\sigma_{t,s,t} = \epsilon_{t,s,t}.E_p$ for $\epsilon_{t,s,t} \leq 0.0086$, else $f_{pu}-0.276/(\epsilon_{t,s,t}-0.007)$														1803		N/mm ²		PCI					
Strand tension, $F_{t,s,t} = \sigma_{t,s,t}.A_{ps}$ Conc. comp., $F_{C,c} = \beta_1.0.85f_c'.b_{w1}.c$												6057		6057		kN		cl.22.2.2.4.1					
Sag prestress moment cap. utilisation, $M_{1,sag} / \phi M_{u1,sag}$														43%				OK					
Shear Design												PT		▼		PT							
P;																							
Eff. depth to sag [RC] or [PT], $d_{effsag1} = d_{sag1}$ or d_{ps1}														683		mm							
Beam ULS shear force, V_1						Full Shear at Support ▼						584		kN									
Ult. shear stress, $v_{ult1} = V_1/b_{w1}d_{effsag1}$														1.01		N/mm ²		cl.22.5.5.1,					
Ult. shear stress utilisation, $v_{ult1} / (\phi(0.66+0.17\lambda)\sqrt{f_c'}$, $\phi=0.75$, $f_c' \leq 70$ N/mm ²)														24%				OK					
Design. shear stress, $v_{d1} = V_1/b_{w1}d_{effsag1}$														1.01		N/mm ²							
Enh. shear strength, $d_{effsag1}/a_v.v_c$ ($< \phi(0.66+0.17\lambda)\sqrt{f_c'}$, $\phi=0.75$, $f_c' \leq 70$ x 1.00														0.70		N/mm ²		cl.9.4.3.2					
		Distance, a_v										2.00d ▼		683		mm		Note					
RC		$v_c = \phi 0.17\lambda\sqrt{f_c'}$; $\phi=0.75$; $f_c' \leq 70$ N/mm ²												0.70		N/mm ²		5.5.1, cl.22					
PT		$v_c = \phi 0.14\lambda\sqrt{f_c'}$; $\phi=0.75$; $f_c' \leq 70$ N/mm ²																cl.22.5.8.3.1					
Minimum shear strength, $v_r = \text{MAX}(0.35, 0.062\sqrt{f_c'})$ N/mm ² , $f_c' \leq 70$ N/mm ²														0.42		N/mm ²		6.2.2, cl.22					
$(A_{sv}/S)_1 > b_{w1}\text{MAX}[v_r, (v_{d1}-d_{effsag1}/a_v.v_c)/\phi/(d/a_v)]/f_{yv}$, $\phi=0.$												0.84 0.81		0.84		mm ² /mm		5, cl.9.4.3.2					
Tried $A_{sv,prov}/S$												4.83		mm ² /mm									
$V_{cap1} = \phi d_{effsag1}/a_v.(A_{sv,prov}/S).f_{yv}.d_{effsag1} + d_{effsag1}/a_v.v_c.(b_{w1}d_{effsag1})$, $\phi=0.75$, $f_{yv} \leq 42$														1448		kN		5.10.5.3, cl.					
Design shear links area utilisation, $(A_{sv}/S)_1 / (A_{sv,prov}/S)$														17%				OK					
Design shear resistance utilisation, V_1 / V_{cap1}														40%				OK					
Deflection Design (Code Based)														RC									
Span, L_1												N/A		m									
Span, L_1 / eff. depth ratio, d_{sag1}												N/A											
Basic span / eff. depth ratio criteria = {16.0}												N/A				cl.9.3.1.1							
Multiplier $F_{1,rect}$ or flanged												N/A											
Multiplier $F_{2,L>10m}$												N/A											
Modification factor for tension $F_3 = 0.4+f_y/700$												N/A				cl.9.3.1.1.1							
Modified span / eff. depth ratio criteria = $F_1.F_2.F_3.\{L/d_{eff}\}_{basic}$														N/A									
Deflection utilisation												N/A				N/A							

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														ACI318		
Structural Design [Stage 2: Insitu]																
Bending Design							PT		▼	PT						
Beam ULS sag bending moment, $M_{2,sag}$											2285	kNm				
Stress coeff., $M_{2,sag}/\phi b_{w2} d_{sag2}^2$, $\phi=0.90$											2.5	N/mm ²		cl.21.2		
Stress coeff., $M_{1,sag}/\phi b_{w1} d_{sag1}^2 + M_{2,sag}/\phi b_{w2} d_{sag2}^2$, $\phi=0.90$											5.4	N/mm ²		cl.21.2		
Stress coeff., $R_{n,sag2}/f_{c,top}' = [M_{1,sag}+M_{2,sag}]/\phi b_{w2} d_{sag2}^2 f_{c,top}'$, $\phi=0.90$											N/A			cl.21.2		
