

DESIGN REPORT

RCC Box Culvert - 2 × 3 m × 3 m

Designed in accordance with IRC 112:2020 and IRC 6:2017

Project	Box Culvert
Client	-
Location	-
Chainage	-
Designed by	-
Checked by	-
Date	31 Oct 2026

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1. Design summary

All design checks are satisfied.

Project	Box Culvert
Client	-
Location / chainage	-
Structure	2 cell RCC box, 2 × 3 m (clear) × 3 m (clear)
Overall size	7.000 m wide × 3.750 m high × 12 m long
Member thickness	Top 350 / bottom 400 / outer walls 350 / inner walls 300 mm
Materials	M30 concrete, Fe500 reinforcement, clear cover 50 mm
Fill over top slab	0.600 m
Reinforcement (barrel)	9,462 kg (789 kg per m of barrel)
Estimated cost (BOQ)	INR 2,935,106
Report date	31 Oct 2026

Main reinforcement

Mark	Member	Face	Bars	As,prov (mm ² /m)	Detailing	Status
A	Top slab	Outer (top) face	16Ø @ 120 c/c	1,676	Design	OK
B	Top slab	Inner (bottom) face	12Ø @ 200 c/c	565	Design	OK
C	Bottom slab	Outer (bottom) face	20Ø @ 125 c/c	2,513	Design	OK
D	Bottom slab	Inner (top) face	12Ø @ 130 c/c	870	Design	OK
E	Outer walls	Outer (earth) face	12Ø @ 115 c/c	983	Design	OK
F	Outer walls	Inner (cell) face	12Ø @ 200 c/c	565	Design	OK
G	Inner walls	Both faces	12Ø @ 200 c/c	565	Design	OK
S1	Approach slab	Bottom face (main)	20Ø @ 190 c/c	1,653	Design	OK
S2	Approach slab	Top face (nominal)	12Ø @ 200 c/c	565	Design	OK
R1	Brackets	Main tie	12Ø @ 200 c/c	565	Design	OK

2. Design basis and assumptions

Clause references in the calculations are to IRC 112:2020 unless prefixed (IRC 6 = IRC 6:2017; EN 1992-1-1 where IRC 112 adopts the same model). Verify references against the code editions and amendments in force for the project.

Notation

Symbol	Meaning
h, d, b	Section depth, effective depth, design strip width (1000 mm)
MEd, VEd	Design bending moment and shear force (ULS)
f _{ck} , f _{cd} , f _{ctm}	Characteristic, design compressive and mean tensile strength of concrete
f _{yk} , f _{yd} , E _s	Characteristic and design yield strength, modulus of steel
x _u , x _{u,lim} , λ, η	Neutral axis depth, limiting depth, stress block factors
A _{s,req} , A _{s,min} , A _{s,prov}	Steel required, minimum and provided per metre
σ _c , σ _s , w _k	Concrete / steel service stress, characteristic crack width
α _e , ρ, ρ _{p,eff}	Modular ratio, steel ratio, effective tension steel ratio
k, ρ ₁ , v _{Rd,c} , V _{Rd,c}	Shear size factor, tension steel ratio, shear resistance

- IRC 112:2020 - Code of Practice for Concrete Road Bridges (limit state design).
- IRC 6:2017 - Standard Specifications and Code of Practice for Road Bridges, Section II (Loads and load combinations).
- IRC SP 13 - Guidelines for the design of small bridges and culverts (general arrangement, aprons and curtain walls).
- Analysis of a 1 m strip of the barrel as a closed plane frame along member centre-lines, supported on Winkler springs under the bottom slab (modulus of subgrade reaction 20,000 kN/m³).
- Haunches are included in self-weight; their stiffening effect is ignored and design moments are taken at the faces of supports (conservative).
- Live load is dispersed through the fill and wearing course at 1H:1V on both directions; vehicles are moved across the span in steps of 100 mm in both directions and the envelope is used.
- Live load surcharge equivalent to 1.2 m of earth fill (IRC 6 cl 214.1.1.3) acting on one or both walls, whichever governs.

- Lateral earth pressure: at-rest $K_0 = 1 - \sin \phi = 0.500$.
- Water inside the cells (full to soffit) is considered as a variable action where it is unfavourable.
- Ultimate limit state: basic combination with adverse / relieving partial factors per IRC 6 Table B.2 applied to each action independently at every section (envelope).
- Serviceability: stress limits under the rare combination ($\sigma_c \leq 0.48 f_{ck}$, $\sigma_s \leq 0.8 f_{yk}$) and crack width under the quasi-permanent combination (IRC 112 cl 12.2, 12.3).
- Shear checked as members without shear reinforcement (IRC 112 cl 10.3.2), at distance d from the face.
- Braking, temperature, seismic forces and skew effects are not included. Wing walls, aprons and curtain walls are proportioned nominally.

3. Input data

Geometry

Parameter	Value
Number of cells	2
Clear span of each cell	3.000 m
Clear height	3.000 m
Top / bottom slab thickness	350 / 400 mm
Outer / inner wall thickness	350 / 300 mm
Haunch (45°)	150 mm
Barrel length	12.000 m
Fill over top slab / wearing course	0.600 m / 65 mm
Carriageway width	7.50 m
Bed (invert) level / HFL	100.000 / 102.000
PCC levelling course	150 mm, projection 150 mm
Apron / curtain wall	3 m × 300 mm / 1 m deep × 300 mm

Materials

Parameter	Value
Concrete grade	M30
Reinforcement grade	Fe500
Clear cover	50 mm
Exposure condition	Moderate
Unit weight of concrete / wearing course	25 / 22 kN/m ³
Bar diameters - main / distribution	12,16,20,25 / 10,12,16 mm
Bar spacing range	100 - 200 mm (distribution up to 250)

Soil and water

Parameter	Value
Unit weight of fill	18 kN/m ³
Angle of internal friction	30°
Safe bearing capacity	200 kPa
Modulus of subgrade reaction	20,000 kN/m ³
Water in cells	Yes
Ground water level (flotation check)	100.500

Live load

Parameter	Value
Vehicles considered	Class A (2 lanes), 70R tracked, 70R wheeled
Impact	IRC 6 cl 208 (by span)
Dispersion through fill	1 H : 1 V
Live load surcharge	1.2 m equivalent fill

