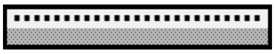
CONCRETE MEMBER V5.13

Engineering Building & Infrastructure Pty Ltd

Section:	**(slab (-)) 250mm thk slab, f'c=32MPa**	
Reinf't:	**N24-200 cts top, N24-200 cts bottom, ku = 0.22**	
Strength:	**(-ve M) M* = 19.9kNm < øMuo = 178.8kNm**	**OK (0.11)**
Cracking:	**fscr = 30MPa < Fscr = 225MPa & fscr1 = 35MPa < Fscr1 = 400MPa, crack width = 0.0mm**	**OK (0.09,0.13)**
Ast.min:	**Ast.min = 388mm² < Ast = 2262mm² (Minimum of Deemed and actual)**	**OK (0.17)**

Geometry Interior environment with εcsd.b*=800x10⁻⁶, εcs*=520x10⁻⁶ L/D ratio = 32.0

Concrete strength (f'c) =	32	MPa
Depth (D) =	250	mm
Web width (W) =	S	mm, (S)lab
Slab type =	T	(O)ne way, Two way & (C)ols, (T)wo way & walls, Two-way (F)ooting



Comp.
Tension

Span (L) =	8000	mm (for moment resisting width)	Fully enclosed =	N	(Y)es,(N)o
			Formwork =	S	(S)tandard,(R)igid
Concrete weight =	25.0	kN/m³	Exposure top =	A2	Table 4.10.3.2
S.Wt =	6.25	kN/m	Exposure bottom =	A2	Table 4.10.3.2
Gross area (Ag) =	250000	mm²			

Design actions

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

	Manual values			**Manual**	**Units**
Design (M*) =	-19.9	kNm/m	**M***	-19.9	kNm
Design (Ms1*, Ψs=1) =	D	kNm/m	**Ms1*(Ψs=1)**	-14.9	kNm
Design (Ms*, Ψs) =	D	kNm/m	**Ms*(Ψs)**	-12.9	kNm
			Ast req'd	228	mm²
Ms1* & Ms* estimated - To be verified			**Ast**	2262	mm²
			Reinf't req'd	N24-1985	

Reinforcement

Bottom reinf't =	**N24-200 cts**		**Top reinf't =**	**N24-200 cts**	
Bar size =	24	mm	Bar size =	24	mm
Bar cts/No/mm² =	200	mm	Bar cts/No/mm² =	200	mm
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 (Asc) =	2262	mm²/m	Steel area (Ast) =	2262	mm²/m
Bottom cover to steel =	30	mm	Top cover to steel =	30	mm
Depth to bottom steel layer (ds.max) =	208	mm	Depth to top steel layer =	42	mm
Depth to bottom steel (ds) =	208	mm	Depth to top steel =	42	mm
D-ds =	42	mm	D-ds =	208	mm
No. bars =	5.0	No.	No. bars =	5.0	No.
Bar centres =	200	mm	Bar centres =	200	mm
Max. bars per layer =	10		Max. bars per layer =	10	
Layers required =	1		Max. bars per 2nd layer =	10	
			Layers required =	1	

Strength in -ve bending for slabs - Cl 9.1

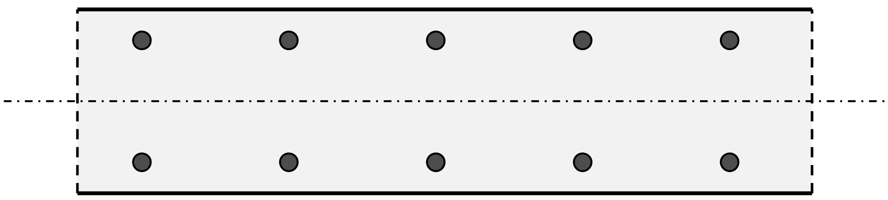
Design width (W) =	1000	mm	Design flange (bef) =	1000	mm
Tensile steel area (As) =	2262	mm²/m	Depth to tensile (ds) =	208	mm
Comp. steel area (Ac) =	2262	mm²/m	Depth to comp. steel (dc) =	42	mm
Ultimate Moment (Mu) =	210.3	kNm/m	ku =	0.218	
Design capacity (øMuo) =	178.8	kNm/m	Factor (ø) =	0.850	Table 2.2.2
			As.min =	388	mm²/m

Crack control in -ve bending for slabs - Cl 9.4

Steel stress (σscr1) =	35	MPa	Max. stress (σscr1.max) =	400	MPa	OK (0.09)
Steel stress (σscr) =	30	MPa	Max. stress (σscr.max) =	225	MPa	OK (0.13)
Max. crack width (w'max) =	0.3	mm	Calculated crack width =	0.05	mm	

Reinf't:	**5.0-N24 top tensile**
	5.0-N24 bottom compression
Ligs:	**2 legs N12-125 cts**

Preview



Strength in Bending

alpha (α_2 = 0.85-0.0015*f'c) =	0.802 ($\alpha_2 \geq 0.67$)	Eq 8.1.3(1)	M* =	-19.9 kNm	
gamma (γ = 0.97-0.0025*f'c) =	0.890 ($\gamma \geq 0.67$)	Eq 8.1.3(2)			
Max. concrete strain (εu) =	0.0030		tf =	0 mm	
Max. steel strain (εsy = fsy/Es) =	0.0025		tf.min (rect. design) =	0.0 mm	LC Eq 3.7(9)
ku.b = $\varepsilon u/(\varepsilon u$+fsy/Es) =	0.545				