Net strain, $\epsilon_{tsag2} = 0.003(d_{tsag2}/c-1) \geq 0.005$, $c=a/\beta_1$, $a=A_{s,sag,prov} \cdot f_y/(0.85f_{c,top}' b_{w2})$											N/A			N/A		
Sag steel req., $A_{s,sag2} = 0.85f_{c,top}'/f_y \cdot [1-\sqrt{(1-2R_{n,sag2}/(0.85f_{c,top}'))}] \cdot b_{w2} d_{sag2}$, $f_y \leq 550$ N/mm ²											N/A	mm ²		, 22.2.2.4.3		
Sag steel prov., $A_{s,sag,prov}$											N/A	mm ²				
Sag steel prov. utilisation, $A_{s,sag2} / A_{s,sag,prov}$											N/A			N/A		
Sag tensile stress limit utilisation, $[\sigma_{b1,SLS}+\sigma_{b2,SLS}] / [\sigma_{b2,limit}]$								2.36			57%			OK		
		Tensile stress limit, $\sigma_{b2,limit} = 0.62\sqrt{f_c'}$									4.16	N/mm ²		cl.24.5.2.1		
Sag prestress moment cap., $\phi M_{u2,sag} = \phi \cdot \{F_{t,s,t} F_{c,c}\} \cdot \{d_{ps2}-\beta_1 \cdot c/2\}$, $\phi=0.75$								0.90			4731	kNm		, 3.1.1, cl.2.2.2.4.3		
		Depth of neutral axis, c									301	mm				
Strand tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t}+\epsilon_{b,s,t} = [P_{lt}/A_{ps}]/E_p+[(d_{ps2}-c)/c] \cdot \epsilon_{cu}$, $\epsilon_{cu}=0.003$											0.0129			cl.22.2.2.1		
Strand tensile stress, $\sigma_{t,s,t} = \epsilon_{t,s,t} \cdot E_p$ for $\epsilon_{t,s,t} \leq 0.0086$, else $f_{pu}-0.276/(\epsilon_{t,s,t}-0.007)$											1813	N/mm ²		PCI		
Strand tension, $F_{t,s,t} = \sigma_{t,s,t} \cdot A_{ps}$ Conc. comp., $F_{c,c} = \beta_1 \cdot 0.85f_{c,top}' \cdot b_{w2} \cdot c$								6092			6092	kN		cl.22.2.2.4.1		
Sag prestress moment cap. utilisation, $[M_{1,sag}+M_{2,sag}] / \phi M_{u2,sag}$											77%			OK		
Beam ULS hog bending moment, $M_{2,hog}$											3062	kNm				
Stress coeff., $M_{2,hog}/\phi b_{w2} d_{hog2}^2$, $\phi=0.90$											3.4	N/mm ²		cl.21.2		
Stress coeff., $R_{n,hog2}/f_c' = M_{2,hog}/\phi b_{w2} d_{hog2}^2 f_c'$, $\phi=0.90$											0.075			cl.21.2		
Net strain, $\epsilon_{thog2} = 0.003(d_{thog2}/c-1) \geq 0.005$, $c=a/\beta_1$, $a=A_{s,hog,prov} \cdot f_y/(0.85f_{c,top}' b_{w2})$											0.019			OK		
Hog steel req., $A_{s,hog2} = 0.85f_c'/f_y \cdot [1-\sqrt{(1-2R_{n,hog2}/(0.85f_c'))}] \cdot b_{w2} d_{hog2}$, $f_y \leq 550$ N/mm ²											6532	mm ²		, 22.2.2.4.3		
Hog steel prov., $A_{s,hog,prov}$											7238	mm ²				
Hog steel prov. utilisation, $A_{s,hog2} / A_{s,hog,prov}$											90%			OK		
Shear Design											RC					
Note Stage 2 shear design assumes top tension, irrespective of support conditions, check Stage 1 capacity otherwise																
Beam ULS shear force, V_2				Full Shear at Support						▼	1440	kN				
Ult. shear stress, $v_{ult2} = V_2/b_{w2} d_{hog2}$											1.55	N/mm ²		cl.22.5.5.1		
Ult. shear stress utilisation, $[v_{ult1}+v_{ult2}] / (\phi(0.66+0.17\lambda)\sqrt{f_c'})$, $\phi=0.75$, $f_c' \leq 70$ N/mm ²											61%			OK		
Design shear stress, $v_{d2} = V_2/b_{w2} d_{hog2}$											1.55	N/mm ²				
Enh. shear strength, $d_{hog2}/a_v \cdot v_c (< \phi(0.66+0.17\lambda)\sqrt{f_c'})$, $\phi=0.75$, $f_c' \leq 70$ N/mm ² x 1.00											0.86	N/mm ²		cl.9.4.3.2		
		Distance, a_v							2.00d		▼	1092	mm		Note	
RC		$v_c = \phi 0.17\lambda\sqrt{f_c'}$; $\phi=0.75$; $f_c' \leq 70$ N/mm ²									0.86	N/mm ²		5.5.1, cl.22.5.5.1		
Minimum shear strength, $v_r = \text{MAX}(0.35,0.062\sqrt{f_c'})$ N/mm ² , $f_c' \leq 70$ N/mm ²											0.42	N/mm ²		6.2.2, cl.22.5.5.1		