4. Material properties (IRC 112)

Concrete and reinforcement				OK
Reference	Description	Calculation	Result	
Table 6.5	Characteristic compressive strength	M30	fck = 30 MPa	
Cl. 6.4.2.8	Design compressive strength	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_m = 0.67 \times 30 / 1.5$	13.40 MPa	
Table 6.5	Mean axial tensile strength	$f_{ctm} = 0.259 \cdot f_{ck}^{(2/3)} = 0.259 \times 30^{(2/3)}$	2.50 MPa	
Cl. 6.4.2.8	Design tensile strength	$f_{ctd} = f_{ctm} / \gamma_m = 0.7 \times 2.50 / 1.5$	1.167 MPa	
Table 6.5	Secant modulus of elasticity	E _{cm} (interpolated)	31,000 MPa	
Cl. 12.2	Effective (long-term) modulus	$E_{c,eff} = E_{cm} / (1 + \phi) = 31,000 / (1 + 1)$	15,500 MPa	
Cl. 6.4.2.8	Ultimate strain / stress block	$\epsilon_{cu3}, \lambda, \eta$	0.0035, 0.8, 1.0	
Cl. 6.2.2	Yield strength of reinforcement	Fe500 (IS 1786)	f_{yk} = 500 MPa	
Cl. 6.2.2	Design yield strength	$f_{yd} = f_{yk} / \gamma_s = 500 / 1.15$	434.8 MPa	
Cl. 6.2.2	Modulus of elasticity of steel	E _s	200,000 MPa	
Table 12.1	Limiting crack width	Moderate exposure, RCC, quasi-permanent	w_{k,max} = 0.3 mm	
Cl. 15.2.3	Basic anchorage length (16 mm bar)	$l_{b,rqd} = (\phi/4) \cdot (f_{yd}/f_{bd})$, $f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.626 \text{ MPa}$	662 mm	
Cl. 15.2.5	Lap length (16 mm bar)	$l_0 = \alpha_6 \cdot l_{b,rqd} \geq \max(15\phi, 200)$, $\alpha_6 = 1.5$	1000 mm	

5. Analysis model and loads

Centre-line spans	3.325, 3.325 m
Centre-line height	3.375 m
Frame model	161 beam elements, 160 nodes, 47 base springs
Earth pressure coefficient	K = 0.500

Characteristic load intensities

Action	Reference	Description	Value
DL	IRC 6 Cl. 203	Top slab self weight = $\gamma_c \times t = 25 \times 0.350$	8.75 kPa
DL	IRC 6 Cl. 203	Bottom slab self weight = 25×0.400	10.00 kPa
DL	IRC 6 Cl. 203	Walls and haunches	self weight of members
SIDL	IRC 6 Cl. 203	Wearing course 65 mm × 22	1.43 kPa
FILL	IRC 6 Cl. 203	Earth fill 0.600 m × 18	10.80 kPa
EP	IRC 6 Cl. 214	Earth pressure at top of top slab = $K \cdot \gamma \cdot z$	6.11 kPa
EP	IRC 6 Cl. 214	Earth pressure at base of bottom slab = $K \cdot \gamma \cdot z$	39.86 kPa
LLS	IRC 6 Cl. 214.1.1.3	Surcharge = $K \cdot \gamma \times 1.2 \text{ m}$	10.80 kPa
WATER	IRC 6 Cl. 213	Water pressure at base of cell = $\gamma_w \times h$	30.00 kPa
RAIL	IRC 6 Cl. 203	Railing + headwall: 2 × 13.63 / 12 m barrel length	2.27 kPa
APP	IRC 6 Cl. 203	Approach slab reaction on bracket at e = 0.325 m from wall centre-line	37.34 kN/m + 12.14 kNm/m
APPLL	IRC 6 Cl. 204, 208	Live load reaction on bracket (one side at a time)	61.36 kN/m + 19.94 kNm/m

6. Live load (IRC 6)

Dispersed width across the barrel for one train = gauge + contact width + 2 · h · tan θ; for several trains it is limited by the train spacing (overlapping dispersion). Load per metre of barrel = n × P × reduction × (1 + impact) / total width; pressure = load per metre / dispersed length.

IRC Class A - lanes 2, reduction 1, impact 48.3 %, dispersion depth 0.665 m; top slab M range -32.2 to 28.5 kNm/m

Offset (m)	Load (kN)	Dispersed length (m)	Width / train (m)	Total width (m)	Load per m (kN/m)	Pressure (kPa)
0.00	27	1.480	3.330	6.660	12.02	8.12
1.10	27	1.480	3.330	6.660	12.02	8.12

Offset (m)	Load (kN)	Dispersed length (m)	Width / train (m)	Total width (m)	Load per m (kN/m)	Pressure (kPa)
4.30	114	1.580	3.630	7.130	47.41	30.01
5.50	114	1.580	3.630	7.130	47.41	30.01
9.80	68	1.530	3.510	7.010	28.76	18.80
12.80	68	1.530	3.510	7.010	28.76	18.80
15.80	68	1.530	3.510	7.010	28.76	18.80
18.80	68	1.530	3.510	7.010	28.76	18.80

IRC Class 70R (Tracked) - lanes 1, reduction 1, impact 25.0 %, dispersion depth 0.665 m; top slab M range -35.1 to 28.1 kNm/m

Offset (m)	Load (kN)	Dispersed length (m)	Width / train (m)	Total width (m)	Load per m (kN/m)	Pressure (kPa)
0.00	700	5.900	4.230	4.230	206.86	35.06

IRC Class 70R (Wheeled) - lanes 1, reduction 1, impact 25.0 %, dispersion depth 0.665 m; top slab M range -44.4 to 29.3 kNm/m

Offset (m)	Load (kN)	Dispersed length (m)	Width / train (m)	Total width (m)	Load per m (kN/m)	Pressure (kPa)
0.00	80	1.560	3.870	3.870	25.84	16.56
3.96	120	1.590	4.070	4.070	36.86	23.18
5.48	120	1.590	4.070	4.070	36.86	23.18
7.61	170	1.690	4.120	4.120	51.58	30.52
8.98	170	1.690	4.120	4.120	51.58	30.52
12.03	170	1.690	4.120	4.120	51.58	30.52
13.40	170	1.690	4.120	4.120	51.58	30.52

7. Load combinations

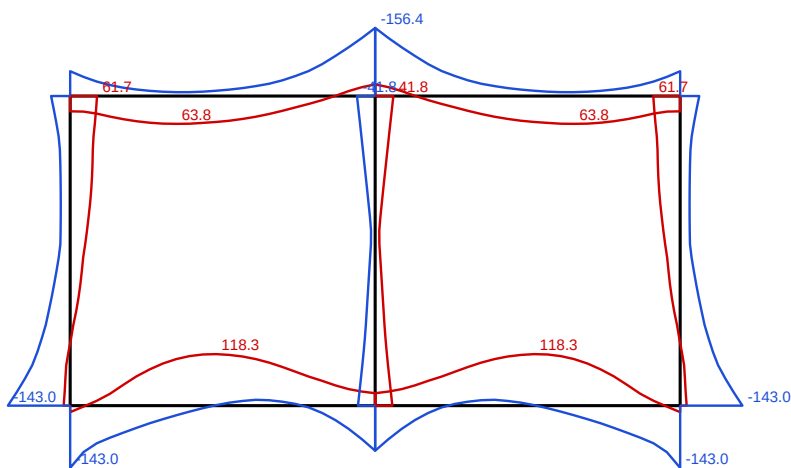
Partial factors (adverse / relieving). Variable actions have a relieving factor of zero, i.e. they are included only where they increase the effect.