Positive (+ve) bending			**Negative (-ve) bending**		
Steel ratio (p) =	0.0109		Steel ratio (p) =	0.0109	
Compression equation valid (Rect) =	Yes				
Bar =	24 mm		Bar =	24 mm	
Centres =	200.0 mm		Centres =	200.0 mm	
Tensile cover =	30 mm		Tensile cover =	30 mm	
Ast =	2262 mm²/m		Ast =	2262 mm²/m	
Asc =	2262 mm²/m		Asc =	2262 mm²/m	
ds =	208 mm		ds =	208 mm	
ds.max =	208 mm		ds.max =	208 mm	
dc =	42 mm		dc =	42 mm	
b =	1000 mm		b =	1000 mm	

Compression steel at yield (γkud in flange if T or L)			**Compression steel at yield**		
ku =	0.000	WRH Eq 4.49	ku =	0.000	WRH Eq 4.49
εst =	0.0000	WRH Eq 4.50	εst =	0.0000	WRH Eq 4.50
εsc =	0.0000	WRH Eq 4.51	εsc =	0.0000	WRH Eq 4.51
Valid ($\varepsilon st,\varepsilon sc \geq \varepsilon sy$) =	0 (0=No, 1=Yes)		Valid ($\varepsilon st,\varepsilon sc \geq \varepsilon sy$) =	0 (0=No, 1=Yes)	
Mu =	0.0 kNm	WRH Eq 4.53	Mu =	0.0 kNm	WRH Eq 4.53

Compression steel not at yield (γkud in flange if T or L)			**Compression steel not at yield**		
u1 =	0.04761	WRH Eq 4.56	u1 =	0.04761	WRH Eq 4.56
u2 =	-0.05768	WRH Eq 4.57	u2 =	-0.05768	WRH Eq 4.57
ku =	0.218	WRH Eq 4.55	ku =	0.218 mm	WRH Eq 4.55
kud (with Asc, if in flange) =	45.2 mm	Comp. steel in comp.	kud (with Asc) =	45.2 mm	Comp. steel in comp.
γ.kud (if in flange) =	40.3 mm		γ.kud =	40.3 mm	
Compression steel in compression =	1 (0=No, 1=Yes)		Compression steel in compression =	1 (0=No, 1=Yes)	
kud =	45.2 mm		kud =	45.2 mm	
Mu (if γkud in flange) =	210.3 kNm	WRH Eq 4.58	Mu =	210.3 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) =	45.2	mm	
Mu (if γ.kud in web) =	194.2	kNm	RCB - Eq 3.78

Design kud =	45.2	mm		Design kud =	45.2	mm	
γ.kud =	40.3	mm		γ.kud =	40.3	mm	
Comp. bar in compr. =	1	(0=No, 1=Yes)		Comp. bar in compr. =	1	(0=No, 1=Yes)	
εs =	0.0108			εs =	0.0108		
Ts =	1131.0	kN	500.0 MPa	Ts =	1131.0	kN	500.0 MPa
εc =	0.0002			εc =	0.0002		
Cs =	97.4	kN	43.1 MPa	Cs =	97.4	kN	43.1 MPa
Cc = Cw =	1033.5	kN		Cc =	1033.5	kN	
Cf =	0.0	kN					
T - Cs - Cc =	0.0	kN		T - Cs - Cc =	0.0	kN	
ku =	0.218			ku =	0.218		
Muo =	210.3	kNm	WRH Eq 4.58	Muo =	210.3	kNm	
Reinf't ductility class =	N	(N)ormal,(L)ow		Reinf't ductility class =	N	(N)ormal,(L)ow	
kuo = kud/ds.max =	0.218			kuo =	0.218		
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) =	178.8	kNm		Capacity (øMuo) =	178.8	kNm	
Ast req'd =	0	mm^2/m		Ast req'd =	228	mm^2/m	
Factor - øMuo/(ø*fsy*Ast*ds) =	0.894			Factor - øMuo/(ø*fsy*Ast*ds) =	0.894		

Minimum strength requirements - Slab supported by walls - Cl 9.1.1

Positive (+ve) bending				Negative (-ve) bending			
Uncracked neutral axis (uk) =	0.500		OS RCB-1.1(1) Fig 5.3	Uncracked (uk) =	0.500		OS RCB-1.1(1) Fig 5.3
Depth of uncracked neutral axis NA = uk*D =	125	mm		Depth of uncracked NA =	125	mm	
Uncracked factor for stiffness (uκ) =	1.136			Uncracked (uκ) =	1.136		
Iuncrk = uκ*W*D^3/12 =	1479	$\times10^6\ mm^4$	OS RCB-1.1(1) Eq 7.2(2)	Iuncrk = uκ*W*D^3/12 =	1479	$\times10^6\ mm^4$	OS RCB-1.1(1) Eq 7.2(2)
Zb = Iuncrk/(D-NA) =	11835	$\times10^3\ mm^3$		Zt = Iuncrk/NA =	11835	$\times10^3\ mm^3$	
Char. flex. tens. strength (f'ct.f = 0.6*√f'c) =	3.39	MPa	Cl 3.1.1.3				
(Muo)min = 1.2*Zb*fct.f =	48.2	kNm	Eq 8.1.6.1(1)	(Muo)min = 1.2*Zt*fct.f =	48.2	kNm	Eq 8.1.6.1(1)
Ast.min (Muo)min =	474	mm^2/m		Ast.min (Muo)min =	474	mm^2/m	
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)		Slab supported by walls - Cl 9.1.1
αb =	0.190			αb =	0.190		
Deemed to comply Ast.min =	388	mm^2/m		Deemed to comply Ast.min =	388	mm^2/m	
Actual bar cts =	200	mm		Actual bar cts =	200	mm	
Max. bar cts =	300	mm	OK Cl 9.5.1(b)	Max. bar cts =	300	mm	OK Cl 9.5.1(b)
Side face reinforcement =	Not applicable to slabs						