$(A_{sv}/S)_2 > b_{w2} \text{MAX}[v_r,(v_{d2}-d_{hog2}/a_v \cdot v_c)/\phi]/f_{yv}$, $\phi=0.75$							0.84	1.88			1.88	mm ² /mm		5, cl.9.4.3.2		
Tried $A_{sv,prov}/S$											4.83	mm ² /mm				
$V_{cap2} = \phi d_{hog2}/a_v \cdot (A_{sv,prov}/S) \cdot f_{yv} \cdot d_{hog2} + d_{hog2}/a_v \cdot v_c \cdot (b_{w2} d_{hog2})$, $\phi=0.75$, $f_{yv} \leq 420$ N/mm ²											2454	kN		5.10.5.3, cl.9.4.3.2		
Design shear links area utilisation, $[(A_{sv}/S)_1 + (A_{sv}/S)_2] / (A_{sv,prov}/S)$											56%			OK		
Design shear resistance utilisation, $[V_1+V_2] / V_{cap2}$											82%			OK		
Deflection Design (Code Based)											RC					
Span, L_2											N/A	m				
Span, L_2 / eff. depth ratio, d_{sag2}											N/A					
Basic span / eff. depth ratio criteria = {21.0, Continuous Internal Span										▼	N/A			cl.9.3.1.1		
Multiplier $F_{1,rect}$ or flanged											N/A					
Multiplier $F_{2,L>10m}$											N/A					
Modification factor for tension $F_3 = 0.4+f_y/700$											N/A			cl.9.3.1.1.1		
Modified span / eff. depth ratio criteria = $F_1 \cdot F_2 \cdot F_3 \cdot \{L/d_{eff}\}_{basic}$											N/A					
Deflection utilisation											N/A			N/A		

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Structure, Member Design - PC Frame, Beam and Slab						Made by	XX	Date	2/10/2026	Chd.
										AS3600
Structural Design [Stage 1: Precast Simply-Supported]										
Bending Design						PT		▼	PT	
Beam ULS sag bending moment, $M_{1,sag}$								1373	kNm	
Stress coeff., $M_{1,sag}/\phi b_{w1} d_{sag1}^2$, $\phi=0.85$								3.0	N/mm ²	cl.2.2.2
Stress coeff., $R_{n,sag1}/f_c' = M_{1,sag}/\phi b_{w1} d_{sag1}^2 f_c'$, $\phi=0.85$								N/A		cl.2.2.2
Neutral axis param., $k_u = a/(\gamma d_{sag1}) \leq 0.36$, $a=(A_{s,sag,prov} \cdot f_y)/(\alpha_2 \cdot f_c' \cdot b_{w1})$						Include	▼	N/A		N/A
Sag steel req., $A_{s,sag1} = \alpha_2 \cdot f_c'/f_y \cdot [1-\sqrt{(1-2R_{n,sag1}/(\alpha_2 \cdot f_c'))}] \cdot b_{w1} d_{sag1}$, $\alpha_2=0.85-0.0015$								N/A	mm ²	cl.8.1.3
Sag steel prov., $A_{s,sag,prov}$								N/A	mm ²	
Sag steel prov. utilisation, $A_{s,sag1} / A_{s,sag,prov}$								N/A		N/A
Hog tensile stress limit utilisation, $[\sigma_{t1, TLS}] / [\sigma_{t1, limit}]$						1.13	32%			OK
	Tensile stress limit, $\sigma_{t1, limit} = 0.60 \sqrt{f_{ci}'}$							3.55	N/mm ²	cl.8.1.6.2
Sag prestress moment cap., $\phi M_{u1, sag} = \phi \cdot \{F_{t, s, t} \cdot F_{c, c}\} \cdot \{d_{ps1} - \gamma \cdot c/2\}$, $\phi=0.$						0.85	3006	kNm		cl.8.1.3
	Depth of neutral axis, c							237	mm	
Strand tensile strain, $\epsilon_{t, s, t} = \epsilon_{p, s, t} + \epsilon_{b, s, t} = [P_{st}/A_{ps}]/E_p + [(d_{ps1} - c)/c] \cdot \epsilon_{cu}$, $\epsilon_{cu}=0.003$								0.0125		cl.8.1.3
Strand tensile stress, $\sigma_{t, s, t} = \epsilon_{t, s, t} \cdot E_p$ for $\epsilon_{t, s, t} \leq 0.0086$, else $f_{pu} - 0.276/(\epsilon_{t, s, t} - 0.007)$								1809	N/mm ²	PCI
Strand tension, $F_{t, s, t} = \sigma_{t, s, t} \cdot A_{ps}$ Conc. comp., $F_{c, c} = \alpha_2 \cdot f_c' \cdot b_{w1} \cdot (\gamma \cdot c)$						6079	6079	kN		cl.8.1.3
Sag prestress moment cap. utilisation, $M_{1, sag} / \phi M_{u1, sag}$								46%		OK

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Job Title		Structure, Member Design - Precast Concrete Frame, Bea				Drg. Ref.									