Action	Description	ULS basic (B.2)	SLS rare (B.3)	SLS quasi-permanent
DL	Dead load (self weight)	1.35 / 1	1 / 1	1 / 1
SIDL	Wearing course	1.75 / 1	1.2 / 1	1.2 / 1
FILL	Earth fill on top slab	1.35 / 1	1 / 1	1 / 1
EP	Lateral earth pressure	1.5 / 1	1 / 1	1 / 1
RAIL	Railing / crash barrier + headwall (edge strip)	1.35 / 1	1 / 1	1 / 1
APP	Approach slab & bracket dead load on walls	1.35 / 1	1 / 1	1 / 1
LL	Vehicular live load (incl. impact)	1.5 / 0	1 / 0	0 / 0
APPLL	Live load on one approach slab (bracket reaction)	1.5 / 0	1 / 0	0 / 0
LLS	Live load surcharge	1.2 / 0	0.8 / 0	0 / 0
WATER	Water inside cells	1 / 0	1 / 0	0 / 0

8. Analysis results

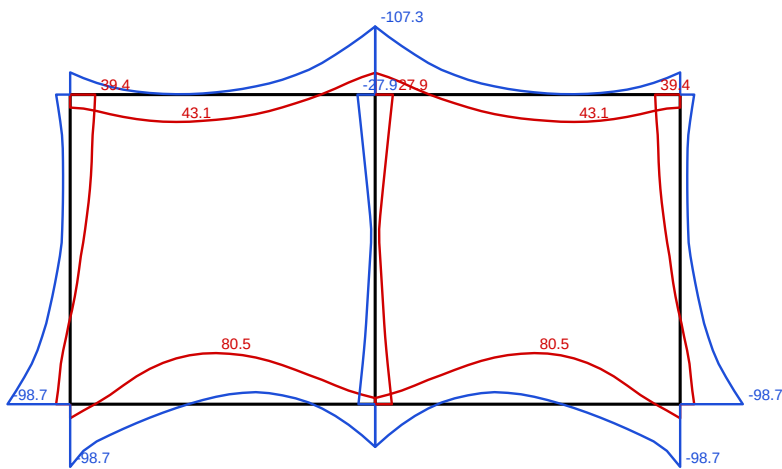
Unfactored moments (kNm/m) at the governing section of each design location, positive when causing tension on the face being designed. LL and LLS are the governing envelope values.

Design location	Governing section	DL	SIDL	FILL	EP	RAIL	APP	LL	APPLL	LLS	WATER	ULS	SLS rare	SLS QP
Top slab - Outer (top) face	cell 1, support face	14.8	1.4	10.3	-7.1	2.2	12.7	37.0	13.5	4.3	2.3	132.5	90.7	34.6
Top slab - Inner (bottom) face	cell 2, s = 2.17 m	7.0	0.9	6.7	-6.4	1.4	-0.1	26.6	3.0	0.6	3.4	63.8	43.1	9.7
Bottom slab - Outer (bottom) face	cell 2, support face	6.1	0.0	0.1	15.4	0.0	14.8	6.9	13.5	16.8	-12.8	102.6	70.4	36.6
Bottom slab - Inner (top) face	cell 1, s = 1.59 m	13.9	1.0	7.2	-5.5	1.5	9.2	26.6	20.1	7.0	0.7	118.3	80.5	29.3
Outer walls - Outer (earth) face	left wall, support face	10.6	0.3	2.6	7.9	0.5	17.2	12.7	18.6	12.2	-8.3	115.9	80.4	39.3
Outer walls - Inner (cell) face	right wall, support face	2.5	-0.4	-3.3	-9.1	-0.7	17.3	6.3	17.2	5.3	4.2	59.0	38.2	12.4
Inner walls - Both faces	inner wall 1, support face	-0.0	-0.0	-0.0	-0.0	-0.0	-0.0	9.7 / -9.7	6.2 / -6.2	11.5 / -11.5	-0.0	37.7	25.1	0.0



ULS envelope (kNm/m). Red: max, blue: min. Plotted on the inner face when positive (tension on cell face).

ULS bending moment envelope (kNm/m)



SLS rare envelope (kNm/m). Red: max, blue: min. Plotted on the inner face when positive (tension on cell face).

SLS rare bending moment envelope (kNm/m)

9. Design for flexure and serviceability (IRC 112 Cl. 8.2, 12.2, 12.3)

Mar k	Location	h (mm)	d (mm)	MEd (kNm/m)	As,req	As,min	Provided	Detailin g	As,prov	σc (MPa)	σs (MPa)	wk (mm)	Statu s
A	Top slab - Outer (top) face	350	292	132.5	1112	380	16Ø @ 120 c/c	Design	1676	7.49	207	0.073	OK
B	Top slab - Inner (bottom) face	350	294	63.8	514	382	12Ø @ 200 c/c	Design	565	5.35	277	0.097	OK

Mar k	Location	h (mm)	d (mm)	MEd (kNm/m)	As,req	As,min	Provided	Detailing	As,prov	σ_c (MPa)	σ_s (MPa)	wk (mm)	Status
C	Bottom slab - Outer (bottom) face	400	340	102.6	719	442	20Ø @ 125 c/c	Design	2513	3.92	93	0.043	OK
D	Bottom slab - Inner (top) face	400	344	118.3	823	447	12Ø @ 130 c/c	Design	870	6.54	291	0.134	OK
E	Outer walls - Outer (earth) face	350	294	115.9	957	382	12Ø @ 115 c/c	Design	983	8.01	304	0.160	OK
F	Outer walls - Inner (cell) face	350	294	59.0	474	382	12Ø @ 200 c/c	Design	565	4.75	246	0.125	OK
G	Inner walls - Both faces	300	244	37.7	364	317	12Ø @ 200 c/c	Design	565	4.20	196	0.000	OK