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Crack control of slabs - Cl 9.5 Interior environment with $\varepsilon csd.b^* = 800 \times 10^{-6}$, $\varepsilon cs^* = 520 \times 10^{-6}$

Uncracked NA =	125 mm	OS RCB-1.1(1) Fig 5.3
Uncracked uκ =	1.14	
Iuncrk = uκ*W*D³/12 =	1479 $\times 10^6$ mm⁴	OS RCB-1.1(1) Eq 7.2(2)

Positive (+ve) bending			**Negative (-ve) bending**		
Tension on bottom face =	N24 bars - Not applicable		Tension on top face =	N24 bars	
Tensile area of conc. prior to cracking (Act) =	125000 mm²		Reverse bending (Act) =	125000 mm²	
Tensile area of conc. (mid-depth) (Actm) =	125000 mm²		Reverse bending (Actm) =	125000 mm²	Between mid depth and tensile Cl 8.2.4.2.2(e)
Tensile area of conc. (Uncracked) (Actn) =	125000 mm²		Reverse bending (Uncracked) (Actn) =	125000 mm²	Between uncracked na and tensile Cl 8.2.4.4-AS 5100.5
Cracked neutral axis (k) =	0.316	OS RCB-1.1(1) Fig 5.7	Cracked (k) =	0.316	OS RCB-1.1(1) Fig 5.7
Depth of cracked NA = k*ds =	66 mm		Depth of cracked NA =	66 mm	
Cracked factor for stiffness (κ) =	0.544		Cracked (κ) =	0.544	
Icr = κ*W*ds³/12 =	408 $\times 10^6$ mm⁴		Icr = κ*W*ds³/12 =	408 $\times 10^6$ mm⁴	
Steel tensile stress (σscr1) =	35 MPa		Steel tensile stress (σscr1) =	35 MPa	
Max. tensile stress (bar) (σscr.max) =	175 MPa	Table 9.5.2.1(A)	bar σscr.max =	175 MPa	Table 9.5.2.1(A)
Max. tensile stress (cts) (σscr.max) =	225 MPa	Table 9.5.2.1(B)	cts σscr.max =	225 MPa	Table 9.5.2.1(B)
Max. tensile stress (σscr.max) =	225 MPa		σscr.max =	225 MPa	
	OK (0.13)			OK (0.13)	
Outermost steel tensile stress (σscr) =	30 MPa		Outermost steel tensile stress (σscr) =	30 MPa	
Max. tensile stress (σscr1.max=0.8*fsy) =	400 MPa	Cl 9.5.2.1	σscr1.max= 0.8*fsy =	400 MPa	Cl 9.5.2.1
	OK (0.09)			OK (0.09)	
AS 3600 - 2009					
Max. tensile stress (bar) (σscr.max) =	210 MPa	Table 9.4.1(A)	bar σscr.max =	210 MPa	Table 9.4.1(A)
Max. tensile stress (cts) (σscr.max) =	240 MPa	Table 9.4.1(B)	cts σscr.max =	240 MPa	Table 9.4.1(B)
Max. tensile stress (σscr.max) =	240 MPa		σscr.max =	240 MPa	

