Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]

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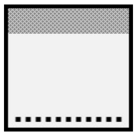
Section:	**(Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa**	
Reinf't:	**2-N20 top, 6-N20 bottom, ku = 0.14, N12 spacer (No ligs)**	
Strength:	**(+ve M) M* = 108.8kNm < øMuo = 186.7kNm**	**OK (0.58)**
Cracking:	**fscr = 188MPa < Fscr = 233MPa & fscr1 = 188MPa < Fscr1 = 400MPa, crack width = 0.2mm**	**OK (0.47,0.81)**
Ast.min:	**Ast.min = 551mm² < Ast = 1885mm² (Minimum of Deemed and actual)**	**OK (0.29)**

Geometry

Interior environment with $\varepsilon csd.b^*=800\times10^{-6}$, $\varepsilon cs^*=540\times10^{-6}$ L/D ratio = 16.7

Concrete strength (f'c) =	40	MPa
Depth (D) =	300	mm
Web width (W) =	1000	mm, (S)lab
Flange width (Bf) =	0	mm



Comp.
Tension

Span (L) =	5000	mm (For effective flange, bef)	Fully enclosed =	N	(Y)es,(N)o
			Side cover =	30	mm
			Formwork =	S	(S)tandard,(R)igid
Concrete weight =	25.0	kN/m³	Exposure top =	A2/A1	Table 4.10.3.2
S.Wt =	7.50	kN/m	Exposure bottom =	B1	Table 4.10.3.2
Gross area (Ag) =	300000	mm²			

Max. dist. b/n lat. restraints for simple/cont. (L1=min(60*bef,180*bef²/D) = 60000 mm - Cl 8.9.2

Design actions

Analysis values = M (M)anual, (L)eft, Position (X) from analysis, (R)ight

	Manual values			**Manual**	**Units**
Design (M*) =	108.8	kNm	**M***	108.8	kNm
Design (Ms1*, Ψs=1) =	80.6	kNm	**Ms1*(Ψs=1)**	80.6	kNm
Design (Ms*, Ψs) =	80.6	kNm	**Ms*(Ψs)**	80.6	kNm
			Ast req'd	1068	mm²
			Ast	1885	mm²
			Reinf't req'd	3.4-N20	

Reinforcement

Ligs = N12 spacer (No ligs)

Bottom reinf't =	**6-N20**		**Top reinf't =**	**2-N20**	
Bar size =	20	mm	Bar size =	20	mm
Bar cts/No/mm² =	6	No	Bar cts/No/mm² =	2	No
Yield strength (fsy) =	500	MPa	Yield strength (fsyc) =	500	MPa
Ductility class =	A	(N)ormal,(L)ow,(A)uto	Ductility class =	A	(N)ormal,(L)ow,(A)uto
Reinf't ductility class =	N	(N)ormal,(L)ow	Reinf't ductility class =	N	(N)ormal,(L)ow
Steel area (Ast) =	1885	mm²	Steel area (Asc) =	628	mm²
Bottom cover to ligs =	30	mm	Top cover to ligs =	20	mm
Depth to bottom steel layer (ds.max) =	248	mm	Depth to top steel layer =	42	mm
Depth to bottom steel (ds) =	248	mm	Depth to top steel =	42	mm
D-ds =	52	mm	D-ds =	258	mm
No. bars =	6.0	No.	No. bars =	2.0	No.
Bar centres =	179	mm	Bar centres =	896	mm
Max. bars per layer =	12		Max. bars per layer =	12	
Layers required =	1		Max. bars per 2nd layer =	12	
			Layers required =	1	

Strength in +ve bending for beams - Cl 8.1

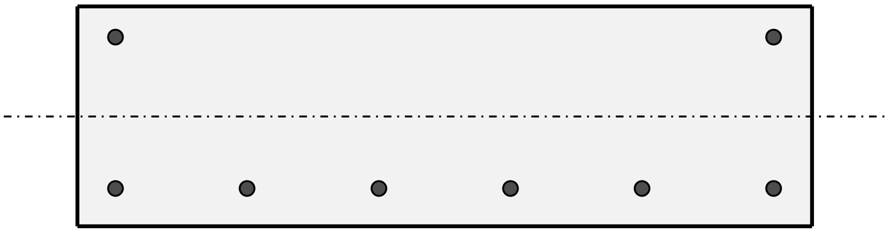
Design width (W) =	1000	mm	Design flange (bef) =	1000	mm
Tensile steel area (As) =	1885	mm²	Depth to tensile (ds) =	248	mm
Comp. steel area (Ac) =	0	mm²	Depth to comp. steel (dc) =	42	mm
Ultimate Moment (Mu) =	219.7	kNm	ku =	0.138	
Design capacity (øMuo) =	186.7	kNm	Factor (ø) =	0.850	Table 2.2.2
			As.min =	551	mm²

Crack control in +ve bending for beams - Cl 8.6

Steel stress (σscr1) =	188	MPa	Max. stress (σscr1.max) =	400	MPa	OK (0.47)
Steel stress (σscr) =	188	MPa	Max. stress (σscr.max) =	233	MPa	OK (0.81)
Max. crack width (w'max) =	0.3	mm	Calculated crack width =	0.22	mm	

Reinf't: **6.0-N20 bottom tensile**
2.0-N20 top compression
Ligs: **N12 spacer (No ligs)**

Preview



Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]

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Strength in Bending

alpha (α2 = 0.85-0.0015*f'c) =	0.790	(α2 ≥ 0.67)	Eq 8.1.3(1)
gamma (γ = 0.97-0.0025*f'c) =	0.870	(γ ≥ 0.67)	Eq 8.1.3(2)
Max. concrete strain (εu) =	0.0030		
Max. steel strain (εsy = fsy/Es) =	0.0025		
ku.b = εu/(εu+fsy/Es) =	0.545		

M* =	108.78	kNm	
tf =	0	mm	
tf.min (rect. design) =	0.0	mm	LC Eq 3.7(9)

Positive (+ve) bending

Steel ratio (p) =	0.0076	
Compression equation valid (Rect) =	Yes	
Bar =	20	mm
Centres =	179.2	mm
Tensile cover =	30	mm
Ast =	1885	mm^2/m
Asc =	628	mm^2/m
ds =	248	mm
ds.max =	248	mm
dc =	42	mm
b =	1000	mm

Negative (-ve) bending

Steel ratio (p) =	0.0024		
Bar =	20	mm	
Centres =	896.0	mm	
Tensile cover =	20	mm	
Ast =	628	mm^2/m	
Asc =	628	mm^2/m	Area Limited to Ast
ds =	258	mm	
ds.max =	258	mm	
dc =	52	mm	
b =	1000	mm	

Compression steel at yield (γkud in flange if T or L)

ku =	0.092		WRH Eq 4.49
εst =	0.0296		WRH Eq 4.50
εsc =	-0.0025		WRH Eq 4.51
Valid (εst,εsc ≥ εsy) =	0	(0=No, 1=Yes)	
Mu =	0.0	kNm	WRH Eq 4.53

Compression steel at yield

ku =	0.000		WRH Eq 4.49
εst =	0.0000		WRH Eq 4.50
εsc =	0.0000		WRH Eq 4.51
Valid (εst,εsc ≥ εsy) =	0	(0=No, 1=Yes)	
Mu =	0.0	kNm	WRH Eq 4.53