A: Top slab - Outer (top) face (cell 1, support face)				OK	
Reference	Description	Calculation	Result		
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	M _{Ed} - envelope at cell 1, support face	132.47 kNm/m		
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	M _{rare} ; M _{qp}	90.74 / 34.57 kNm/m		
Cl. 14.3	Effective depth	d = h – c _{nom} – ϕ/2 = 350 – 50 – 16/2	292.0 mm		
Cl. 6.4.2.8	Design compressive strength	f _{cd} = α _{cc} ·f _{ck} /γ _m = 0.67×30/1.5	13.40 MPa		
Cl. 6.2.2	Design yield strength	f _{yd} = f _{yk} /γ _s = 500/1.15	434.8 MPa		
Cl. 8.2	Limiting neutral axis depth	x _{u,lim} = ε _{cu3} /(ε _{cu3} + f _{yd} /E _s)·d = 0.0035/(0.0035 + 0.00217)×292.0	180.1 mm		
Cl. 8.2	Limiting moment of resistance	M _{Rd,lim} = η·f _{cd} ·b·λ·x _{u,lim} ·(d – λ·x _{u,lim} /2)	424.7 kNm/m	OK	
Cl. 8.2, Fig. 6.5	Neutral axis depth (λ = 0.8, η = 1.0)	M _{Ed} = η·f _{cd} ·b·λ·x·(d – λ·x/2) → solve for x	45.1 mm	OK	
Cl. 8.2	Tension steel required	A _{s,req} = η·f _{cd} ·b·λ·x/f _{yd} = 1.0×13.40×1000×0.8×45.1/434.8	1112 mm²/m		
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	A _{s,min} = max(0.26·f _{ctm} /f _{yk} ·b·d, 0.0013·b·d) = max(0.26×2.50/500×1000×292, 0.0013×1000×292)	380 mm²/m		
	Steel provided	16Ø @ 120 c/c: A _s = π·ϕ²/4 × 1000/s = π×16²/4×1000/120	1676 mm²/m ≥ 1112	OK	
Cl. 16.5.1.1	Maximum steel	A _s ≤ 0.025·b·h = 0.025×1000×350	8750 mm²/m	OK	
Cl. 16.6.1.1	Maximum bar spacing	s ≤ min(2h, 250 mm) = min(2×350, 250)	120 ≤ 250 mm	OK	
Cl. 16.2	Minimum clear spacing	s – ϕ ≥ max(ϕ, d _g + 5) = max(16, 20 + 5)	104 ≥ 25 mm	OK	
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	α _e = E _s /E _{c,eff} = E _s ·(1 + ϕ)/E _{cm} = 200000×2.00/31000	12.90		
Cl. 12.2	Cracked section neutral axis	x = d·[–α _e ·ρ + √((α _e ·ρ)² + 2·α _e ·ρ)], ρ = A _s /(b·d) = 0.00574	92.8 mm		
Cl. 12.2	Cracked moment of inertia	I _{cr} = b·x³/3 + α _e ·A _s ·(d – x)²	1124.3 ×10^6 mm ⁴		
Cl. 12.2.1	Concrete stress, rare combination	σ _c = M·x/I _{cr} ≤ 0.48·f _{ck} = 0.48×30	7.49 ≤ 14.40 MPa	OK	
Cl. 12.2.2	Steel stress, rare combination	σ _s = α _e ·M·(d – x)/I _{cr} ≤ 0.8·f _{yk} = 0.8×500	207 ≤ 400 MPa	OK	
Cl. 12.3.4	Steel stress, quasi-permanent	σ _s = α _e ·M _{qp} ·(d – x)/I _{cr}	79.0 MPa		
Cl. 12.3.4	Effective tension area	h _{c,eff} = min[2.5(h – d), (h – x)/3, h/2]; ρ _{p,eff} = A _s /(b·h _{c,eff})	85.7 mm; 0.0195		
Cl. 12.3.4	Mean strain difference	ε _{sm} – ε _{cm} = [σ _s – k _t ·f _{ctm} /ρ _{p,eff} ·(1 + α _e ·ρ _{p,eff})]/E _s ≥ 0.6·σ _s /E _s (k _t = 0.4, α _e = 6.45)	0.000237		
Cl. 12.3.4	Maximum crack spacing	s _{r,max} = 3.4·c + 0.425·k ₁ ·k ₂ ·ϕ/ρ _{p,eff} = 3.4×50 + 0.425×0.8×0.5×16/0.0195	309.2 mm		
Cl. 12.3.4, Table 12.1	Crack width	w _k = s _{r,max} ·(ε _{sm} – ε _{cm}) ≤ w _{k,max} (moderate exposure)	0.073 ≤ 0.3 mm	OK	
	Note	Steel increased to 16Ø @ 120 c/c so that V _{Rd,c} ≥ V _{Ed} (shear).			

B: Top slab - Inner (bottom) face (cell 2, s = 2.17 m)				OK
Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	MEd - envelope at cell 2, s = 2.17 m	63.83 kNm/m	

Reference	Description	Calculation	Result	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	$M_{\text{rare}} ; M_{\text{qp}}$	43.06 / 9.67 kNm/m	
Cl. 14.3	Effective depth	$d = h - c_{\text{nom}} - \phi/2 = 350 - 50 - 12/2$	294.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{\text{cd}} = \alpha_{\text{cc}} \cdot f_{\text{ck}} / \gamma_{\text{m}} = 0.67 \times 30 / 1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{\text{yd}} = f_{\text{yk}} / \gamma_{\text{s}} = 500 / 1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{\text{u,lim}} = \epsilon_{\text{cu}3} / (\epsilon_{\text{cu}3} + f_{\text{yd}} / E_{\text{s}}) \cdot d = 0.0035 / (0.0035 + 0.00217) \times 294.0$	181.4 mm	
Cl. 8.2	Limiting moment of resistance	$M_{\text{Rd,lim}} = \eta \cdot f_{\text{cd}} \cdot b \cdot \lambda \cdot x_{\text{u,lim}} \cdot (d - \lambda \cdot x_{\text{u,lim}} / 2)$	430.5 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8, \eta = 1.0$)	$M_{\text{Ed}} = \eta \cdot f_{\text{cd}} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x / 2) \rightarrow \text{solve for } x$	20.8 mm	OK
Cl. 8.2	Tension steel required	$A_{\text{s,req}} = \eta \cdot f_{\text{cd}} \cdot b \cdot \lambda \cdot x / f_{\text{yd}} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 20.8 / 434.8$	514 mm ² /m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{\text{s,min}} = \max(0.26 \cdot f_{\text{ctm}} / f_{\text{yk}} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50 / 500 \times 1000 \times 294, 0.0013 \times 1000 \times 294)$	382 mm ² /m	
	Steel provided	12Ø @ 200 c/c: $A_{\text{s}} = \pi \cdot \phi^2 / 4 \times 1000 / s = \pi \times 12^2 / 4 \times 1000 / 200$	565 mm ² /m $\geq$ 514	OK
Cl. 16.5.1.1	Maximum steel	$A_{\text{s}} \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 350$	8750 mm ² /m	OK
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 350, 250)$	200 $\leq$ 250 mm	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, d_{\text{g}} + 5) = \max(12, 20 + 5)$	188 $\geq$ 25 mm	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_{\text{e}} = E_{\text{s}} / E_{\text{c,eff}} = E_{\text{s}} \cdot (1 + \phi) / E_{\text{cm}} = 200000 \times 2.00 / 31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_{\text{e}} \cdot \rho + \sqrt{(\alpha_{\text{e}} \cdot \rho)^2 + 2 \cdot \alpha_{\text{e}} \cdot \rho}]$, $\rho = A_{\text{s}} / (b \cdot d) = 0.00192$	58.6 mm	
Cl. 12.2	Cracked moment of inertia	$I_{\text{cr}} = b \cdot x^3 / 3 + \alpha_{\text{e}} \cdot A_{\text{s}} \cdot (d - x)^2$	471.4 $\times 10^6$ mm ⁴	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_{\text{c}} = M \cdot x / I_{\text{cr}} \leq 0.48 \cdot f_{\text{ck}} = 0.48 \times 30$	5.35 $\leq$ 14.40 MPa	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_{\text{s}} = \alpha_{\text{e}} \cdot M \cdot (d - x) / I_{\text{cr}} \leq 0.8 \cdot f_{\text{yk}} = 0.8 \times 500$	277 $\leq$ 400 MPa	OK
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_{\text{s}} = \alpha_{\text{e}} \cdot M_{\text{qp}} \cdot (d - x) / I_{\text{cr}}$	62.3 MPa	
Cl. 12.3.4	Effective tension area	$h_{\text{c,eff}} = \min[2.5(h - d), (h - x) / 3, h / 2]$; $\rho_{\text{p,eff}} = A_{\text{s}} / (b \cdot h_{\text{c,eff}})$	97.1 mm; 0.0058	
Cl. 12.3.4	Mean strain difference	$\epsilon_{\text{sm}} - \epsilon_{\text{cm}} = [\sigma_{\text{s}} - k_{\text{t}} \cdot f_{\text{ctm}} / \rho_{\text{p,eff}} \cdot (1 + \alpha_{\text{e}} \cdot \rho_{\text{p,eff}})] / E_{\text{s}} \geq 0.6 \cdot \sigma_{\text{s}} / E_{\text{s}}$ ($k_{\text{t}} = 0.4, \alpha_{\text{e}} = 6.45$)	0.000187	
Cl. 12.3.4	Maximum crack spacing	$s_{\text{r,max}} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / \rho_{\text{p,eff}} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 12 / 0.0058$	520.4 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_{\text{k}} = s_{\text{r,max}} \cdot (\epsilon_{\text{sm}} - \epsilon_{\text{cm}}) \leq w_{\text{k,max}}$ (moderate exposure)	0.097 $\leq$ 0.3 mm	OK