Crack control of slabs by calculation of crack widths - Cl 9.5

Design shrinkage strain (εcs) =	520	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) =	2.747		Cl 3.1.8.3	(Refer Creep/Shrinkage)					
Eff. modular ratio (ne) = (1+øcc)*Es/Ec =	25.1		Cl 8.6.2.3						
hc.eff =min(2.5*(D-ds),(D-NA)/3,D/2) =	61.5	mm	Cl 8.6.2.3		hc.eff =	61.5	mm	Cl 8.6.2.3	
hc.effa = min(hc.eff,D-cracked NA) =	61.5	mm			hc.effa = min(hc.eff,cracked NA)	61.5	mm		
Eff. area of concrete in tension (Ac.eff) =	61455	mm^2	Cl 8.6.2.3		Ac.eff =	61455	mm^2	Cl 8.6.2.3	
Reinforcement ratio (Peff) = Ast/Ac.eff =	0.037		Cl 8.6.2.3		Peff =	0.037		Cl 8.6.2.3	
εdm.min = (εsm - εcm).min = 0.6*σscr/Es =	91	x10^{-6}	Eq 8.6.2.3(2)		εdm.min =	91	x10^{-6}	Eq 8.6.2.3(2)	
Strain in tensile steel = σscr/Es =	151	x10^{-6}	Eq 8.6.2.3(2)		σscr/Es =	151	x10^{-6}	Eq 8.6.2.3(2)	
Flexural tensile f'ct.f = 0.6*√(f'c) =	3.39	MPa	Cl 3.1.1.3						
Char. uniaxial tensile (fct = 0.6*f'ct.f) =	2.04	MPa	Cl 3.1.1.3						
Mean char. (fct.m=1.4*fct) =	2.85	MPa	Cl 3.1.1.3		Mean (fct.m=1.4*fct) =	2.85	MPa	Cl 3.1.1.3	
-0.6*fct.m/(Es*Peff) =	-232	x10^{-6}			-0.6*fct.m/(Es*Peff) =	-232	x10^{-6}		
1+ne*peff =	1.923				1+ne*peff =	1.923			
Design shrinkage strain εcs =	520	x10^{-6} ±30%			Design shrinkage strain εcs =	520	x10^{-6} ±30%		
.1 = σscr/Es-0.6*fct/(Es*Peff)*(1+n*Peff))+ecs =	224	x10^{-6}			εdm.1 =	224	x10^{-6}		
εdm = εsm - εcm = max(εdm.min,εdm.1) =	224	x10^{-6}	Eq 8.6.2.3(2)		εdm = εsm - εcm =	224	x10^{-6}	Eq 8.6.2.3(2)	
Axial (N*) =	0	kN (Tension -ve)							
Cross section area (Area) =	250000	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) =	-54	x10^{-6}			εt =	195	x10^{-6}		
Tensile strain at bottom (εb) =	195	x10^{-6}			εb =	-54	x10^{-6}		
ε1 = max(εt,εb) =	195	x10^{-6}	Cl 8.6.2.3		ε1 = max(εt,εb) =	195	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) =	200	mm			Bar centres (cts) =	200	mm		
Clear cover to long. reinf't (c = cover+ligs) =	42	mm			Clear cover (c = cover+ligs) =	42	mm		
Max. bar spacing (5*(c+0.5*db)) =	270	mm			Maximum bar spacing (cts) =	270	mm		
sr.max.1 = 1.3*(D-kd) =	240	mm	Eq 8.6.2.3(3)		sr.max.1 =	240	mm	Eq 8.6.2.3(3)	
sr.max.2 = 3.4*c+0.3*k1*k2*db/Peff) =	221	mm	Eq 8.6.2.3(3)		sr.max.2 =	221	mm	Eq 8.6.2.3(3)	
sr.max = min(srmax.1,sr.max.2) =	221	mm			sr.max = min(srmax.1,sr.max.2) =	221	mm		
Max. final crack spacing (sr.max) =	221	mm	Eq 8.6.2.3(1)		Max. final crack spacing (sr.max) =	221	mm	Eq 8.6.2.3(1)	
Char. maximum crack width (w'max) =	0.3	mm	Cl 9.5.2.1						
Calculated crack width (w) =	0.050	mm	Eq 8.6.2.3(3)	OK	w =	0.050	mm	Eq 8.6.2.3(3)	OK

Primary reinforcement

Minimum for strength =	228	mm²/m	N24-1985
Deemed-to-comply minimum =	388	mm²/m	N24-1165
Actual minimum for øMuo =	474	mm²/m	N24-950

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Section:	**(slab (-)) 250mm thk slab, f'c=32MPa**	
Reinf't:	**5.0-N24 top tensile**	
	5.0-N24 bottom compression	
	No ligs required	
Strength:	**V* = 0.0kN ≤ ks*øVuc = 276.0kN, V* = 0.0kN ≤ øVu = 276.0kN - No shear ligs required**	**OK (0.00)**
	øVuc = 276.0kN, øVu.min = 379.5kN, øVus = 0.0kN	
Torsion:	**T* = 0.0kNm ≤ øTus = 155.1kNm - No torsion ligs required**	**OK (0.00)**
	0.25øTcr = 8.2kNm, øTcr = 32.7kNm	
Combined:	**Vcomb* = 0.0kN ≤ øVu.max = 895.6kN**	**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/m	Design shear (V*) =	0	kN/m
Manual moment (M*) =	-19.9	kNm/m	Design moment (M*) =	-20	kNm/m (+ve or -ve)
Manual section max. moment (Mmax*) =	-19.9	kNm/m	Section Max. (Mmax*) =	-20	kNm/m (+ve or -ve)
			(blank = M*)		
Axial (N*) =	0	kN (Tension -ve) (Refer column for bending/axial capacity)			
Torsion (T*) =	0	kNm			
do (+ve bending) =	208	mm	dv (+ve bending) =	187	mm
do (-ve bending) =	208	mm	dv (-ve bending) =	187	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))) =	32.7	kNm	Cl 8.2.1.2(2) - Refer below	
0.25*øTcr =	8.2	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 Note - V* ≤ øVu.min, Toggle the increase spacing option - Refer to the right

Description =	**2 legs N12-125 cts**		Ignore ligs =	N	(Y)es,(N)o
Lig size =	12	mm	Leg area (Asl) =	113	mm²
Spacing (s) =	125	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 =	250	mm
Fitment class =	N	(N)ormal,(L)ow	s.max.t =	250	mm
Angle between shear & tensile reinf't (αv) =	90.0	°			

Shear spacing - Cl 8.3.2.2

Max. cts = 0.5*D =	125	mm < 300mm	Ligs area (Asv) =	226	mm²
Max. cts =	125	mm	Adopted ligs area (Asv) =	0	mm²
			Asv.min =	113	mm² (2-8.5mm dia.)

Transverse spacing - Cl 8.3.2.2

Transverse spacing =	500	mm	Waive max. spacing =	N	(Y)es,(N)o
Max.transverse spacing =	250	mm	Waive transv. spacing =	N	(Y)es,(N)o

Torsion spacing - Cl 8.3.3

			Lig area (Asw) =	113	mm²
Max. cts = 0.12*uh =	265	mm < 300mm	Adopted ligs area (Asw) =	113	mm²
Max. cts =	265	mm	Asw.min =	57	mm² (8.5mm dia.)