Structure, Member Design - PC Frame, Beam and Slab						Made by		XX		Date		2/10/2026		Chd.	
														AS3600	
Structural Design [Stage 2: Insitu]															
Bending Design							PT		▼	PT					
Beam ULS sag bending moment, $M_{2,sag}$										2156	kNm				
Stress coeff., $M_{2,sag}/\phi b_{w2}d_{sag2}^2$, $\phi=0.85$										2.5	N/mm ²			cl.2.2.2	
Stress coeff., $M_{1,sag}/\phi b_{w1}d_{sag1}^2 + M_{2,sag}/\phi b_{w2}d_{sag2}^2$, $\phi=0.85$										5.6	N/mm ²			cl.2.2.2	
Stress coeff., $R_{n,sag2}/f_{c,top}' = [M_{1,sag}+M_{2,sag}]/\phi b_{w2}d_{sag2}^2 f_{c,top}'$, $\phi=0.85$										N/A				cl.2.2.2	
Neutral axis param., $k_u = a/(\gamma d_{sag2}) \leq 0.36$, $a=(A_{s,sag,prov} \cdot f_y)/(\alpha_2 \cdot f_{c,top}' \cdot b_{w2})$, $\gamma=0.97-0.0015$										N/A				N/A	
Sag steel req., $A_{s,sag2} = \alpha_2 \cdot f_{c,top}'/f_y \cdot [1-\sqrt{(1-2R_{n,sag2}/(\alpha_2 \cdot f_{c,top}'))}] \cdot b_{w2}d_{sag2}$, $\alpha_2=0.85-0.0015$										N/A	mm ²			cl.8.1.3	
Sag steel prov., $A_{s,sag,prov}$										N/A	mm ²				
Sag steel prov. utilisation, $A_{s,sag2} / A_{s,sag,prov}$										N/A				N/A	
Sag tensile stress limit utilisation, $[\sigma_{b1,SLS}+\sigma_{b2,SLS}] / [\sigma_{b2,limit}]$							2.36			59%				OK	
	Tensile stress limit, $\sigma_{b2,limit} = 0.60\sqrt{f_c'}$									4.02	N/mm ²			cl.8.6.3	
Sag prestress moment cap., $\phi M_{u2,sag} = \phi \cdot \{F_{t,s,t} \cdot F_{c,c}\} \cdot \{d_{ps2}-\gamma \cdot c/2\}$, $\phi=0.9$							0.85			4432	kNm			cl.8.1.3	
	Depth of neutral axis, c									291	mm				
Strand tensile strain, $\epsilon_{t,s,t} = \epsilon_{p,s,t}+\epsilon_{b,s,t} = [P_{lt}/A_{ps}]/E_p+[(d_{ps2}-c)/c] \cdot \epsilon_{cu}$, $\epsilon_{cu}=0.003$										0.0132				cl.8.1.3	
Strand tensile stress, $\sigma_{t,s,t} = \epsilon_{t,s,t} \cdot E_p$ for $\epsilon_{t,s,t} \leq 0.0086$, else $f_{pu}-0.276/(\epsilon_{t,s,t}-0.007)$										1816	N/mm ²			PCI	
Strand tension, $F_{t,s,t} = \sigma_{t,s,t} \cdot A_{ps}$ Conc. comp., $F_{c,c} = \alpha_2 \cdot f_{c,top}' \cdot b_{w2} \cdot (\gamma \cdot c)$							6100			6100	kN			cl.8.1.3	
Sag prestress moment cap. utilisation, $[M_{1,sag}+M_{2,sag}] / \phi M_{u2,sag}$										80%				OK	
Beam ULS hog bending moment, $M_{2,hog}$										2896	kNm				
Stress coeff., $M_{2,hog}/\phi b_{w2}d_{hog2}^2$, $\phi=0.85$										3.4	N/mm ²			cl.2.2.2	
Stress coeff., $R_{n,hog2}/f_c' = M_{2,hog}/\phi b_{w2}d_{hog2}^2 f_c'$, $\phi=0.90$										0.075				cl.2.2.2	
Neutral axis param., $k_u = a/(\gamma d_{hog2}) \leq 0.36$, $a=(A_{s,hog,prov} \cdot f_y)/(\alpha_2 \cdot f_c' \cdot b_{w2})$, $\gamma=0.97-0.0015$										0.13				OK	
Hog steel req., $A_{s,hog2} = \alpha_2 \cdot f_c'/f_y \cdot [1-\sqrt{(1-2R_{n,hog2}/(\alpha_2 \cdot f_c'))}] \cdot b_{w2}d_{hog2}$, $\alpha_2=0.85-0.0015$										6570	mm ²			cl.8.1.3	
Hog steel prov., $A_{s,hog,prov}$										7238	mm ²				
Hog steel prov. utilisation, $A_{s,hog2} / A_{s,hog,prov}$										91%				OK	
Shear Design										RC					