Compression steel not at yield (γkud in flange if T or L)

u1 =	-0.08294		WRH Eq 4.56
u2 =	-0.00936		WRH Eq 4.57
ku =	0.147		WRH Eq 4.55
kud (with Asc, if in flange) =	36.4	mm	Comp. steel ignored in tension
γ.kud (if in flange) =	29.8	mm	Asc=0mm^2 for ku calc.
Compression steel in compression =	0	(0=No, 1=Yes)	
kud =	34.3	mm	
Mu (if γkud in flange) =	219.7	kNm	WRH Eq 4.58

Compression steel not at yield

u1 =	0.00886		WRH Eq 4.56
u2 =	-0.01071		WRH Eq 4.57
ku =	0.099	mm	WRH Eq 4.55
kud (with Asc) =	25.6	mm	Comp. steel ignored in tension
γ.kud =	9.9	mm	Asc=0mm^2 for ku calc.
Compression steel in compression =	0	(0=No, 1=Yes)	
kud =	11.4	mm	
Mu =	79.5	kNm	WRH Eq 4.58

Calculation for T or L beam with of γkud not in the flange - Not applicable

ku in flange =	1	(0=No, 1=Yes)	
kud (if in web) =	34.3	mm	
Mu (if γ.kud in web) =	219.7	kNm	RCB - Eq 3.78

Design kud =	34.3	mm		Design kud =	11.4	mm		
γ.kud =	29.8	mm		γ.kud =	9.9	mm		
Comp. bar in compr. =	0	(0=No, 1=Yes)		Comp. bar in compr. =	0	(0=No, 1=Yes)		
εs =	0.0187			εs =	0.0647			
Ts =	942.5	kN	500.0 MPa	Ts =	314.2	kN	500.0 MPa	
εc =	-0.0007			εc =	-0.0107			
Cs =	0.0	kN	0.0 MPa	Cs =	0.0	kN	0.0 MPa	
Cc = Cw =	942.5	kN		Cc =	314.2	kN		
Cf =	0.0	kN						
T - Cs - Cc =	0.0	kN		T - Cs - Cc =	0.0	kN		
ku =	0.138			ku =	0.044			
Muo =	219.7	kNm	WRH Eq 4.58	Muo =	79.5	kNm		
Reinf't ductility class =	N	(N)ormal,(L)ow		Reinf't ductility class =	N	(N)ormal,(L)ow		
kuo = kud/ds.max =	0.138			kuo =	0.044			
ø = 1.24-13*kuo/12 =	0.850	(0.65 ≤ ø ≤ 0.85) Table 2.2.2(b)(i) for N Class		ø = 1.24-13*kuo/12 =	0.850	(0.65 ≤ ø ≤ 0.85) Table 2.2.2(b)(i) for N Class		
Capacity (øMuo) =	186.7	kNm		Capacity (øMuo) =	67.6	kNm		
Ast req'd =	1068	mm²/m		Ast req'd =	0	mm²/m		
Factor - øMuo/(ø*fsy*Ast*ds) =	0.940			Factor - øMuo/(ø*fsy*Ast*ds) =	0.981			

Minimum strength requirements - Cl 8.1.6.1

Positive (+ve) bending				Negative (-ve) bending			
Uncracked neutral axis (uk) =	0.506		OS RCB-1.1(1) Fig 5.3	Uncracked (uk) =	0.506		OS RCB-1.1(1) Fig 5.3
Depth of uncracked neutral axis NA = uk*D =	152	mm		Depth of uncracked NA =	152	mm	
Uncracked factor for stiffness (uκ) =	1.057			Uncracked (uκ) =	1.057		
Iuncrk = uκ*W*D³/12 =	2379	x10⁶ mm⁴	OS RCB-1.1(1) Eq 7.2(2)	Iuncrk = uκ*W*D³/12 =	2379	x10⁶ mm⁴	OS RCB-1.1(1) Eq 7.2(2)
Zb = Iuncrk/(D-NA) =	16066	x10³ mm³		Zt = Iuncrk/NA =	15661	x10³ mm³	
Char. flex. tens. strength (f'ct.f = 0.6*√f'c) =	3.79	MPa	Cl 3.1.1.3				
(Muo)min = 1.2*Zb*fct.f =	73.2	kNm	Eq 8.1.6.1(1)	(Muo)min = 1.2*Zt*fct.f =	71.3	kNm	Eq 8.1.6.1(1)
Ast.min (Muo)min =	602	mm²		Ast.min (Muo)min =	563	mm²	
Deemed to comply Ast.min =	[αb*(D/d)²*f'ct.f/fsy]*(bw*d)			Deemed to comply Ast.min =	[αb*(D/d)²*f'ct.f/fsy]*(bw*d)		Cl 8.1.6.1
αb =	0.200			αb =	0.200		
Deemed to comply Ast.min =	551	mm²		Deemed to comply Ast.min =	529	mm²	
Actual bar cts =	179	mm		Actual bar cts =	896	mm	
Max. bar cts =	300	mm	OK Cl 8.6.1(b)	Max. bar cts =	300	mm	No Good Cl 8.6.1(b)

Side face reinforcement = Not required as face depth ≤ 750mm - Face depth = 300mm Cl 8.6.4

Crack control of beams - Cl 8.6 Interior environment with $\varepsilon csd.b^*=800x10^{-6}$, $\varepsilon cs^*=540x10^{-6}$

Uncracked NA =	152 mm	OS RCB-1.1(1) Fig 5.3
Uncracked uκ =	1.06	
Iuncrk = uκ*W*D³/12 =	2379 x10⁶ mm⁴	OS RCB-1.1(1) Eq 7.2(2)

Positive (+ve) bending			**Negative (-ve) bending**		
Tension on bottom face =	N20 bars		Tension on top face =	N20 bars - Not applicable	
Tensile area of conc. prior to cracking (Act) =	148087 mm²		Reverse bending (Act) =	151913 mm²	
Tensile area of conc. (mid-depth) (Actm) =	150000 mm²		Reverse bending (Actm) =	150000 mm²	Between mid depth and tensile Cl 8.2.4.2.2(e)
Tensile area of conc. (Uncracked) (Actn) =	148087 mm²		Reverse bending (Uncracked) (Actn) =	151913 mm²	Between uncracked na and tensile Cl 8.2.4.4-AS 5100.5
Cracked neutral axis (k) =	0.262	OS RCB-1.1(1) Fig 5.7	Cracked (k) =	0.161	OS RCB-1.1(1) Fig 5.7
Depth of cracked NA = k*ds =	65 mm		Depth of cracked NA =	42 mm	
Cracked factor for stiffness (κ) =	0.377		Cracked (κ) =	0.143	
Icr = κ*W*ds³/12 =	480 x10⁶ mm⁴		Icr = κ*W*ds³/12 =	204 x10⁶ mm⁴	
Steel tensile stress (σscr1) =	188 MPa		Steel tensile stress (σscr1) =	522 MPa	
Max. tensile stress (bar) (σscr.max) =	195 MPa	Table 8.6.2.2(A)	bar σscr.max =	195 MPa	Table 8.6.2.2(A)
Max. tensile stress (cts) (σscr.max) =	233 MPa	Table 8.6.2.2(B)	cts σscr.max =	0 MPa	Table 8.6.2.2(B)
Max. tensile stress (σscr.max) =	233 MPa		σscr.max =	195 MPa	
	OK (0.81)			No Good (2.68)	
Outermost steel tensile stress (σscr) =	188 MPa		Outermost steel tensile stress (σscr) =	522 MPa	
Max. tensile stress (σscr1.max=0.8*fsy) =	400 MPa	Cl 8.6.2.2	σscr1.max= 0.8*fsy =	400 MPa	Cl 8.6.2.2
	OK (0.47)			No Good (1.31)	
AS 3600 - 2009					
Max. tensile stress (bar) (σscr.max) =	242 MPa	Table 8.6.1(A)	bar σscr.max =	242 MPa	Table 8.6.1(A)
Max. tensile stress (cts) (σscr.max) =	257 MPa	Table 8.6.1(B)	cts σscr.max =	0 MPa	Table 8.6.1(B)
Max. tensile stress (σscr.max) =	257 MPa		σscr.max =	242 MPa	

Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]

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Crack control of beams by calculation of crack widths - Cl 8.6

Design shrinkage strain (εcs) =	540	x10^{-6}	Cl 3.1.7.2	(Refer Creep/Shrinkage)					
Constant sustained stress (σo) =	0	MPa	Cl 3.1.8.1	(Refer Creep/Shrinkage)					
Creep coefficient (øcc) =	2.316		Cl 3.1.8.3	(Refer Creep/Shrinkage)					
Eff. modular ratio (ne) = (1+øcc)*Es/Ec =	20.3		Cl 8.6.2.3						
hc.eff =min(2.5*(D-ds),(D-NA)/3,D/2) =	78.3	mm	Cl 8.6.2.3		hc.eff =	86.1	mm	Cl 8.6.2.3	
hc.effa = min(hc.eff,D-cracked NA) =	78.3	mm			hc.effa = min(hc.eff,cracked NA)	41.6	mm		
Eff. area of concrete in tension (Ac.eff) =	78336	mm^2	Cl 8.6.2.3		Ac.eff =	41613	mm^2	Cl 8.6.2.3	
Reinforcement ratio (Peff) = Ast/Ac.eff =	0.024		Cl 8.6.2.3		Peff =	0.015		Cl 8.6.2.3	
εdm.min = (εsm - εcm).min = 0.6*σscr/Es =	565	x10^{-6}	Eq 8.6.2.3(2)		εdm.min =	1566	x10^{-6}	Eq 8.6.2.3(2)	
Strain in tensile steel = σscr/Es =	941	x10^{-6}	Eq 8.6.2.3(2)		σscr/Es =	2610	x10^{-6}	Eq 8.6.2.3(2)	
Flexural tensile f'ct.f = 0.6*√(f'c) =	3.79	MPa	Cl 3.1.1.3						
Char. uniaxial tensile (fct = 0.6*f'ct.f) =	2.28	MPa	Cl 3.1.1.3						
Mean char. (fct.m=1.4*fct) =	3.19	MPa	Cl 3.1.1.3		Mean (fct.m=1.4*fct) =	3.19	MPa	Cl 3.1.1.3	
-0.6*fct.m/(Es*Peff) =	-397	x10^{-6}			-0.6*fct.m/(Es*Peff) =	-633	x10^{-6}		
1+ne*peff =	1.489				1+ne*peff =	1.307			
Design shrinkage strain εcs =	540	x10^{-6} ±30%			Design shrinkage strain εcs =	540	x10^{-6} ±30%		
.1 = σscr/Es-0.6*fct/(Es*Peff)*(1+n*Peff))+ecs =	889	x10^{-6}			εdm.1 =	2323	x10^{-6}		
εdm = εsm - εcm = max(εdm.min,εdm.1) =	889	x10^{-6}	Eq 8.6.2.3(2)		εdm = εsm - εcm =	2323	x10^{-6}	Eq 8.6.2.3(2)	
Axial (N*) =	0	kN (Tension -ve)							
Cross section area (Area) =	300000	mm^2							
Stress from axial (σaxial) =	0.00	MPa							
Strain from axial (εaxial) =	0	x10^{-6} (Tension +ve)			εAxial =	0	x10^{-6} (Tension +ve)		
Tensile strain at top (εt) =	-260	x10^{-6}			εt =	3117	x10^{-6}		
Tensile strain at bottom (εb) =	1208	x10^{-6}			εb =	-420	x10^{-6}		
ε1 = max(εt,εb) =	1208	x10^{-6}	Cl 8.6.2.3		ε1 = max(εt,εb) =	3117	x10^{-6}	Cl 8.6.2.3	
ε2 = min(εt,εb) =	0	x10^{-6}	Cl 8.6.2.3		ε2 = min(εt,εb) =	0	x10^{-6}	Cl 8.6.2.3	
Bar shape =	D	(P)lain,(D)eformed			Bar shape =	D	(P)lain,(D)eformed		
Coefficient for bonded reinforcement (k1) =	0.8		Cl 8.6.2.3		k1 =	0.8		Cl 8.6.2.3	
Coefficient for long. strain dist. (k2) =	0.50		Cl 8.6.2.3		k2 =	0.50		Cl 8.6.2.3	
Bar centres (cts) =	179	mm			Bar centres (cts) =	896	mm		
Clear cover to long. reinf't (c = cover+ligs) =	42	mm			Clear cover (c = cover+ligs) =	32	mm		
Max. bar spacing (5*(c+0.5*db)) =	260	mm			Maximum bar spacing (cts) =	210	mm		
sr.max.1 = 1.3*(D-kd) =	306	mm	Eq 8.6.2.3(3)		sr.max.1 =	336	mm	Eq 8.6.2.3(3)	
sr.max.2 = 3.4*c+0.3*k1*k2*db/Peff) =	243	mm	Eq 8.6.2.3(3)		sr.max.2 =	268	mm	Eq 8.6.2.3(3)	
sr.max = min(srmax.1,sr.max.2) =	243	mm			sr.max = min(srmax.1,sr.max.2) =	268	mm		
Max. final crack spacing (sr.max) =	243	mm	Eq 8.6.2.3(1)		Max. final crack spacing (sr.max) =	0	mm	Eq 8.6.2.3(1)	
Char. maximum crack width (w'max) =	0.3	mm	Cl 8.6.2.2						
Calculated crack width (w) =	0.216	mm	Eq 8.6.2.3(3)	OK	w =	0.622	mm	Eq 8.6.2.3(3)	No Good

Primary reinforcement

Minimum for strength =	1068 mm²	3.4-N20
Deemed-to-comply minimum =	551 mm²	1.8-N20
Actual minimum for øMuo =	602 mm²	1.9-N20

Section:	**(Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa**	
Reinf't:	**6.0-N20 bottom tensile, N12 spacer (No ligs) anchored in compr. zone**	
	2.0-N20 top compression	
	No ligs required	
Strength:	**V* = 0.0kN ≤ ks*øVuc = 213.3kN, V* = 0.0kN ≤ øVu = 213.3kN - No shear ligs required**	**OK (0.00)**
	øVuc = 213.3kN, øVu.min = 332.6kN, øVus = 0.0kN	
Torsion:	**T* = 0.0kNm ≤ øTus = 0.0kNm - No torsion ligs required**	**OK (0.00)**
	0.25øTcr = 12.6kNm, øTcr = 50.6kNm	
Combined:	**Vcomb* = 0.0kN ≤ øVu.max = 1424.3kN**	**OK (0.00)**
Add'l reinf't:	**Additional tensile shear reinforcement not required at Manual location**	