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

Ast Req'd for M* =	228	mm^2 (0.5-N24)	Top bars in tension	
1-N24 =	452	mm^2		
Design (Ast) =	2262	mm^2 (N24-200 cts)	Design (Asc) =	2262 mm^2 (N24-200 cts)
Manual (Ast) =		mm^2 (blank = design)	Manual (Asc) =	mm^2 (blank = design)
Adopted (Ast) =	2262	mm^2 (N24-200 cts)	Adopted (Asc) =	2262 mm^2 (N24-200 cts)
Ast req'd for Mmax* =	228	mm^2 (0.5-N24)		

Concrete contribution to shear strength - Cl 8.2.4

do = D - dc.max =	208	mm	
Dist. to centroid of D/2 tensile reinf't (ds) =	208	mm	(Negative bending, top bars in tension)
Effective shear depth dv = max(0.72*D,0.9*ds) =	187	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) =	0.161		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	$x10^{-6}$	
General method for kv & θv - Cl 8.2.4.2			
Tensile area of concrete (Act) =	125000	mm^2	Area between mid depth and tensile fibre
Adopted (Ast) =	2262	mm^2	
Shear only $\varepsilon x1 \le 3000x10^{-6}$ =	117.5	$x10^{-6}$	Eq 8.2.4.2.2(1)
Shear only $-200x10^{-6} \le \varepsilon x2$ (where $\varepsilon x1 < 0$) ≤ 0 =	0.0	$x10^{-6}$	Eq 8.2.4.2.2(2)
Shear/torsion $\varepsilon xt1 \le 3000x10^{-6}$ =	-	$x10^{-6}$	Eq 8.2.4.2.3(1)
Shear/torsion $-200x10^{-6} \le \varepsilon xt2$ (where $\varepsilon xt1 < 0$) ≤ 0 =	-	$x10^{-6}$	Eq 8.2.4.2.3(2)
Long. mid-depth concrete strain (εx) =	117.5	$x10^{-6}$	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)) =	0.372		Eq 8.2.4.2(2)
Angle (θv) = (29+7000*εx) =	29.8	°	Eq 8.2.4.2(1)
Adopted kv & θv - General method - Cl 8.2.4.2			
kv =	0.372		
Angle of inclination of concrete comp. strut (θv) =	29.8	°	
Concrete contribution to shear stength - Cl 8.2.4.1			
Strength reduction factor (ø) =	0.7		Table 2.2.2(e)
kv =	0.372		General method - Cl 8.2.4.2
Effective flange (bv) =	1000	mm	
Effective shear depth (dv) =	187	mm	Cl 8.2.1.9
min(√f'c,8.0) =	5.66	MPa	
Vuc = kv*bv*dv*min(√f'c, 8.0) =	394.3	kN	Eq 8.2.4.1
D ≤ 300mm, ks =	1.00		
(ø=0.7)Vuc =	276.0	kN	
(ks=1.00)*(ø=0.7)Vuc =	276.0	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 =	276.0	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) =	29.8	°	General method - Cl 8.2.4.2	
Angle (θvr) =	0.520	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 =	276.0	kN		
Ultimate shear strength (øVu = øVuc) =	276.0	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 =	0.45	MPa	Cl 8.2.1.7
Asv.min = σmin*bv*s/fsy.f =	113	mm²	Eq 8.2.1.7
s.max = Asv*fsy.f/(σmin*bv) =	250	mm	
Vus.min = (Asv.min*fsy.f*dv/s)*cot(θv) =	147.8	kN	Eq 8.2.1.7
øVu.min = ø(Vuc + Vus.min) =	379.5	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)))]	1279.4	kN	Eq 8.2.3.3(1)
øVu.max = (ø=0.7)*Vu.max =	895.6	kN	Table 2.2.2(e)(ii)
øVus.max = øVu.max - øVuc =	619.5	kN	
Asv.max = øVus.max/(ø(fsy.f*dv/s)*cot(θv)) =	678	mm²	Eq 8.2.5.2(2)
when s =	125	mm	

Torsional strength of a beam without reinforcement - Cl 8.2.1.2

Outside perimeter of concrete cross-section (uc) =	2500	mm	Cl 8.2.1.2	
Area enclosed by oustide perimeter (Acp) =	250000	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))) =	46.7	kNm	Eq 8.2.1.2(2)	
øTcr =	32.7	kNm		
0.25*øTcr =	8.2	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) =	250	mm	Dxyo =	178	mm	xo =	178 mm
y = Max(D & W) =	1000	mm	Wxyo =	928	mm	yo =	928 mm

Perimeter of torsion ligs (centreline) (uh = 2*(xo + yo)) =	2212	mm	Cl 8.2.3.3	
Area enclosed by ligs (Aoh = xo*yo) =	165184	mm²	Cl 8.2.3.3	
Enclosed area by shear flow (Ao = 0.85*Aoh) =	140406	mm²	Cl 8.2.5.6	
Angle of inclination of concrete compression strut (θv) =	29.8	°		
Angle (θvr) =	0.520	rad		
Area of ligs (Asw) =	113	mm²		
Tus = 2.0*Ao*Asw*fsy.f*/s*cot(θv) =	221.6	kNm	Eq 8.2.5.6	
øTus =	155.1	kNm		
T*/øTus =	0.00			OK (0.00)