C: Bottom slab - Outer (bottom) face (cell 2, support face)

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	M_{Ed} - envelope at cell 2, support face	102.58 kNm/m	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	$M_{\text{rare}} ; M_{\text{qp}}$	70.41 / 36.57 kNm/m	
Cl. 14.3	Effective depth	$d = h - c_{\text{nom}} - \phi/2 = 400 - 50 - 20/2$	340.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{\text{cd}} = \alpha_{\text{cc}} \cdot f_{\text{ck}} / \gamma_{\text{m}} = 0.67 \times 30 / 1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{\text{yd}} = f_{\text{yk}} / \gamma_{\text{s}} = 500 / 1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{\text{u,lim}} = \epsilon_{\text{cu}3} / (\epsilon_{\text{cu}3} + f_{\text{yd}} / E_{\text{s}}) \cdot d = 0.0035 / (0.0035 + 0.00217) \times 340.0$	209.7 mm	
Cl. 8.2	Limiting moment of resistance	$M_{\text{Rd,lim}} = \eta \cdot f_{\text{cd}} \cdot b \cdot \lambda \cdot x_{\text{u,lim}} \cdot (d - \lambda \cdot x_{\text{u,lim}} / 2)$	575.8 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8, \eta = 1.0$)	$M_{\text{Ed}} = \eta \cdot f_{\text{cd}} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x / 2) \rightarrow \text{solve for } x$	29.1 mm	OK
Cl. 8.2	Tension steel required	$A_{\text{s,req}} = \eta \cdot f_{\text{cd}} \cdot b \cdot \lambda \cdot x / f_{\text{yd}} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 29.1 / 434.8$	719 mm ² /m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{\text{s,min}} = \max(0.26 \cdot f_{\text{ctm}} / f_{\text{yk}} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50 / 500 \times 1000 \times 340, 0.0013 \times 1000 \times 340)$	442 mm ² /m	

Reference	Description	Calculation	Result	
	Steel provided	$20\emptyset @ 125 \text{ c/c: } A_s = \pi \cdot \phi^2/4 \times 1000/s = \pi \times 20^2/4 \times 1000/125$	2513 mm²/m ≥ 719	OK
Cl. 16.5.1.1	Maximum steel	$A_s \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 400$	10000 mm²/m	OK
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 400, 250)$	125 $\leq 250 \text{ mm}$	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, dg + 5) = \max(20, 20 + 5)$	105 $\geq 25 \text{ mm}$	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_e = E_s/E_{c,eff} = E_s \cdot (1 + \phi)/E_{cm} = 200000 \times 2.00/31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_e \cdot \rho + \sqrt{((\alpha_e \cdot \rho)^2 + 2 \cdot \alpha_e \cdot \rho)}], \rho = A_s/(b \cdot d) = 0.00739$	119.6 mm	
Cl. 12.2	Cracked moment of inertia	$I_{cr} = b \cdot x^3/3 + \alpha_e \cdot A_s \cdot (d - x)^2$	2145.6 $\times 10^6 \text{ mm}^4$	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_c = M \cdot x/I_{cr} \leq 0.48 \cdot f_{ck} = 0.48 \times 30$	3.92 $\leq 14.40 \text{ MPa}$	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_s = \alpha_e \cdot M \cdot (d - x)/I_{cr} \leq 0.8 \cdot f_{yk} = 0.8 \times 500$	93 $\leq 400 \text{ MPa}$	OK
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_s = \alpha_e \cdot M_{qp} \cdot (d - x)/I_{cr}$	48.5 MPa	
Cl. 12.3.4	Effective tension area	$h_{c,eff} = \min[2.5(h - d), (h - x)/3, h/2]; \rho_{p,eff} = A_s/(b \cdot h_{c,eff})$	93.5 mm; 0.0269	
Cl. 12.3.4	Mean strain difference	$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot f_{ctm}/\rho_{p,eff} \cdot (1 + \alpha_e \cdot \rho_{p,eff})]/E_s \geq 0.6 \cdot \sigma_s/E_s \text{ (} k_t = 0.4, \alpha_e = 6.45 \text{)}$	0.000145	
Cl. 12.3.4	Maximum crack spacing	$s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi/\rho_{p,eff} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 20/0.0269$	296.5 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k,max} \text{ (moderate exposure)}$	0.043 $\leq 0.3 \text{ mm}$	OK
	Note	Steel increased to 20 $\emptyset @ 125 \text{ c/c}$ so that $V_{Rd,c} \geq V_{Ed}$ (shear).		