Strength of beams in shear - Cl 8.2 - Design actions

Warning - Capacity only valid for current combination of V* and M*

Analysis values =	M	(M)anual, (L)eft, Position (X) from analysis, (R)ight, (C)ritical			
Manual shear (V*) =	0.0	kN	Design shear (V*) =	0	kN
Manual moment (M*) =	108.8	kNm	Design moment (M*) =	109	kNm (+ve or -ve)
Manual section max. moment (Mmax*) =	108.8	kNm	Section Max. (Mmax*) =	109	kNm (+ve or -ve)
				(blank = M*)	
Axial (N*) =	0	kN (Tension -ve) (Refer column for bending/axial capacity)			
Torsion (T*) =	0	kNm			
do (+ve bending) =	248	mm	dv (+ve bending) =	223	mm
do (-ve bending) =	258	mm	dv (-ve bending) =	232	mm

Consideration of torsion - Cl 8.2.1.2 No torsion - Torsional reinf't not required

øTcr = ø*0.33*√f'c*(Acp²/uc)√(1+(σcp/(0.33*√f'c))) =	50.6	kNm	Cl 8.2.1.2(2) - Refer below	
0.25*øTcr =	12.6	kNm	Cl 8.2.1.2(1)	
Consider torsional effects (T* > 0.25*øTcr) =	N	(Y)es,(N)o	Cl 8.2.1.2	
T*/0.25øTcr =	0.00		Cl 8.2.1.2(1)	No torsion (0.00)
Torsion ignored, V* =	0.0	kN		

Strength of beams in shear - Cl 8.2

Use Simplified method =	N	(Y)es,(N)o	
Max. nominal aggregate size (dg) =	16	mm	(General method)
Lightweight concrete =	N	(Y)es,(N)o	(General method)
Cracking in compression zone =	N	(Y)es,(N)o	Cl 8.2.4.5

Reinforcement - No shear ligs required

Description =	**N12 spacer (No ligs)**		Ignore ligs =	Y	(Y)es,(N)o
Lig size =	12	mm	Leg area (Asl) =	0	mm²
Spacing (s) =	225	mm	Asv.req'd =	0	mm² (2-0.0mm dia.)
Legs in cross-section =	2		s.req'd =	0	mm
Steel yield strength (fsy.f) =	500	MPa	s.max =	0	mm
Fitment class =	N	(N)ormal,(L)ow	s.max.t =	0	mm
Angle between shear & tensile reinf't (αv) =	90.0	°			

Shear spacing - Cl 8.3.2.2

Max. cts = 0.75*D =	225	mm < 500mm	Ligs area (Asv) =	0	mm²
Max. cts =	225	mm	Adopted ligs area (Asv) =	0	mm²
			Asv.min =	228	mm² (2-12.0mm dia.)

Transverse spacing - Cl 8.3.2.2

Transverse spacing =	928	mm	Waive max. spacing =	N	(Y)es,(N)o
Max.transverse spacing =	300	mm	Waive transv. spacing =	N	(Y)es,(N)o

Torsion spacing - Cl 8.3.3

			Lig area (Asw) =	0	mm²
Max. cts = 0.12*uh =	279	mm < 300mm	Adopted ligs area (Asw) =	0	mm²
Max. cts =	280	mm	Asw.min =	114	mm² (12.0mm dia.)

Tensile reinforcement within tensile D/2 (Used for General Method and additional longitudinal forces):

Ast Req'd for M* =	1068 mm² (3.4-N20)	Bottom bars in tension		
4-N20 =	1257 mm²			
Design (Ast) =	1885 mm² (6-N20)	Design (Asc) =	628 mm² (2-N20)	
Manual (Ast) =		mm² (blank = design)	Manual (Asc) =	mm² (blank = design)
Adopted (Ast) =	1885 mm² (6-N20)	Adopted (Asc) =	628 mm² (2-N20)	
Ast req'd for Mmax* =	1068 mm² (3.4-N20)			

Concrete contribution to shear strength - Cl 8.2.4

do = ds.max =	248	mm	
Dist. to centroid of D/2 tensile reinf't (ds) =	248	mm	(Positive bending, bottom bars in tension)
Effective shear depth dv = max(0.72*D,0.9*ds) =	223	mm	Cl 8.2.1.9
Effective flange (bv) =	1000	mm	

Simple method for kv & θv - Cl 8.2.4.3

Asv < Asv.min, kvo = 200/(1000+1.3*dv) =	0.155		Eq 8.2.4.3(1)
Asv < Asv.min, kv = min(kvo, 0.15) =	0.150		Eq 8.2.4.3(1)
Angle of inclination of concrete comp. strut (θv) =	36.0	°	
Long. mid-depth concrete strain (εx) = 0.85*fsy/(2*Es) =	1062.5	$\times 10^{-6}$	

General method for kv & θv - Cl 8.2.4.2

Tensile area of concrete (Act) =	150000	mm²	Area between mid depth and tensile fibre
Adopted (Ast) =	1885	mm²	
Shear only εx1 ≤ 3000x10^{-6} =	646.4	$\times 10^{-6}$	Eq 8.2.4.2.2(1)
Shear only -200x10^{-6} ≤ εx2 (where εx1 < 0) ≤ 0 =	0.0	$\times 10^{-6}$	Eq 8.2.4.2.2(2)
Shear/torsion εxt1 ≤ 3000x10^{-6} =	-	$\times 10^{-6}$	Eq 8.2.4.2.3(1)
Shear/torsion -200x10^{-6} ≤ εxt2 (where εxt1 < 0) ≤ 0 =	-	$\times 10^{-6}$	Eq 8.2.4.2.3(2)
Long. mid-depth concrete strain (εx) =	646.4	$\times 10^{-6}$	Cl 8.2.4.2.2
kdg = max(32/(16+dg), 0.8) =	1.000		Eq 8.2.4.2(3)
Asv < Asv.min, kv = (0.4/(1+1500*εx))*(1300/(1000+kdg*dv)) =	0.216		Eq 8.2.4.2(2)
Angle (θv) = (29+7000*εx) =	33.5	°	Eq 8.2.4.2(1)

Adopted kv & θv - General method - Cl 8.2.4.2

kv =	0.216	
Angle of inclination of concrete comp. strut (θv) =	33.5	°

Concrete contribution to shear stength - Cl 8.2.4.1

Strength reduction factor (ø) =	0.7		Table 2.2.2(e)
kv =	0.216		General method - Cl 8.2.4.2
Effective flange (bv) =	1000	mm	
Effective shear depth (dv) =	223	mm	Cl 8.2.1.9
min(√f'c,8.0) =	6.32	MPa	
Vuc = kv*bv*dv*min(√f'c, 8.0) =	304.7	kN	Eq 8.2.4.1
D ≤ 300mm, ks =	1.00		
(ø=0.7)Vuc =	213.3	kN	
(ks=1.00)*(ø=0.7)Vuc =	213.3	kN	Eq 8.2.1.6(1)

Transverse shear and torsion reinforcement contribution - Cl 8.2.5

Strength reduction factor (ø) =	0.7		Table 2.2.2(e)	
øVuc =	213.3	kN		
Torsion ignored, V* =	0.0	kN	Cl 8.2.1.2	
Angle between shear & tensile reinf't (αv) =	90	°		
Angle (αvr) =	1.571	rad		
Angle of inclination of concrete compression strut (θv) =	33.5	°	General method - Cl 8.2.4.2	
Angle (θvr) =	0.585	rad		
Vus = (Asv*fsy.f*dv/s)*cot(θv) =	0.0	kN	Eq 8.2.5.2(1)	
øVus =	0.0	kN		
øVu = øVuc + øVus =	213.3	kN		
Ultimate shear strength (øVu = øVuc) =	213.3	kN	Cl 8.2.3.1	OK (0.00)
øVus.req'd = V* - øVuc =	0.0	kN		
Vus.reqd =	0.0	kN		
Asv.reqd = Vus/((Asv*fsy.f*dv/s)*cot(θv)) =	0	mm² (0.0mm dia.)	Eq 8.2.5.2(1)	
s.reqd = (Asv*fsy.f*dv/Vus)*cot(θv) =	0	mm	Eq 8.2.5.2(1)	