Note Stage 2 shear design assumes top tension, irrespective of support conditions, check Stage 1 capacity otherwise															
Eff. depth to hog, $d_{effhog2} = \text{MAX} \{0.72h_2, 0.9d_{hog2}\}$										983	mm			cl.8.2.1.9	
Beam ULS shear force, V_2				Full Shear at Support					▼	1365	kN				
Ult. shear stress, $v_{ult2} = V_2/b_{w2}d_{effhog2}$										1.63	N/mm ²			cl.8.2.3.3	
Ult. shear stress utilisation, $[v_{ult1}+v_{ult2}] / (\phi 0.55[0.9f_c'(\cot\theta/(1+\cot^2\theta))])$, $\phi=0.75$,										33%				OK	
Design shear stress, $v_{d2} = V_2/b_{w2}d_{effhog2}$										1.63	N/mm ²				
Enh. shear strength, $d_{effhog2}/a_v \cdot v_c (< (\phi 0.55[0.9f_c'(\cot\theta/(1+\cot2\theta))])$, $\phi=0.75$									$\times 1.00$	0.57	N/mm ²			cl.8.2.3.2	
	Distance, a_v							2.00d	▼	983	mm			Note	
	E.A = $E_s \cdot A_{s,hog,prov}$									1448	MN			cl.8.2.4.2.2	
RC	$v_c = \phi k_v \cdot \sqrt{f_c'}$; $\phi=0.75$; $k_v=[0.4/(1+1500\epsilon_x)]$; $f_c' \leq 65\text{N/mm}^2$									0.57	N/mm ²			cl.8.2.4.2.1	
	$\epsilon_x = 0.000 \leq [\text{MAX}(M=0, (V_1+V_2) \cdot d_{effhog2})/d_{effhog2} + V_1+V_2]/[2(E.A)]$									0.00169				cl.8.2.4.2.2	
Minimum shear strength, $v_r = 0.08\sqrt{f_c'}$ N/mm ²										0.54	N/mm ²			cl.8.2.1.7	
$(A_{sv}/S)_1 > b_{w1} \text{MAX}[v_r, (v_{d1}-d_{effhog2}/a_v \cdot v_c)/\cot\theta/\phi/(d/a_v)]/f_{yv}$,							0.91		2.09	2.09	mm ² /mm			cl.8.2.3.2,	
Tried $A_{sv,prov}/S$										4.83	mm ² /mm				
$V_{cap2} = \phi d_{effhog2}/a_v \cdot (A_{sv,prov}/S) \cdot f_{yv} \cdot d_{effhog2} \cdot \cot\theta + d_{effhog2}/a_v \cdot v_c \cdot (b_{w2}d_{effhog2})$							40.8			2533	kN			cl.8.2.3.2,	
Design shear links area utilisation, $[(A_{sv}/S)_1 + (A_{sv}/S)_2] / (A_{sv,prov}/S)$										43%				OK	
Design shear resistance utilisation, $[V_1+V_2] / V_{cap2}$										77%				OK	
Deflection Design (Code Based)										RC					
Modified span / eff. depth ratio criteria = $[k_1 \cdot (\Delta/L_{ef}) \cdot b_{eff2} \cdot E_c/(k_2 \cdot (1/C_{MF}) \cdot (\omega_{SLS1}+\omega_{SLS2}))]^{1/3}$															
Span, L_2										N/A	m				
Span, L_2 / eff. depth ratio, d_{sag2}										N/A					
Basic span / eff. depth ratio criteria, $\Delta/L_{ef} = \{ \text{Continuous Internal Span} \}$									▼	N/A				cl.2.3.2(a)	
			$k_1 = I_{ef}/b_{eff2}d_{sa}^3$, taken as— $(5-0.04f_c')p+0.002 \leq 0.1/\beta^{2/3}$ when $p \geq 0.001(f_c')^{1/3}/\beta^{2/3}$ $0.055(f_c')^{1/3}/\beta^{2/3}-50p \leq 0.06/\beta^{2/3}$ when $p < 0.001(f_c')^{1/3}/\beta^{2/3}$												
Multiplier $k_1 = I_{ef}/b_{eff2}d_{sa}^3$										N/A				cl.8.5.4	
Multiplier $k_2 = \{1.5/384, 2.4/384, 5/384\}$										N/A				cl.8.5.4	
Modified span / eff. depth ratio criteria = $[k_1 \cdot (\Delta/L_{ef}) \cdot b_{eff2} \cdot E_c/(k_2 \cdot (1/C_{MF}) \cdot (\omega_{SLS1}+\omega_{SLS2}))]^{1/3}$										N/A				cl.8.5.4	
Deflection utilisation										N/A				N/A	

[illegible]

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Job Title	Structure, Member Design - Precast Concrete Frame, Bea					Drg. Ref.																																					