D: Bottom slab - Inner (top) face (cell 1, s = 1.59 m)				OK
Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	M _{Ed} - envelope at cell 1, s = 1.59 m	118.32 kNm/m	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	M _{rare} ; M _{qp}	80.50 / 29.35 kNm/m	
Cl. 14.3	Effective depth	$d = h - c_{nom} - \phi/2 = 400 - 50 - 12/2$	344.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_m = 0.67 \times 30/1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{yd} = f_{yk}/\gamma_s = 500/1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{u,lim} = \epsilon_{cu3}/(\epsilon_{cu3} + f_{yd}/E_s) \cdot d = 0.0035/(0.0035 + 0.00217) \times 344.0$	212.2 mm	
Cl. 8.2	Limiting moment of resistance	$M_{Rd,lim} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{u,lim} \cdot (d - \lambda \cdot x_{u,lim}/2)$	589.4 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8, \eta = 1.0$)	$M_{Ed} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x/2) \rightarrow \text{solve for } x$	33.4 mm	OK
Cl. 8.2	Tension steel required	$A_{s,req} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x/f_{yd} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 33.4/434.8$	823 mm²/m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{s,min} = \max(0.26 \cdot f_{ctm}/f_{yk} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50/500 \times 1000 \times 344, 0.0013 \times 1000 \times 344)$	447 mm²/m	
	Steel provided	$12\emptyset @ 130 \text{ c/c: } A_s = \pi \cdot \phi^2/4 \times 1000/s = \pi \times 12^2/4 \times 1000/130$	870 mm²/m ≥ 823	OK
Cl. 16.5.1.1	Maximum steel	$A_s \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 400$	10000 mm²/m	OK
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 400, 250)$	130 $\leq 250 \text{ mm}$	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, dg + 5) = \max(12, 20 + 5)$	118 $\geq 25 \text{ mm}$	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_e = E_s/E_{c,eff} = E_s \cdot (1 + \phi)/E_{cm} = 200000 \times 2.00/31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_e \cdot \rho + \sqrt{((\alpha_e \cdot \rho)^2 + 2 \cdot \alpha_e \cdot \rho)}], \rho = A_s/(b \cdot d) = 0.00253$	77.4 mm	
Cl. 12.2	Cracked moment of inertia	$I_{cr} = b \cdot x^3/3 + \alpha_e \cdot A_s \cdot (d - x)^2$	952.4 $\times 10^6 \text{ mm}^4$	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_c = M \cdot x/I_{cr} \leq 0.48 \cdot f_{ck} = 0.48 \times 30$	6.54 $\leq 14.40 \text{ MPa}$	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_s = \alpha_e \cdot M \cdot (d - x)/I_{cr} \leq 0.8 \cdot f_{yk} = 0.8 \times 500$	291 $\leq 400 \text{ MPa}$	OK

Reference	Description	Calculation	Result	
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_s = \alpha_e \cdot M_{qp} \cdot (d - x) / I_{cr}$	106.0 MPa	
Cl. 12.3.4	Effective tension area	$h_{c,eff} = \min[2.5(h - d), (h - x)/3, h/2]$; $p_{p,eff} = A_s / (b \cdot h_{c,eff})$	107.5 mm; 0.0081	
Cl. 12.3.4	Mean strain difference	$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot f_{ctm} / p_{p,eff} \cdot (1 + \alpha_e \cdot p_{p,eff})] / E_s \geq 0.6 \cdot \sigma_s / E_s$ ($k_t = 0.4$, $\alpha_e = 6.45$)	0.000318	
Cl. 12.3.4	Maximum crack spacing	$s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / p_{p,eff} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 12 / 0.0081$	422.2 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k,max}$ (moderate exposure)	$0.134 \leq 0.3$ mm	OK

E: Outer walls - Outer (earth) face (left wall, support face)				OK
Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	M _{Ed} - envelope at left wall, support face	115.92 kNm/m	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	M _{rare} ; M _{qp}	80.38 / 39.27 kNm/m	
Cl. 14.3	Effective depth	$d = h - c_{nom} - \phi/2 = 350 - 50 - 12/2$	294.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_m = 0.67 \times 30 / 1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{yd} = f_{yk} / \gamma_s = 500 / 1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{u,lim} = \epsilon_{cu3} / (\epsilon_{cu3} + f_{yd} / E_s) \cdot d = 0.0035 / (0.0035 + 0.00217) \times 294.0$	181.4 mm	
Cl. 8.2	Limiting moment of resistance	$M_{Rd,lim} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{u,lim} \cdot (d - \lambda \cdot x_{u,lim} / 2)$	430.5 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8$, $\eta = 1.0$)	$M_{Ed} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x / 2) \rightarrow$ solve for x	38.8 mm	OK
Cl. 8.2	Tension steel required	$A_{s,req} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{f_{yd}} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 38.8 / 434.8$	957 mm ² /m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{s,min} = \max(0.26 \cdot f_{ctm} / f_{yk} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50 / 500 \times 1000 \times 294, 0.0013 \times 1000 \times 294)$	382 mm ² /m	
	Steel provided	12Ø @ 115 c/c: $A_s = \pi \cdot \phi^2 / 4 \times 1000 / s = \pi \times 12^2 / 4 \times 1000 / 115$	983 mm ² /m ≥ 957	OK
Cl. 16.5.1.1	Maximum steel	$A_s \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 350$	8750 mm ² /m	OK
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 350, 250)$	115 ≤ 250 mm	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, d_g + 5) = \max(12, 20 + 5)$	103 ≥ 25 mm	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_e = E_s / E_{c,eff} = E_s \cdot (1 + \phi) / E_{cm} = 200000 \times 2.00 / 31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_e \cdot \rho + \sqrt{(\alpha_e \cdot \rho)^2 + 2 \cdot \alpha_e \cdot \rho}]$, $\rho = A_s / (b \cdot d) = 0.00335$	74.6 mm	
Cl. 12.2	Cracked moment of inertia	$I_{cr} = b \cdot x^3 / 3 + \alpha_e \cdot A_s \cdot (d - x)^2$	749.2 $\times 10^6$ mm ⁴	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_c = M \cdot x / I_{cr} \leq 0.48 \cdot f_{ck} = 0.48 \times 30$	8.01 ≤ 14.40 MPa	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_s = \alpha_e \cdot M \cdot (d - x) / I_{cr} \leq 0.8 \cdot f_{yk} = 0.8 \times 500$	304 ≤ 400 MPa	OK
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_s = \alpha_e \cdot M_{qp} \cdot (d - x) / I_{cr}$	148.4 MPa	
Cl. 12.3.4	Effective tension area	$h_{c,eff} = \min[2.5(h - d), (h - x)/3, h/2]$; $p_{p,eff} = A_s / (b \cdot h_{c,eff})$	91.8 mm; 0.0107	
Cl. 12.3.4	Mean strain difference	$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot f_{ctm} / p_{p,eff} \cdot (1 + \alpha_e \cdot p_{p,eff})] / E_s \geq 0.6 \cdot \sigma_s / E_s$ ($k_t = 0.4$, $\alpha_e = 6.45$)	0.000445	
Cl. 12.3.4	Maximum crack spacing	$s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / p_{p,eff} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 12 / 0.0107$	360.4 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k,max}$ (moderate exposure)	$0.160 \leq 0.3$ mm	OK

F: Outer walls - Inner (cell) face (right wall, support face)				OK
Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	M _{Ed} - envelope at right wall, support face	58.99 kNm/m	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	M _{rare} ; M _{qp}	38.19 / 12.39 kNm/m	