Minimum transverse shear reinforcement - Cl 8.2.1.7

σmin = 0.08*√f'c =	0.51	MPa	Cl 8.2.1.7
Asv.min = σmin*bv*s/fsy.f =	228	mm²	Eq 8.2.1.7
s.max = Asv*fsy.f/(σmin*bv) =	0	mm	
Vus.min = (Asv.min*fsy.f*dv/s)*cot(θv) =	170.5	kN	Eq 8.2.1.7
øVu.min = ø(Vuc + Vus.min) =	332.6	kN	

Maximum shear strength limited by web crushing - Cl 8.2.3.3

kc =	0.55		Cl 8.2.3.3(1)
Vu.max = kc*[0.9*f'c*bv*dv*((cot(θv)+cot(αv))/(1+cot²(θv)))]	2034.8	kN	Eq 8.2.3.3(1)
øVu.max = (ø=0.7)*Vu.max =	1424.3	kN	Table 2.2.2(e)(ii)
øVus.max = øVu.max - øVuc =	1211.1	kN	
Asv.max = øVus.max/(ø(fsy.f*dv/s)*cot(θv)) =	2311	mm²	Eq 8.2.5.2(2)
when s =	225	mm	

Torsional strength of a beam without reinforcement - Cl 8.2.1.2

Outside perimeter of concrete cross-section (uc) =	2600	mm	Cl 8.2.1.2	
Area enclosed by oustide perimeter (Acp) =	300000	mm²		

Torsional capacity of concrete - Eq 8.2.1.2(2):

Strength reduction factor (ø) =	0.70		Table 2.2.2	
σcp =	0.00	MPa	Cl 8.2.1.2	
Tcr = 0.33*√f'c*(Acp²/uc)√(1+(σcp/(0.33*√f'c))) =	72.2	kNm	Eq 8.2.1.2(2)	
øTcr =	50.6	kNm		
0.25*øTcr =	12.6	kNm		
Consider torsional effects (T*>0.25*øTcr) =	N	(Y)es,(N)o	Cl 8.2.1.2	
T*/0.25øTcr =	0.00		Cl 8.2.1.2(1)	No torsion (0.00)

Torsional strength of a beam including reinforcement - Cl 8.2.5.6

x = Min(D & W) =	300 mm	Dxyo =	238 mm	xo =	238 mm
y = Max(D & W) =	1000 mm	Wxyo =	928 mm	yo =	928 mm

Perimeter of torsion ligs (centreline) (uh = 2*(xo + yo)) =	2332	mm	Cl 8.2.3.3	
Area enclosed by ligs (Aoh = xo*yo) =	220864	mm²	Cl 8.2.3.3	
Enclosed area by shear flow (Ao = 0.85*Aoh) =	187734	mm²	Cl 8.2.5.6	
Angle of inclination of concrete compression strut (θv) =	33.5	°		
Angle (θvr) =	0.585	rad		
Area of ligs (Asw) =	0	mm²		
Tus = 2.0*Ao*Asw*fsy.f*/s*cot(θv) =	0.0	kNm	Eq 8.2.5.6	
øTus =	0.0	kNm		
T*/øTus =	0.00			OK (0.00)

Minimum torsional reinforcement - Cl 8.2.5.5

Tcr =	72.2	kNm	
Asw.min1 = Asv.min/legs =	114	mm²	Cl 8.2.5.5(b)(i) & Eq 8.2.1.7
Asw.min2 = 0.25*Tcr/(2.0*Ao*fsy.f/s*cot(θv)) =	14	mm²	Cl 8.2.5.5(b)(ii) & Eq 8.2.5.6
Asw.min = max(Asw.min1, Asw.min2) =	114	mm²	
s.max1 = Asv*fsy.f/(σmin*bv) =	0	mm	Cl 8.2.5.5(b)(i) & Eq 8.2.1.7
s.max2 = (2.0*Ao*Asw*fsy.f*cot(θv))/0.25*Tcr =	0	mm	Cl 8.2.5.5(b)(ii) & Eq 8.2.5.6
s.max = min(s.max1, s.max2) =	0	mm	No ligs
Tus.min = 2.0*Ao*Asw.min*fsy.f*/s*cot(θv) =	143.4	kNm	Eq 8.2.5.6
øTu.min =	100.4	kNm	

Shear and torsional strength limited by crushing - Cl 8.2.3.3

V*/(bv*dv) =	0.000	MPa		
T*.*uh/(1.7*Aoh²) =	0.000	MPa		
Vcombσ* = √[(V*/(bv*dv))²+(T*.*uh/(1.7*Aoh²))²] =	0.000	MPa	Eq 8.2.3.3(5)	
øVu.max =	1424.3	kN		
Vcombσ.limit = øVu.max/(bv*dv) =	6.381	MPa	Eq 8.2.3.3(5)	
Vcomb* = Vcombσ*.*(bv*dv) =	0.0	kN		
Vcombσ*/Vcombσ.limit or Vcomb*/øVu.max =	0.00			OK (0.00)

Additional longitudinal tension forces caused by shear and torsion - Cl 8.2.7 (Not applicable)

M* =	108.8	kNm	
V* =	0.0	kN	
T* =	0.0	kNm	
N* =	0.0	kN	
Steel area req'd (Ast.reqd) =	1068	mm²	
Adopted area (Ast) =	1885	mm²	
Lever z = ds*(1-fsy*Ast/(2*α2*f'c*bw*ds)) =	233	mm	
Compressive steel area (Asc) =	628	mm²	
uo = 0.92*uh =	2145	mm	
0.5*(V*+øVuc) =	106.6	kN > V* (Limited)	
Shear contribution ΔFtds* = V*.*cot(θv) =	0.0	kN	Eq 8.2.7(2)
Torsion contribution ΔFtdt* = (Ignored) =	0.0	kN	Eq 8.2.7(3)
ΔFtd* = ΔFtds + ΔFtt* =	0.0	kN	Eq 8.2.7(1)
Flexural tension side	Normal ductility reinforcement		
Strength reduction factor in bending (øbt) =	0.85		
Strength reduction factor on tension (øtt) =	0.85		Table 2.2.2(b)
Flexural tensile side = Ttd* = (M*/z-N*/2+ΔFtd*) =	466.7	kN	Eq 8.2.8.2(1)
Ast = ΔTtd*/(øtt*fsy) =	1098	mm²	Eq 8.2.8.2(2)
Add'l Ast = min(Max. Ast.Req'd, Ast) - Ast prov'd =	0	mm²	
Add'l reinf't req'd =	0.0-N20		
Flexural compression side	Normal ductility reinforcement		
Strength reduction factor in bending (øbc) =	0.85		
Strength reduction factor on compression side (øtc) =	0.85		Table 2.2.2(b)
Flexural compression side = Tcd* = (-M*/z-N*/2+ΔFtd*) =	0.0	kN	Eq 8.2.8.3(1)
Asc = ΔTcd*/(øtc*fsy) =	0	mm²	Eq 8.2.8.3(2)
Add'l Asc = Asc - Asc prov'd =	0	mm²	
Add'l reinf't req'd =	0.0-N20		