Structure, Member Design - PC Frame, Beam and Slab						Made by	XX	Date	2/10/2026																																		
						Chd.																																					
						BS5400-4																																					
Longitudinal Shear Between Web and Flange Rectangular or Flanged Beam (BS5400-4)																																											
Note that this check is performed for both rectangular and flanged section designs, although theoretically only applicable in the latter case;																																											
Longitudinal shear force per unit length, $V_1 = K_S \cdot \Delta F_d / \Delta x$						627		kN/m																																			
Change of normal force in flange half over Δx , $\Delta F_d = K_B \cdot (M^* - 0) / z$						1178		kN																																			
Note $M^* = \text{MAX} \{M_{2,sag}, M_{2,hog}\}$;						Hogging		3129 kNm																																			
Note conservatively factor, $K_B = 0.5(b_{eff} - b_w) / b_{eff}$ employed even if neutral axis within web;																																											
Lever arm, $z = \{z_{sag2}, z_{hog2}\}$						Hogging		1.019 m																																			
Thickness of the flange at the junctions, $h_f = \text{insitu}_{slab}$								100 mm																																			
Length under consideration, Δx								2500 mm																																			
Note Δx is the beam length between the point of maximum design moment and the point of zero moment;																																											
Shear stress distribution factor, K_S								1.33																																			
The longitudinal shear should be calculated per unit length. For UDLs, K_S may be taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;									cl.7.4.2.3																																		
Effective width, $b_{eff} = b_{eff2}$								3650 mm																																			
Note for rectangular sections, b_{eff} equivalent to that of T-sections assumed;																																											
Width (rectangular) or web width (flanged), $b_w = b_{w2}$								850 mm																																			
Longitudinal shear force limit per unit length, $V_{1,limit}$						586		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)	675	kN/m	cl.7.4.2.3																																		
						(b)	586	kN/m	cl.7.4.2.3																																		
Table 31 — Ultimate longitudinal shear stress, v_1, and values of k_1 for composite members <table border="1"> <thead> <tr> <th rowspan="2">Type of shear plane</th> <th colspan="4">Longitudinal shear stress for concrete grade</th> <th rowspan="2">k_1</th> </tr> <tr> <th>20</th> <th>25</th> <th>30</th> <th>40 or more</th> </tr> <tr> <th></th> <th>N/mm²</th> <th>N/mm²</th> <th>N/mm²</th> <th>N/mm²</th> <th></th> </tr> </thead> <tbody> <tr> <td>Monolithic construction</td> <td>0.90</td> <td>0.90</td> <td>1.25</td> <td>1.25</td> <td>0.15</td> </tr> <tr> <td>Surface type 1</td> <td>0.50</td> <td>0.63</td> <td>0.75</td> <td>0.80</td> <td>0.15</td> </tr> <tr> <td>Surface type 2</td> <td>0.30</td> <td>0.38</td> <td>0.45</td> <td>0.50</td> <td>0.09</td> </tr> </tbody> </table> NOTE For construction with lightweight aggregate concrete, the values given in this table should be reduced by 25 %.										Type of shear plane	Longitudinal shear stress for concrete grade				k_1	20	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
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																																						
Concrete bond constant, k_1						0.15		T.31																																			
Ultimate longitudinal shear stress limit, v_1						0.80		N/mm ² T.31																																			
Surface type						Surface type 1 (brushed / washed off laitance)		▼ T.31																																			
Length of shear plane, $L_s = h_f = \text{insitu}_{slab}$						100		mm																																			
Provided transverse reinforcement per unit length, A_e						1571		mm ² /m																																			
Note reinforcement provided for coexistent bending effects and shear reinforcement crossing the shear plane, provided to resist vertical shear, may be included provided they are fully anchored;									cl.7.4.2.3																																		
Characteristic strength of reinforcement, $f_y \leq 460 \text{ N/mm}^2$						460		N/mm ²																																			
Longitudinal shear force limit per unit length utilisation, $V_1 / V_{1,limit}$						107%		NOT OK																																			
Required nominal transverse reinforcement per unit length, $0.15\% L_s$						150		mm ² /m cl.7.4.2.3																																			
Required nominal transverse reinforcement per unit length utilisation, $0.15\% L_s / A_e$						10%		OK																																			

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers		Job No.	Sheet No.	Rev.
				jXXX	20	
				Member/Location		
Job Title	Structure, Member Design - Precast Concrete Frame, Beam			Drg. Ref.		