Reference	Description	Calculation	Result	
Cl. 14.3	Effective depth	$d = h - c_{nom} - \phi/2 = 350 - 50 - 12/2$	294.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_m = 0.67 \times 30 / 1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{yd} = f_{yk} / \gamma_s = 500 / 1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{u,lim} = \epsilon_{cu3} / (\epsilon_{cu3} + f_{yd} / E_s) \cdot d = 0.0035 / (0.0035 + 0.00217) \times 294.0$	181.4 mm	
Cl. 8.2	Limiting moment of resistance	$M_{Rd,lim} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{u,lim} \cdot (d - \lambda \cdot x_{u,lim} / 2)$	430.5 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8, \eta = 1.0$)	$M_{Ed} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x / 2) \rightarrow$ solve for x	19.2 mm	OK
Cl. 8.2	Tension steel required	$A_{s,req} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{f_{yd}} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 19.2 / 434.8$	474 mm ² /m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{s,min} = \max(0.26 \cdot f_{ctm} / f_{yk} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50 / 500 \times 1000 \times 294, 0.0013 \times 1000 \times 294)$	382 mm ² /m	
	Steel provided	12Ø @ 200 c/c: $A_s = \pi \cdot \phi^2 / 4 \times 1000 / s = \pi \times 12^2 / 4 \times 1000 / 200$	565 mm ² /m ≥ 474	OK
Cl. 16.5.1.1	Maximum steel	$A_s \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 350$	8750 mm ² /m	OK
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 350, 250)$	200 $\leq$ 250 mm	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, d_g + 5) = \max(12, 20 + 5)$	188 $\geq$ 25 mm	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_e = E_s / E_{c,eff} = E_s \cdot (1 + \phi) / E_{cm} = 200000 \times 2.00 / 31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_e \cdot \rho + \sqrt{(\alpha_e \cdot \rho)^2 + 2 \cdot \alpha_e \cdot \rho}]$, $\rho = A_s / (b \cdot d) = 0.00192$	58.6 mm	
Cl. 12.2	Cracked moment of inertia	$I_{cr} = b \cdot x^3 / 3 + \alpha_e \cdot A_s \cdot (d - x)^2$	471.4 $\times 10^6$ mm ⁴	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_c = M \cdot x / I_{cr} \leq 0.48 \cdot f_{ck} = 0.48 \times 30$	4.75 $\leq$ 14.40 MPa	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_s = \alpha_e \cdot M \cdot (d - x) / I_{cr} \leq 0.8 \cdot f_{yk} = 0.8 \times 500$	246 $\leq$ 400 MPa	OK
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_s = \alpha_e \cdot M_{qp} \cdot (d - x) / I_{cr}$	79.8 MPa	
Cl. 12.3.4	Effective tension area	$h_{c,eff} = \min[2.5(h - d), (h - x) / 3, h / 2]$; $\rho_{p,eff} = A_s / (b \cdot h_{c,eff})$	97.1 mm; 0.0058	
Cl. 12.3.4	Mean strain difference	$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot f_{ctm} / \rho_{p,eff} \cdot (1 + \alpha_e \cdot \rho_{p,eff})] / E_s \geq 0.6 \cdot \sigma_s / E_s$ ($k_t = 0.4, \alpha_e = 6.45$)	0.000239	
Cl. 12.3.4	Maximum crack spacing	$s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / \rho_{p,eff} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 12 / 0.0058$	520.4 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k,max}$ (moderate exposure)	0.125 $\leq$ 0.3 mm	OK

G: Inner walls - Both faces (inner wall 1, support face)

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	M_{Ed} - envelope at inner wall 1, support face	37.69 kNm/m	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	M_{rare} ; M_{qp}	25.13 / 0.00 kNm/m	
Cl. 14.3	Effective depth	$d = h - c_{nom} - \phi/2 = 300 - 50 - 12/2$	244.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_m = 0.67 \times 30 / 1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{yd} = f_{yk} / \gamma_s = 500 / 1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{u,lim} = \epsilon_{cu3} / (\epsilon_{cu3} + f_{yd} / E_s) \cdot d = 0.0035 / (0.0035 + 0.00217) \times 244.0$	150.5 mm	
Cl. 8.2	Limiting moment of resistance	$M_{Rd,lim} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{u,lim} \cdot (d - \lambda \cdot x_{u,lim} / 2)$	296.6 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8, \eta = 1.0$)	$M_{Ed} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x / 2) \rightarrow$ solve for x	14.8 mm	OK
Cl. 8.2	Tension steel required	$A_{s,req} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{f_{yd}} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 14.8 / 434.8$	364 mm ² /m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{s,min} = \max(0.26 \cdot f_{ctm} / f_{yk} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50 / 500 \times 1000 \times 244, 0.0013 \times 1000 \times 244)$	317 mm ² /m	
	Steel provided	12Ø @ 200 c/c: $A_s = \pi \cdot \phi^2 / 4 \times 1000 / s = \pi \times 12^2 / 4 \times 1000 / 200$	565 mm ² /m ≥ 364	OK
Cl. 16.5.1.1	Maximum steel	$A_s \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 300$	7500 mm ² /m	OK

Reference	Description	Calculation	Result	
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 300, 250)$	$200 \leq 250 \text{ mm}$	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, dg + 5) = \max(12, 20 + 5)$	$188 \geq 25 \text{ mm}$	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_e = E_s/E_{c,eff} = E_s \cdot (1 + \phi)/E_{cm} = 200000 \times 2.00/31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_e \cdot \rho + \sqrt{(\alpha_e \cdot \rho)^2 + 2 \cdot \alpha_e \cdot \rho}]$, $\rho = A_s/(b \cdot d) = 0.00232$	52.8 mm	
Cl. 12.2	Cracked moment of inertia	$I_{cr} = b \cdot x^3/3 + \alpha_e \cdot A_s \cdot (d - x)^2$	$315.8 \times 10^6 \text{ mm}^4$	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_c = M \cdot x/I_{cr} \leq 0.48 \cdot f_{ck} = 0.48 \times 30$	$4.20 \leq 14.40 \text{ MPa}$	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_s = \alpha_e \cdot M \cdot (d - x)/I_{cr} \leq 0.8 \cdot f_{yk} = 0.8 \times 500$	$196 \leq 400 \text{ MPa}$	OK
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_s = \alpha_e \cdot M_{qp} \cdot (d - x)/I_{cr}$	0.0 MPa	
Cl. 12.3.4	Effective tension area	$h_{c,eff} = \min[2.5(h - d), (h - x)/3, h/2]$; $p_{p,eff} = A_s/(b \cdot h_{c,eff})$	82.4 mm; 0.0069	
Cl. 12.3.4	Mean strain difference	$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot f_{ctm}/p_{p,eff} \cdot (1 + \alpha_e \cdot p_{p,eff})]/E_s \geq 0.6 \cdot \sigma_s/E_s$ ($k_t = 0.4$, $\alpha_e = 6.45$)	0.000000	
Cl. 12.3.4	Maximum crack spacing	$s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi/p_{p,eff} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 12/0.0069$	467.2 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k,max}$ (moderate exposure)	$0.000 \leq 0.3 \text{ mm}$	OK

10. Detailing of negative reinforcement (curtailment)

Negative (hogging) steel of the slabs is detailed as alternate bars: continuous bars at twice the design spacing plus extra bars at the supports, cut off where the hogging moment can be resisted by the continuous bars alone. Positive (sagging) steel is continuous.