Section: **(Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa**
Reinf't: **2-N20 top, 6-N20 bottom (Additional reo specified)**
Defl'n: **δ.dl = 9.7mm, δ.ll = 0.0mm, δ.inc = 9.7mm, δtotal = 15.8mm (span = 5000mm, L/316)** **OK**
σcs for Interior environment with $\varepsilon csd.b^*=800x10^{-6}$, $\varepsilon cs^*=540x10^{-6}$

Deflections - Cl 8.5.3 Fixed-End Beam at 4782mm from LHS

Concrete density (ρ) =	2400	kg/m³ Cl 3.1.3	Gross area (Ag) =	300000	mm²
Use fcmi? =	Y	(Y)es,(N)o	Uncr.g. neutral axis (NA) =	150	mm from top
fcmi =	43.3	MPa	Gross Stiffness (Ig) =	2250	x10⁶ mm⁴ (w/o reinf't)
Deflection at =	M	(M)anual, (C)ritical, (L)eft, Position (X) from analysis, (R)ight			
Position (x) =	Manual	mm	Steel Modulus (Es) =	200000	MPa Cl 3.2.2
Span type =	Manual	Mod. of elast. (Ec = $\rho^{1.5}$*(0.024*√fcmi+0.12)) =		32668	MPa ± 20% Cl 3.1.2
			Modular ratio (n = Es/Ec) =	6.122	

Deflection calculation - Interior/Fixed

		Left	At x	Right	Units		
	Manual ultimate (M*) =	-217.6	108.8	-217.6	kNm	At x = 0 for cantilever	
	Manual short-term serviceability (Ms*) =	-161.2	80.6	-161.2	kNm		
	Analysis ultimate (M*) =	-217.6	108.8	-217.6	kNm	Red values of M*/Ms* manually input	
	Analysis short-term serviceability (Ms*) =	-161.2	80.6	-161.2	kNm		
Top reinf't:	Ast req'd =	2122	0	2122	mm²	**Manual:**	
	Design Ast =	2827	628	2827	mm²	Short term LL factor (ψs) =	0.70
	Ast =	2827	628	2827	mm²	Long term LL factor (ψl) =	0.40
		9.0-N20	2.0-N20	9.0-N20		Short term LL factor (ψs) =	0.70
Bottom reinf't:	Ast req'd =	0	1068	0	mm²	Long term LL factor (ψl) =	0.40
	Design Ast =	628	1885	628	mm²		
	Ast =	628	1885	628	mm²		
		2.0-N20	6.0-N20	2.0-N20			
	Uncracked NA =	146	152	146	mm	OS RCB-1.1(1) Fig 5.3	
	Uncracked uκ =	1.087	1.057	1.087	x10⁶ mm⁴		
	Iuncrk = uκ*W*D³/12 =	2445	2379	2445	x10⁶ mm⁴	OS RCB-1.1(1) Eq 7.2(2)	
	Tensile steel (Ast) =	2827	1885	2827	mm²		
	Depth to ds =	258	248	258	mm (From comp. face)		
	Comp. steel (Asc) =	628	628	628	mm²		
	dc =	52	42	52	mm (From comp. face)		
	Cracked κ =	0.305	0.262	0.305	mm	OS RCB-1.1(1) Fig 5.7	
	Depth to cracked NA = κ*ds =	78.8	65.0	78.8	mm (From top)		
	Use Comp. steel =	Yes	Yes	Yes			
	yt =	146	148	146	mm (From tensile fibre)		
	Design shrinkage strain εcs =	540	540	540	x10⁻⁶ ±30%	Interior env. refer Shrinkage tab	
	W =	1000	1000	1000	mm		
	Tension steel ratio (pw=Ast/(ds*W)) =	0.0110	0.0076	0.0110			
	Comp. steel ratio (pcw=Asc/((D-dc)*W)) =	0.0025	0.0024	0.0025			
	σcs =	1.77	1.33	1.77	MPa		
	Mcr = (f'ct.f-σcs)*Ig/yt =	33.9	39.5	33.9	kNm	f'ct.f =	3.79 MPa
	Cracked κc =	0.504	0.377	0.504		OS RCB-1.1(1) Fig 5.7	
	Icr=κc*W*ds³/12=	721	480	721	x10⁶ mm⁴		
	Ief.max =	2250	2250	2250	x10⁶ mm⁴	Cl 8.5.3.1	
	Ief =	744	594	744	x10⁶ mm⁴		
	Iav =	Interior/Fixed - (M + (L + R) / 2) / 2 = 669x10⁶ mm⁴					
	Ratio Iuncrk/Iav =	3.56	Iuncrk at position x				
	kcs = [2-1.2*(Asc/Ast)] ≥ 0.8 =	1.600	kcs at position x			Cl 8.5.3.2	

Deflection summary - Manual values

			Manual Values:		
Gross δdl.g.imm =	1.7	mm	Gross δdl.g.imm =	1.71	mm
Gross δll.g.imm =	0.0	mm	Gross δll.g.imm =	0.00	mm
Short term δ.short = [δdl.g+ψs*δll.g]*Iuncrk/Ief =	6.1	mm	Short term δdl.short =	6.1	mm
Sustained δ.sus = [δdl.g+ψl*δll.g]*Iuncrk/Ief =	6.1	mm	Short term δll.short =	0.0	mm
Long term δ.long = kcs*δ.sus =	9.7	mm	Long term δdl.long =	9.7	mm
Incremental δ.inc = δ.long + δll.short =	9.7	mm	Long term δll.long =	0.0	mm
			Total δdl.total =	15.8	mm
Total δ.total = δ.short + δ.long =	15.8	mm	Span / 316		

Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]

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Section: (Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa
Defl'n: δ.dl = 9.7mm, δ.ll = 0.0mm, δ.inc = 9.7mm, δtotal = 15.8mm (span = 5000mm, L/316) **OK**

Results - Pos 16 of 31 at 2500mm (3) Average Ief based on (Iav) = (M + (L + R) / 2) / 2