Minimum torsional reinforcement - Cl 8.2.5.5

Tcr =	46.7	kNm		
Asw.min1 = Asv.min/legs =	57	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)) =	6	mm²	Cl 8.2.5.5(b)(ii) & Eq 8.2.5.6	
Asw.min = max(Asw.min1, Asw.min2) =	57	mm²		
s.max1 = Asv*fsy.f/(σmin*bv) =	250	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 =	2374	mm	Cl 8.2.5.5(b)(ii) & Eq 8.2.5.6	
s.max = min(s.max1, s.max2) =	250	mm	N12-250 cts min.	
Tus.min = 2.0*Ao*Asw.min*fsy.f*/s*cot(θv) =	110.8	kNm	Eq 8.2.5.6	
øTu.min =	77.6	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 =	895.6	kN		
Vcombσ.limit = øVu.max/(bv*dv) =	4.784	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* =	-19.9	kNm	
V* =	0.0	kN	
T* =	0.0	kNm	
N* =	0.0	kN	
Steel area req'd (Ast.reqd) =	228	mm²	
Adopted area (Ast) =	2262	mm²	
Lever z = ds*(1-fsy*Ast/(2*α2*f'c*bw*ds)) =	186	mm	
Compressive steel area (Asc) =	2262	mm²	
uo = 0.92*uh =	2035	mm	
0.5*(V*+øVuc) =	138.0	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*) =	107.0	kN	Eq 8.2.8.2(1)
Ast = ΔTtd*/(øtt*fsy) =	252	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-N24		
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-N24		

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Section: **(slab (-)) 250mm thk slab, f'c=32MPa**
Reinf't: **N24-200 cts top, N24-200 cts bottom**
Defl'n: **δ.dl = 0.0mm, δ.ll = 0.0mm, δ.inc = 0.0mm, δtotal = 0.0mm (span = 8000mm, L/-)** **OK**
σcs for Interior environment with εcsd.b*=800x10⁻⁶, εcs*=520x10⁻⁶

Deflections - Cl 8.5.3 at

Concrete density (ρ) =	2400	kg/m³ Cl 3.1.3	Gross area (Ag) =	250000	mm²
Use fcmi? =	Y	(Y)es,(N)o	Uncr.g. neutral axis (NA) =	125	mm from top
fcmi =	35.0	MPa	Gross Stiffness (Ig) =	1302	x10⁶ mm⁴ (w/o reinf't)
Deflection at =	X	(M)anual, (C)ritical, (L)eft, Position (X) from analysis, (R)ight			
Position (x) =	0	mm	Steel Modulus (Es) =	200000	MPa Cl 3.2.2
Span type =	S		Mod. of elast. (Ec = ρ$^{1.5}$*0.043*√fcmi) =	29900	MPa ± 20% Cl 3.1.2
			Modular ratio (n = Es/Ec) =	6.689	

Deflection calculation - Interior/Fixed

	Left	At x	Right	Units		
Manual ultimate (M*) =				kNm	At x = 0 for cantilever	
Manual short-term serviceability (Ms*) =				kNm		
Analysis ultimate (M*) =	0.0	0.0	0.0	kNm	Red values of M*/Ms* manually input	
Analysis short-term serviceability (Ms*) =	0.0	0.0	0.0	kNm		
Top reinf't: Ast req'd =	0	0	0	mm²/m		
Design Ast =				mm²/m		
Ast =	0	0	0	mm²/m		
	-	-	-		Short term LL factor (ψs) =	0.00
Bottom reinf't: Ast req'd =	0	0	0	mm²/m	Long term LL factor (ψl) =	0.00
Design Ast =				mm²/m		
Ast =	0	0	0	mm²/m		
	-	-	-			
Uncracked NA =	125	125	125	mm	OS RCB-1.1(1) Fig 5.3	
Uncracked uκ =	1.000	1.000	1.000	x10⁶ mm⁴		
Iuncrk = uκ*W*D³/12 =	1302	1302	1302	x10⁶ mm⁴	OS RCB-1.1(1) Eq 7.2(2)	
Tensile steel (Ast) =	0	0	0	mm²/m		
Depth to ds =	208	208	208	mm (From comp. face)		
Comp. steel (Asc) =	0	0	0	mm²/m		
dc =	42	42	42	mm (From comp. face)		
Cracked κ =	0.000	0.000	0.000	mm	OS RCB-1.1(1) Fig 5.7	
Depth to cracked NA = κ*ds =	0.0	0.0	0.0	mm (From top)		
Use Comp. steel =	-	-	-			
yt =	125	125	125	mm (From tensile fibre)		
Design shrinkage strain εcs =	520	520	520	x10⁻⁶ ±30%	Interior env. refer Shrinkage tab	
W (slab) =	1000	1000	1000	mm		
Tension steel ratio (pw=Ast/(ds*W)) =	0.0000	0.0000	0.0000	pw < 0.005, Ief.max = 0.6*Ig		
Comp. steel ratio (pcw=Asc/((D-dc)*W)) =	0.0000	0.0000	0.0000			
σcs =	0.00	0.00	0.00	MPa		
Mcr = (f'ct.f-σcs)*Ig/yt =	35.4	35.4	35.4	kNm	f'ct.f =	3.39 MPa
Cracked κc =	0.000	0.000	0.000		OS RCB-1.1(1) Fig 5.7	
Icr=κc*W*ds³/12=	0	0	0	x10⁶ mm⁴		
Ief.max =	781	781	781	x10⁶ mm⁴, Ief = 0.6*Ig	Cl 8.5.3.1	
Ief =	0	0	0	x10⁶ mm⁴		

Iav = Interior/Fixed - (M + (L + R) / 2) / 2 = 0x10⁶ mm⁴ Error - Iav Invalid
Ratio Iuncrk/Iav = 0.00 Iuncrk at position x
kcs = [2-1.2*(Asc/Ast)] ≥ 0.8 = 0.800 kcs at position x Cl 8.5.3.2