Structure, Member Design - PC Frame, Beam and Slab				Made by	XX	Date 2/10/2026
				Chd. EC2		
Longitudinal Shear Within Web Rectangular or Flanged Beam (EC2)						
Longitudinal shear stress,		$V_{Edi} = \beta V_{Ed} / (z b_i)$		1.18 N/mm ²		cl.6.2.5
Ratio, $\beta = 1.0$				1.0		cl.6.2.5
Transverse shear force (averaged), $V_{Ed} = V_d^*/2 = (V_1 + V_2)/2$				1026 kN		cl.6.2.5
Lever arm, $z = z_{hog2}$				1.026 m		cl.6.2.5
Width of the interface, $b_i = b_w = b_{w2}$				850 mm		cl.6.2.5
Longitudinal shear stress limit, v_{Rdi}				2.33 N/mm ²		
		$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) \leq 0.5 v f_{cd}$				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 $\mu = 0.5$ Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: $c = 0.20$ and $\mu = 0.6$ Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: $c = 0.40$ and $\mu = 0.7$ (EC1) Indented: a surface with indentations complying with Figure 6.9: $c = 0.50$ and $\mu = 0.9$						
Design tensile strength, f_{ctd}				1.50 N/mm ²		
		$f_{ctd} = \alpha_{ct} f_{ctk,0.05} / \gamma_c$ with $\alpha_{ct} = 1.0, \gamma_c = 1.5$				cl.3.1.6
		$f_{ctk,0.05} = 0.7 \times f_{ctm}$		2.25 N/mm ²		T.3.1
		$f_{ctm} = 0.30 \times f_{ck}^{(2/3)} \leq C50/60$ $f_{ctm} = 2.12 \cdot \ln(1 + (f_{cm}/10)) > C50/60$		3.21 N/mm ²		T.3.1
		$f_{cm} = f_{ck} + 8 \text{ (MPa)}$		43 N/mm ²		T.3.1
		Characteristic cylinder strength of concrete, $f_{ck,top}$		35 N/mm ²		T.3.1
		Characteristic cube strength of concrete, $f_{cu,top}$		45 N/mm ²		T.3.1
Normal stress across longitudinal shear interface, $\sigma_n = 0$				0.00 N/mm ²		
Reinforcement ratio, $\rho = A_s / A_i$				0.006		cl.6.2.5
		Area of reinforcement, $A_s = A_{sv,prov}/S$		4825 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, $A_i = 1000 \cdot b_i$		850000 mm ² /m		
Design yield strength of reinforcement, $f_{yd} = f_{yv}/\gamma_s, \gamma_s = 1.15, f_{yv} \leq 600 \text{ N/mm}^2$				435 N/mm ²		1.2.4, cl.3.2
Angle of reinforcement, $\alpha = 90.0^\circ$				90.0 degrees		cl.6.2.5
Design compressive strength, f_{cd}				23 N/mm ²		
		$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$ with $\alpha_{cc} = 1.0, \gamma_c = 1.5$				cl.3.1.6
Strength reduction factor for concrete cracked in shear, v				0.516		
		$v = 0.6 \left[1 - \frac{f_{ck}}{250} \right]$				cl.6.2.2
Longitudinal shear stress limit utilisation, v_{Edi}/v_{Rdi}				51%		OK

CONSULTING ENGINEERS		Engineering Calculation Sheet Consulting Engineers				Job No.	Sheet No.		Rev.																																		
						jXXX	21																																				
						Member/Location																																					
Job Title	Structure, Member Design - Precast Concrete Frame, Bea					Drg. Ref.																																					
Structure, Member Design - PC Frame, Beam and Slab						Made by	XX	Date	2/10/2026																																		
									Chd.																																		