Position mm	Top Reinf't	Spacing mm	Bot. Reinf't	Spacing mm	Shear Ligs	Legs	M* kNm	V* kN	N* kN	T* kNm	Ratio M	Ratio V	Saved / Comments / Errors
0	9-N20	112	2-N20	896	*		-217.6	261.1			0.93	0.42	[Ouside dv] [Reo.δ] [3 iter.s]
218	7-N20	149	2-N20	896	*		-163.2	238.3			0.95	0.39	[Ouside dv] [3/4xM*-] [2 iter.s]
232	7-N20	149	2-N20	896	N12-225 cts	5	-159.8	236.8			0.93	0.38	[dvLeft] [2 iter.s]
459	6-N20	179	2-N20	896	N12-225 cts	5	-108.8	213.2			0.78	0.43	[1/2xM*-] [3 iter.s]
500	6-N20	179	2-N20	896	N12-225 cts	5	-100.1	208.9			0.51	0.41	[3 iter.s]
637	5-N20	224	2-N20	896	N12 spacer	2	-72.5	194.6			0.66	0.99	[1/3xM*-] [2 iter.s]
732	4-N20	299	2-N20	896	N12 spacer	2	-54.4	184.6			0.67	0.96	[1/4xM*-]
1000	4-N20	299	2-N20	896	N12 spacer	2	-8.7	156.6			0.79	0.70	
1057	2-N20	896	4-N20	299	N12 spacer	2	0.0	150.7			0.00	0.68	[M*=0 point]
1250	2-N20	896	4-N20	299	N12 spacer	2	27.2	130.5			0.35	0.55	[1/4xM*]
1322	2-N20	896	4-N20	299	N12 spacer	2	36.3	123.1			0.46	0.54	[1/3xM*]
1479	2-N20	896	4-N20	299	N12 spacer	2	54.4	106.6			0.70	0.52	[1/2xM*]
1500	2-N20	896	7-N20	149	N12 spacer	2	56.6	104.4			0.93	0.40	[3 iter.s]
1778	2-N20	896	5-N20	224	N12 spacer	2	81.6	75.4			0.77	0.37	[3/4xM*] [2 iter.s]
2000	2-N20	896	9-N20	112	N12 spacer	2	95.7	52.2			0.91	0.20	[3 iter.s]
2500	2-N20	896	6-N20	179	N12 spacer	2	108.8	0.0			0.81	0.00	[Max.δ] [Reo.δ] [V*=0] [3 iter.s]
3000	2-N20	896	9-N20	112	N12 spacer	2	95.7	-52.2			0.91	0.20	[3 iter.s]
3222	2-N20	896	5-N20	224	N12 spacer	2	81.6	-75.4			0.77	0.37	[3/4xM*] [2 iter.s]
3500	2-N20	896	7-N20	149	N12 spacer	2	56.6	-104.4			0.93	0.40	[3 iter.s]
3521	2-N20	896	4-N20	299	N12 spacer	2	54.4	-106.6			0.70	0.52	[1/2xM*]
3678	2-N20	896	4-N20	299	N12 spacer	2	36.3	-123.1			0.46	0.54	[1/3xM*]
3750	2-N20	896	4-N20	299	N12 spacer	2	27.2	-130.5			0.35	0.55	[1/4xM*]
3943	2-N20	896	4-N20	299	N12 spacer	2	0.0	-150.7			0.00	0.68	[M*=0 point]
4000	4-N20	299	2-N20	896	N12 spacer	2	-8.7	-156.6			0.79	0.70	
4268	4-N20	299	2-N20	896	N12 spacer	2	-54.4	-184.6			0.67	0.96	[1/4xM*-]
4363	5-N20	224	2-N20	896	N12 spacer	2	-72.5	-194.6			0.66	0.99	[1/3xM*-] [2 iter.s]
4500	6-N20	179	2-N20	896	N12-225 cts	5	-100.1	-208.9			0.51	0.41	[3 iter.s]
4541	6-N20	179	2-N20	896	N12-225 cts	5	-108.8	-213.2			0.78	0.43	[1/2xM*-] [3 iter.s]
4768	7-N20	149	2-N20	896	N12-225 cts	5	-159.8	-236.8			0.93	0.38	[dvRight] [2 iter.s]
4782	7-N20	149	2-N20	896	*		-163.2	-238.3			0.95	0.39	[Ouside dv] [3/4xM*-] [2 iter.s]
5000	9-N20	112	2-N20	896	*		-217.6	-261.1			0.93	0.42	[Ouside dv] [Reo.δ] [3 iter.s]

Section: **(Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa**

Creep and Shrinkage

Environment =	I	(A)rid, (I)nterior, (T)emperate, t(R)opical - Cl 3.1.7.2	
Drying shrinakge =	S	(S)tandard,(T)esting	
Time in days after commencing drying (t) =	10950	days (30 years)	
Time in days of loading after commencing drying (τ) =	28	in days	Cl 3.1.8.3
Manual design shrinkage =	N	(Y)es,(N)o	
Manual design creep =	N	(Y)es,(N)o	

Shrinkage - Cl 3.1.7

Gross area (Ag) =	300000	mm^2	
Exposed perimeter (ue = 2*W+2*D) =	2600	mm	Cl 1.7
Manual hypothetical thickness =	N	(Y)es,(N)o	
Hypothetical thickness (th = 2*Ag/ue) =	231	mm	Cl 1.7
Concrete strength (f'c) =	40	MPa	
$\alpha 1 = 0.8+1.2*\exp(-0.005*th)$ =	1.179		Fig 3.1.7.2
$k1 = \alpha 1*t^{0.8}/(t^{0.8}+0.15*th)$ =	1.155		Fig 3.1.7.2
Environment (k4) =	0.65	Interior	Cl 3.1.7.2
Final drying basic shrinkage strain ($\varepsilon csd.b*$) =	800	$x10^{-6}$	Standard
Final autogenous strain ($\varepsilon cse* = (0.07*f'c-0.5)*50x10^{-6}$) =	115	$x10^{-6}$	Eq 3.1.7.2(3)
Autogenous shrinkage strain (εcse) =	115	$x10^{-6}$	Eq 3.1.7.2(2)
Basic drying shr. strain $\varepsilon csd.b = (0.9-0.005*f'c)*\varepsilon csd.b*$ =	560	$x10^{-6}$	Eq 3.1.7.2(5)
$\varepsilon csd = k1*k4*\varepsilon csd.b$ =	420	$x10^{-6}$	Eq 3.1.7.2(4)
$\varepsilon cs = \varepsilon cse + \varepsilon csd$ =	535	$x10^{-6}$	Eq 3.1.7.2(1)
Final design shrinkage strain εcs =	535	$x10^{-6}$ ±30%	
Rounded final design shrinkage strain εcs =	540	$x10^{-6}$ ±30%	Cl 8.5.3.1
Design shrinkage strain εcs =	540	$x10^{-6}$ ±30%	Calculated value

Creep - Cl 3.1.8

Basic creep coefficient ($\phi cc.b$) =	2.80		Table 3.1.8.2
$\alpha 2 = 1.0+1.12*\exp(-0.008*th)$ =	1.177		Fig 3.1.8.3
$k2 = \alpha 2*t^{0.8}/(t^{0.8}+0.15*th)$ =	1.153		Fig 3.1.8.3
$k3 = 2.7/[1+\log(\tau)]$ =	1.10		
Environmental factor (k4) =	0.650		Cl 3.1.8.3
$\alpha 3 = 0.7/(k4*\alpha 2)$ =	0.915		
k5 =	1.00		
Constant sustained stress (σo) =	0.0	MPa	Cl 3.1.8.1
0.45*fcmi =	19.5	MPa	
Non-linear creep coefficient when $\sigma o \le 0.45*fcmi$ (k6) =	1.000		Cl 3.1.8.3
$\phi cc = k2*k3*k4*k5*k6*\phi cc.b$ =	2.316	±30%	Eq 3.1.8.3
Adopted ϕcc =	2.316	±30%	Calculated value

Shrinkage Deflection - Determined midspan [WRH Ch 9.3]

Ast at midspan =	1885 mm^2	(Area of steel must be pre-set in the Defl tab)	
Depth to tens. steel (ds) =	248 mm		
Asc at midspan =	628 mm^2		
κsh = 1.15*εcs/ds*(1-(Asc/Ast) =	1669 $x10^{-9}$ mm^{-1}		WRH - Eq 9.14
βsh =	0.063 For span type fixed end beam		
Span (L) =	5000 mm		
Shrinkage δsh = κsh*βsh*L^2 =	2.6 mm (Uniform beams only)		WRH - Eq 9.15

Creep Deflection - Determined midspan [WRH Ch 9.3]

Tensile steel ratio (pn) =	0.047	(Deflections must have been calculated with analysis)
kcsi =	1.600	(From Analysis and Defl tab)
δe,sus = δ.short = [δdl.g+ψs*δll.g]*i/ieff =	6.1 mm	
δcr = ϕcc*(1-6*pn*(1-6*pn))/(3*(1+Asc/Ast))*δe,sus) =	2.8 mm	WRH - Eq 9.21