Deflection summary - Manual values

Gross δdl.g.imm =	0.0 mm		
Gross δll.g.imm =	0.0 mm		
Short term δ.short = [δdl.g+ψs*δll.g]*Iuncrk/Ief =	0.0 mm	Short term δdl.short =	0.0 mm
Sustained δ.sus = [δdl.g+ψl*δll.g]*Iuncrk/Ief =	0.0 mm	Short term δll.short =	0.0 mm
Long term δ.long = kcs*δ.sus =	0.0 mm	Long term δdl.long =	0.0 mm
Incremental δ.inc = δ.long + δll.short =	0.0 mm	Long term δll.long =	0.0 mm
		Total δdl.total =	0.0 mm
Total δ.total = δ.short + δ.long =	0.0 mm	Span / -	

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Section: **(slab (-)) 250mm thk slab, f'c=32MPa**
Defl'n: **Press the [Deflection Design..] to show**

Results - Press [Design...] to start

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

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Section: **(slab (-)) 250mm thk slab, f'c=32MPa**
Defl'n: **Press the [Deflection Design..] to show**

Results - Press [Design...] to start 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

Section: **(slab (-)) 250mm thk slab, f'c=32MPa**

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) =	250000	mm^2	
Exposed perimeter (ue = 2*1000) =	2000	mm	Cl 1.7
Manual hypothetical thickness =	N	(Y)es,(N)o	
Hypothetical thickness (th = 2*Ag/ue) =	250	mm	Cl 1.7
Concrete strength (f'c) =	32	MPa	
$\alpha 1 = 0.8+1.2*\exp(-0.005*th)$ =	1.144		Fig 3.1.7.2
$k1 = \alpha 1*t^{0.8}/(t^{0.8}+0.15*th)$ =	1.119		Fig 3.1.7.2
Environment (k4) =	0.65	Interior	Cl 3.1.7.2
Final drying basic shrinkage strain (εcsd.b*) =	800	$x10^{-6}$	Standard
Final autogenous strain (εcse* = (0.07*f'c-0.5)*50x10^{-6}) =	87	$x10^{-6}$	Eq 3.1.7.2(3)
Autogenous shrinkage strain (εcse) =	87	$x10^{-6}$	Eq 3.1.7.2(2)
Basic drying shr. strain εcsd.b = (0.9-0.005*f'c)*εcsd.b* =	592	$x10^{-6}$	Eq 3.1.7.2(5)
εcsd = k1*k4*εcsd.b =	431	$x10^{-6}$	Eq 3.1.7.2(4)
εcs = εcse + εcsd =	518	$x10^{-6}$	Eq 3.1.7.2(1)
Final design shrinkage strain εcs =	518	$x10^{-6}$ ±30%	
Rounded final design shrinkage strain εcs =	520	$x10^{-6}$ ±30%	Cl 8.5.3.1
Design shrinkage strain εcs =	520	$x10^{-6}$ ±30%	Calculated value

Creep - Cl 3.1.8

Basic creep coefficient (ϕcc.b) =	3.40		Table 3.1.8.2
$\alpha 2 = 1.0+1.12*\exp(-0.008*th)$ =	1.152		Fig 3.1.8.3
$k2 = \alpha 2*t^{0.8}/(t^{0.8}+0.15*th)$ =	1.127		Fig 3.1.8.3
k3 = 2.7/[1+log(τ)] =	1.10		
Environmental factor (k4) =	0.650		Cl 3.1.8.3
α3 = 0.7/(k4*α2) =	0.935		
k5 =	1.00		
Constant sustained stress (σo) =	0.0	MPa	Cl 3.1.8.1
0.45*fcmi =	15.7	MPa	
Non-linear creep coefficient when σo ≤ 0.45*fcmi (k6) =	1.000		Cl 3.1.8.3
ϕcc = k2*k3*k4*k5*k6*ϕcc.b =	2.747	±30%	Eq 3.1.8.3
Adopted ϕcc =	2.747	±30%	Calculated value

Shrinkage Deflection - Determined midspan [WRH Ch 9.3]

Ast at midspan =	0 mm²	(Area of steel must be pre-set in the Defl tab)	
Depth to tens. steel (ds) =	208 mm		
Asc at midspan =	0 mm²		
κsh = 1.15*εcs/ds*(1-(Asc/Ast) =	0 x10⁻⁹ mm⁻¹		WRH - Eq 9.14
βsh =	0.125 For span type simple beam		
Span (L) =	8000 mm		
Shrinkage δsh = κsh*βsh*L² =	0.0 mm (Uniform beams only)		WRH - Eq 9.15

Creep Deflection - Determined midspan [WRH Ch 9.3]

Tensile steel ratio (pn) =	0.000	(Deflections must have been calculated with analysis)
kcsi =	0.800	(From Analysis and Defl tab)
δe,sus = δ.short = [δdl.g+ψs*δll.g]*i/ieff =	0.0 mm	
δcr = ϕcc*(1-6*pn*(1-6*pn))/(3*(1+Asc/Ast))*δe,sus) =	0.0 mm	WRH - Eq 9.21

Deflection summary for calculated creep and shrinkage

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

Slab crack control for shrinkage and temperature effects - Cl 9.5.3 (Total area)