									BS8110																																		
Longitudinal Shear Within Web Rectangular or Flanged Beam (BS8110)																																											
Longitudinal shear stress, $v_h = K_S \cdot \Delta F_c / (b_w \cdot \Delta x)$							3.06	N/mm ²	cl.5.4.7.2																																		
Change of total compression force over Δx , $\Delta F_c = (M-0)/z$							4892	kN	cl.5.4.7.1																																		
Note $M = \text{MAX} \{M_{1,sag} + M_{2,sag}, M_{2,hog}\}$						Sagging	3923	kNm																																			
Lever arm, $z = \{z_{sag2}, z_{hog2}\}$						Sagging	0.802	m																																			
Length under consideration, Δx							2500	mm																																			
Note Δx is the beam length between the point of maximum design moment and the point of zero moment;									cl.5.4.7.2																																		
Shear stress distribution factor, K_S							1.33																																				
The average design shear stress should then be distributed in proportion to the vertical design shear force diagram to give the horizontal shear stress at any point along the length of the member. For UDLs, K_S maybe taken as 2.00 for simply supported beams, 1.33 for continuous beams and 2.00 for cantilever beams;									cl.5.4.7.2																																		
Width (rectangular) or web width (flanged), $b_w = b_{w2}$							850	mm																																			
Longitudinal shear stress limit for no nominal / design vertical reinforcement, $v_{h,l}$							2.50	N/mm ²																																			
Surface type		Washed to remove laitance etc							T.5.5																																		
<table border="1"> <caption>Table 5.5 — Design ultimate horizontal shear stresses at interface</caption> <thead> <tr> <th rowspan="2">Precast unit</th> <th rowspan="2">Surface type</th> <th colspan="3">Grade of in-situ concrete</th> </tr> <tr> <th>25 N/mm²</th> <th>30 N/mm²</th> <th>40 and over N/mm²</th> </tr> </thead> <tbody> <tr> <td rowspan="3">Without links</td> <td>As-cast or as-extruded</td> <td>0.4</td> <td>0.55</td> <td>0.65</td> </tr> <tr> <td>Brushed, screeded or rough-tamped</td> <td>0.6</td> <td>0.65</td> <td>0.75</td> </tr> <tr> <td>Washed to remove laitance or treated with retarder and cleaned</td> <td>0.7</td> <td>0.75</td> <td>0.80</td> </tr> <tr> <td rowspan="3">With nominal links projecting into in-situ concrete</td> <td>As-cast or as-extruded</td> <td>1.2</td> <td>1.8</td> <td>2.0</td> </tr> <tr> <td>Brushed, screeded or rough-tamped</td> <td>1.8</td> <td>2.0</td> <td>2.2</td> </tr> <tr> <td>Washed to remove laitance or treated with retarder and cleaned</td> <td>2.1</td> <td>2.2</td> <td>2.5</td> </tr> </tbody> </table> NOTE 1 The description "as-cast" covers those cases where the concrete is placed and vibrated leaving a rough finish. The surface is rougher than would be required for finishes to be applied directly without a further finishing screed but not as rough as would be obtained if tamping, brushing or other artificial roughening had taken place. NOTE 2 The description "as-extruded" covers those cases in which an open-textured surface is produced direct from an extruding machine. NOTE 3 The description "brushed, screeded or rough-tamped" covers those cases where some form of deliberate surface roughening has taken place but not to the extent of exposing the aggregate. NOTE 4 For structural assessment purposes, it may be assumed that the appropriate value of γ_m included in the table is 1.5.										Precast unit	Surface type	Grade of in-situ concrete			25 N/mm ²	30 N/mm ²	40 and over N/mm ²	Without links	As-cast or as-extruded	0.4	0.55	0.65	Brushed, screeded or rough-tamped	0.6	0.65	0.75	Washed to remove laitance or treated with retarder and cleaned	0.7	0.75	0.80	With nominal links projecting into in-situ concrete	As-cast or as-extruded	1.2	1.8	2.0	Brushed, screeded or rough-tamped	1.8	2.0	2.2	Washed to remove laitance or treated with retarder and cleaned	2.1	2.2	2.5
Precast unit	Surface type	Grade of in-situ concrete																																									
		25 N/mm ²	30 N/mm ²	40 and over N/mm ²																																							
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With nominal links projecting into in-situ concrete	As-cast or as-extruded	1.2	1.8	2.0																																							
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	Washed to remove laitance or treated with retarder and cleaned	2.1	2.2	2.5																																							
Longitudinal shear stress limit for no nominal / design vertical reinforcement util							122%		NOT OK																																		
Required nominal vertical reinforcement per unit length, $0.15\%b_w$							1275	mm ² /m	cl.5.4.7.3																																		
Provided vertical reinforcement per unit length, A_e							4825	mm ² /m																																			
Note $A_e = A_{sv,prov} / S_i$																																											
Required nominal vertical reinforcement per unit length utilisation, $0.15\%b_w/A_e$							26%		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							5955	mm ² /m																																			
$A_h = \frac{1\,000b v_h}{0.95f_y}$									cl.5.4.7.4, Forev																																		
Note $f_{yv} \leq 460\text{N/mm}^2$																																											
Required design vertical reinforcement per unit length utilisation, A_h/A_e							123%		NOT OK																																		
Note UT set to 0% if longitudinal shear stress limit for no design vertical reinforcement UT <= 100%;																																											

[illegible]