Deflection summary for calculated creep and shrinkage

Gross δdl.g.imm =	1.7 mm	
Gross δll.g.imm =	0.0 mm	
Short term δ.imm ([δdl.g + ψs * δll.g]*i/ieff) =	6.1 mm	
Permanent δ.long (δsh + δcr) =	5.4 mm	
Total δ =	11.5 mm	= Span / 433

Slab crack control for shrinkage and temperature effects - Cl 9.5.3 (Total area) (Not applicable for beam)

Design thickness = 300 mm
Shrinkage design thickness = 300 mm
Average prestress (σcp) = 0.00 MPa

		Cts for secondary reinforcement		
		N10	N12	N16
Unrestrained slabs Cl 9.5.3.3				
Ast.min = (1.75-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-
Restrained slabs (internal) Cl 9.5.3.4 (a)				
Ast.min (minor) = (1.75-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-
Ast.min (moderate) = (3.50-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-
Ast.min (strong) = (6.00-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-
Restrained slabs (A1 & A2 - external) Cl 9.5.3.4 (b)				
Ast.min (moderate) = (3.50-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-
Ast.min (strong) = (6.00-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-
Restrained slabs (B1, B2, C1, C2 - external) Cl 9.5.3.4 (c)				
Ast.min = (6.00-2.5*σcp)*b*Dx10⁻³ =	0 mm²/m	-	-	-

Note: Tabled values for reinf't provided to each face of the element.

Section:	**(Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa**	
Reinf't:	**2-N20 top, 6-N20 bottom**	
Defl'n:	**ds = 248mm > d.min = 152mm (D.min = 192mm)**	**OK**
Defl'n (Incr.):	**ds = 248mm > d.min.i = 168mm (D.min.i = 208mm)**	**OK**

Deemed to comply deflection - Cl 8.5.4 (Internal span)

Span (L) =	5000	mm
Depth =	300	mm
Width =	1000	mm
Span type =	I	(i)nternal, (E)nd, (S)imple
Spans =	2	(2),(>2)

Loading

Dead load S.Wt (wdl) =	7.50	kN/m			
Superimposed dead load (wsdl) =	1.00	kN/m			
Live load (wll) =	3.00	kN/m			
Load type =	N	(N)ormal, (S)torage			
Short term LL factor (ψs) =	0.7	(Normal load use)			
Long term LL factor (ψl) =	0.4				
Fd* = 1.2*(wdl+wsdl)+1.5*wll =	14.70	kN/m			
Interior span +ve = M+* = Fd*.*Ln²/16 =	23.0	kNm/m	Cl 6.10.2.3(b)	Ductility Class (btm) =	N
Interior span -ve = M-* = Fd*.*Ln²/11 =	-33.4	kNm/m	Cl 6.10.2.2(b)	Ductility Class (top) =	N

Beam deflection (Deemed to Comply) - Cl 8.5.4

Tensile steel area (Ast) =	1885	mm²		Depth to tensile steel (ds) =	248	mm
Compr. steel area (Asc) =	628	mm²		Depth to comp. steel (dc) =	42	mm
kcs = 2-1.2*(Asc/As) ≥ 0.8 =	1.60	Cl 8.5.3.2		p = Ast/(bef*ds) =	0.00760	(Midspan)
				pc = Asc/(bef*ds) =	0.00253	(Midspan)
Effective design load (Fd.ef) =	26.12	kN/m	Cl 8.5.4(a)	dc/ds =	0.16935	
Incremental (Fd.ef.inc) =	17.62	kN/m	Cl 8.5.4(b)	n = Es / Ec =	6.01	
				NA =	0.256	
od. of elasticity (Ec = p¹·⁵*(0.024*√fcmi+0.12)) =	33251	MPa ± 20%	Cl 3.1.2	Cracked NA =	63.6	mm

β = bef/bw ≥ 1 =	1.00		Cl 8.5.4
k1 = (5-0.04*f'c)*p+0.002 ≤ 0.1/β²ᐟ³ =	0.028		p ≥ 0.001*fc¹ᐟ³/β²ᐟ³ = 0.0034
k2 =	0.0039		5/384 Simple, 1.5/384 Interior span, 2.4/384 End span
Lef/Δ =	250		Table 2.3.2
Lef/Δ.inc =	500		Table 2.3.2
d.min = Lef/[k1*(Δ/Lef)*bef*Ec/(k2*Fd.ef)]¹ᐟ³) =	152	mm	Eq 8.5.4
d.min. (inc) =	168	mm	
D.min. =	192	mm	
D.min. (inc) =	208	mm	

Section:	**(Analysis Beam [Span 1 (M1), 1.2 DLxg + 1.5 LL]) 300mm (D) x 1000mm (W) beam, f'c=40MPa**	
Reinf't:	**2-N20 top, 6-N20 bottom**	
Defl'n:	**ds = 248mm > d.min = 138mm (D.min = 178mm)**	**OK**
Defl'n (Incr.):	**ds = 248mm > d.min.i = 154mm (D.min.i = 194mm)**	**OK**

Deemed to comply deflection - Cl 9.4.4.1 (Not applicable - refer BeamDefl tab)

Span (Ln) =	5000	mm
Slab thickness =	300	mm
Span type =	I	(i)nternal, (E)nd, (S)imple
Spans =	2	(2),(>2)

Loading

Dead load S.Wt (wdl) =	7.50	kPa		
Superimposed dead load (wsdl) =	1.00	kPa		
Live load (wll) =	3.00	kPa		
Load type =	N	(N)ormal, (S)torage		
Short term LL factor (ψs) =	0.7	(Normal load use)		
Long term LL factor (ψl) =	0.4			
Fd* = 1.2*(wdl+wsdl)+1.5*wll =	14.70	kPa		
Interior span +ve = M+* = Fd*.*Ln²/16 =	23.0	kNm/m	Cl 6.10.2.3(b)	Ductility Class (btm) = N
Interior span -ve = M-* = Fd*.*Ln²/11 =	-33.4	kNm/m	Cl 6.10.2.2(b)	Ductility Class (top) = N

Slab deflection (Deemed to Comply)

Tensile steel area (Ast) =	1885	mm²		Depth to tensile steel (ds) =	248	mm
Compr. steel area (Asc) =	628	mm²		Depth to comp. steel (dc) =	42	mm
kcs = 2-1.2*(Asc/Ast) ≥ 0.8 =	1.60	Cl 8.5.3.2		p = Ast/(W*ds) =	0.00760	(Midspan)
				pc = Asc/(W*ds) =	0.00253	(Midspan)
Effective design load (Fd.ef) =	26.12	kPa	Cl 9.3.4.1(a)	dc/ds =	0.16935	
Incremental (Fd.ef.inc) =	17.62	kPa	Cl 9.3.4.1(b)	n = Es/Ec =	6.01	
od. of elasticity (Ec = p$^{1.5}$*(0.024*Vfcmi+0.12)) =	33251	MPa ± 20%	Cl 3.1.2	Neutral axis (NA) =	0.256	
				Cracked NA =	63.6	mm

k3 =	1.00		1.0 for one-way spab
k4 =	2.10		1.4 Simple, 2.1 Internal span, 1.75 End span
Lef/Δ =	250		Table 2.3.2
Lef/Δ.inc =	500		Table 2.3.2
d.min = Lef/(k3*k4*[(Δ/Lef)*1000*Ec/Fd.ef]$^{1/3}$) =	138	mm	Eq 9.4.4.1
d.min (inc) =	154	mm	
D.min =	178	mm	
D.min (inc) =	194	mm	