Design thickness =	250	mm
Shrinkage design thickness =	250	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} =	438	mm^2/m	350	510	910
Restrained slabs (internal) Cl 9.5.3.4 (a)					
Ast.min (minor) = (1.75-2.5*σcp)*b*Dx10^{-3} =	438	mm^2/m	350	510	910
Ast.min (moderate) = (3.50-2.5*σcp)*b*Dx10^{-3} =	875	mm^2/m	170	250	450
Ast.min (strong) = (6.00-2.5*σcp)*b*Dx10^{-3} =	1500	mm^2/m	100	150	260
Restrained slabs (A1 & A2 - external) Cl 9.5.3.4 (b)					
Ast.min (moderate) = (3.50-2.5*σcp)*b*Dx10^{-3} =	875	mm^2/m	170	250	450
Ast.min (strong) = (6.00-2.5*σcp)*b*Dx10^{-3} =	1500	mm^2/m	100	150	260
Restrained slabs (B1, B2, C1, C2 - external) Cl 9.5.3.4 (c)					
Ast.min = (6.00-2.5*σcp)*b*Dx10^{-3} =	1500	mm^2/m	100	150	260

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

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Section:	**(slab (-)) 250mm thk slab, f'c=32MPa**	
Reinf't:	**N24-200 cts top, N24-200 cts bottom**	
Defl'n:	**ds = 208mm > d.min = 185mm (D.min = 227mm)**	**OK**
Defl'n (Incr.):	**ds = 208mm > d.min.i = 191mm (D.min.i = 233mm)**	**OK**

Deemed to comply deflection - Cl 8.5.4 (Not applicable - refer SlabDefl tab)

Span (L) =	8000	mm
Depth =	250	mm
Width =	1000	mm
Span type =	I	(i)nternal, (E)nd, (S)imple
Spans =	2	(2),(>2)

Loading

Dead load S.Wt (wdl) =	6.25	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 =	13.20	kN/m		
Interior span +ve = M+* = Fd*.*Ln²/16 =	52.8	kNm/m	Cl 6.10.2.3(b)	Ductility Class (btm) = N
Interior span -ve = M-* = Fd*.*Ln²/11 =	-76.8	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) =	2262	mm²		Depth to tensile steel (ds) =	208 mm
Compr. steel area (Asc) =	2262	mm²		Depth to comp. steel (dc) =	42 mm
kcs = 2-1.2*(Asc/As) ≥ 0.8 =	0.80	Cl 8.5.3.2		p = Ast/(bef*ds) =	0.01087 (Midspan)
				pc = Asc/(bef*ds) =	0.01087 (Midspan)
Effective design load (Fd.ef) =	16.11	kN/m	Cl 8.5.4(a)	dc/ds =	0.20192
Incremental (Fd.ef.inc) =	8.86	kN/m	Cl 8.5.4(b)	n = Es / Ec =	6.69
				NA =	0.300
Mod. of elasticity ($Ec = p^{1.5}*0.043*\sqrt{fcmi}$) =	29900	MPa ± 20%	Cl 3.1.2	Cracked NA =	62.3 mm

β = bef/bw ≥ 1 =	1.00		Cl 8.5.4
$k1 = (5-0.04*f'c)*p+0.002 \le 0.1/\beta^{2/3}$ =	0.042		$p \ge 0.001*fc^{1/3}/\beta^{2/3} = 0.0032$
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
$d.min = Lef/[k1*(\Delta/Lef)*bef*Ec/(k2*Fd.ef)]^{1/3}$ =	185	mm	Eq 8.5.4
d.min. (inc) =	191	mm	
D.min. =	227	mm	
D.min. (inc) =	233	mm	

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Section:	**(slab (-)) 250mm thk slab, f'c=32MPa**	
Reinf't:	**N24-200 cts top, N24-200 cts bottom**	
Defl'n:	**ds = 208mm > d.min = 195mm (D.min = 237mm)**	**OK**
Defl'n (Incr.):	**ds = 208mm > d.min.i = 202mm (D.min.i = 244mm)**	**OK**

Deemed to comply deflection - Cl 9.4.4.1 (Internal span)

Span (Ln) =	8000	mm
Slab thickness =	250	mm
Span type =	I	(i)nternal, (E)nd, (S)imple
Spans =	2	(2),(>2)

Loading

Dead load S.Wt (wdl) =	6.25	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 =	13.20	kPa		
Interior span +ve = M+* = Fd*.*Ln²/16 =	52.8	kNm/m	Cl 6.10.2.3(b)	Ductility Class (btm) = N
Interior span -ve = M-* = Fd*.*Ln²/11 =	-76.8	kNm/m	Cl 6.10.2.2(b)	Ductility Class (top) = N

Slab deflection (Deemed to Comply)

Tensile steel area (Ast) =	2262 mm²/m		Depth to tensile steel (ds) =	208	mm
Compr. steel area (Asc) =	2262 mm²/m		Depth to comp. steel (dc) =	42	mm
kcs = 2-1.2*(Asc/Ast) ≥ 0.8 =	0.80 Cl 8.5.3.2		p = Ast/(W*ds) =	0.01087	(Midspan)
			pc = Asc/(W*ds) =	0.01087	(Midspan)
Effective design load (Fd.ef) =	16.11 kPa	Cl 9.3.4.1(a)	dc/ds =	0.20192	
Incremental (Fd.ef.inc) =	8.86 kPa	Cl 9.3.4.1(b)	n = Es/Ec =	6.69	
Mod. of elasticity (Ec = $\rho^{1.5}$*0.043*√fcmi) =	29900 MPa ± 20%	Cl 3.1.2	Neutral axis (NA) =	0.300	
			Cracked NA =	62.3	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]$^{1/3}$) =	195	mm	Eq 9.4.4.1
d.min (inc) =	202	mm	
D.min =	237	mm	
D.min (inc) =	244	mm	