A, A1, A2: Top slab negative steel - curtailment				OK
Reference	Description	Calculation	Result	
Cl. 16.6.1.1	Continuous (alternate) bars	$16\phi @ 240 \text{ c/c}$: $A_s = 838 \text{ mm}^2/\text{m} \geq A_{s,min} = 380$; $s = 240 \leq \min(2h, 250)$	OK	OK
Cl. 8.2	Moment capacity of continuous bars	$M_{Rd} = A_s \cdot f_{yd} \cdot (d - \lambda x/2) = 838 \times 434.8 \times 278.4/10^6$	101.4 kNm/m	
Cl. 16.5.1.3	Theoretical cut-off of extra bars	point where hogging MEd (ULS envelope) = MRd	see table	
Cl. 16.5.1.3	Shift rule (no shear links)	$a_l = d \geq 12\phi = \max(292, 12 \times 16)$	292 mm	
Cl. 15.2.3	Anchorage length of extra bars	$l_{bd} = (\phi/4) \cdot (f_{yd}/f_{bd})$ for 16ϕ	662 mm	
	Extra bar length from support face	$L = \max(x_{\text{cut-off}} + a_l, l_{bd})$, rounded up to 50 mm	corner 700 / inner 700 mm	
Cl. 15.2.3	Corner bar leg into wall	$leg = l_{bd} \leq \text{clear height}/2$	700 mm	

Top slab: extra bar lengths from support faces

Support	Span side	MEd at face (kNm/m)	Theoretical cut-off (mm)	Length from face (mm)
outer wall (left)	cell 1	41.8	0	700
inner wall 1	cell 1	132.5	218	700
inner wall 1	cell 2	132.5	218	700
outer wall (right)	cell 2	41.8	0	700

C, C1, C2: Bottom slab negative steel - curtailment				OK
Reference	Description	Calculation	Result	
Cl. 16.6.1.1	Continuous (alternate) bars	$20\phi @ 250 \text{ c/c}$: $A_s = 1257 \text{ mm}^2/\text{m} \geq A_{s,min} = 442$; $s = 250 \leq \min(2h, 250)$	OK	OK
Cl. 8.2	Moment capacity of continuous bars	$M_{Rd} = A_s \cdot f_{yd} \cdot (d - \lambda x/2) = 1257 \times 434.8 \times 319.6/10^6$	174.6 kNm/m	
Cl. 16.5.1.3	Theoretical cut-off of extra bars	point where hogging MEd (ULS envelope) = MRd	see table	
Cl. 16.5.1.3	Shift rule (no shear links)	$a_l = d \geq 12\phi = \max(340, 12 \times 20)$	340 mm	

Reference	Description	Calculation	Result	
Cl. 15.2.3	Anchorage length of extra bars	$l_{bd} = (\phi/4) \cdot (f_y d / f_{bd})$ for 20 ϕ	828 mm	
	Extra bar length from support face	$L = \max(x_{\text{cut-off}} + a_l, l_{bd})$, rounded up to 50 mm	corner 850 / inner 850 mm	
Cl. 15.2.3	Corner bar leg into wall	$leg = l_{bd} \leq \text{clear height}/2$	850 mm	

Bottom slab: extra bar lengths from support faces

Support	Span side	MEd at face (kNm/m)	Theoretical cut-off (mm)	Length from face (mm)
outer wall (left)	cell 1	102.6	0	850
inner wall 1	cell 1	76.2	0	850
inner wall 1	cell 2	76.2	0	850
outer wall (right)	cell 2	102.6	0	850

11. Design for shear (IRC 112 Cl. 10.3.2)

Member	Section	h (mm)	d (mm)	VEd (kN/m)	ρ_1	VRd,c (kN/m)	Status
Top slab	cell 1, d from face	350	292	152.6	0.0057	153.5	OK
Bottom slab	cell 1, d from face	400	340	186.2	0.0074	188.0	OK
Outer walls	right wall, d from face	350	294	112.6	0.0033	128.9	OK
Inner walls	inner wall 1, d from face	300	244	23.4	0.0023	109.0	OK

Shear - Top slab

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design shear force at d from face	VEd - envelope, cell 1, d from face	152.55 kN/m	
Cl. 10.3.2	Effective depth	$d = h - c_{nom} - \phi/2 = 350 - 50 - 16/2$	292.0 mm	
Cl. 10.3.2	Size effect factor	$k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/292.0)} \leq 2.0$	1.828	
Cl. 10.3.2	Tension reinforcement ratio	$\rho_1 = A_{sl}/(b_w \cdot d) = 1676/(1000 \times 292.0) \leq 0.02$	0.00574	
Cl. 10.3.2 Eq. 10.1	Shear resistance stress	$v_{Rd,c} = 0.12 \cdot k \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} = 0.12 \times 1.828 \times (80 \times 0.00574 \times 30)^{0.33}$	0.526 MPa	
Cl. 10.3.2 Eq. 10.2	Minimum shear resistance stress	$v_{min} = 0.031 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.031 \times 1.828^{1.5} \times 30^{0.5}$	0.420 MPa	
Cl. 10.3.2	Design shear resistance ($\sigma_{cp} = 0$, conservative)	$VR_{d,c} = \max(v_{Rd,c}, v_{min}) \cdot b_w \cdot d = 0.526 \times 1000 \times 292.0 / 1000$	153.50 $\geq$ 152.55 kN/m	OK

Shear - Bottom slab

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design shear force at d from face	VEd - envelope, cell 1, d from face	186.17 kN/m	
Cl. 10.3.2	Effective depth	$d = h - c_{nom} - \phi/2 = 400 - 50 - 20/2$	340.0 mm	
Cl. 10.3.2	Size effect factor	$k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/340.0)} \leq 2.0$	1.767	
Cl. 10.3.2	Tension reinforcement ratio	$\rho_1 = A_{sl}/(b_w \cdot d) = 2513/(1000 \times 340.0) \leq 0.02$	0.00739	
Cl. 10.3.2 Eq. 10.1	Shear resistance stress	$v_{Rd,c} = 0.12 \cdot k \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} = 0.12 \times 1.767 \times (80 \times 0.00739 \times 30)^{0.33}$	0.553 MPa	
Cl. 10.3.2 Eq. 10.2	Minimum shear resistance stress	$v_{min} = 0.031 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.031 \times 1.767^{1.5} \times 30^{0.5}$	0.399 MPa	
Cl. 10.3.2	Design shear resistance ($\sigma_{cp} = 0$, conservative)	$VR_{d,c} = \max(v_{Rd,c}, v_{min}) \cdot b_w \cdot d = 0.553 \times 1000 \times 340.0 / 1000$	188.02 $\geq$ 186.17 kN/m	OK

Shear - Outer walls

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design shear force at d from face	VEd - envelope, right wall, d from face	112.61 kN/m	
Cl. 10.3.2	Effective depth	$d = h - c_{nom} - \phi/2 = 350 - 50 - 12/2$	294.0 mm	
Cl. 10.3.2	Size effect factor	$k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/294.0)} \leq 2.0$	1.825	

Reference	Description	Calculation	Result	
Cl. 10.3.2	Tension reinforcement ratio	$\rho_1 = A_{sl}/(b_w \cdot d) = 983/(1000 \times 294.0) \leq 0.02$	0.00335	
Cl. 10.3.2 Eq. 10.1	Shear resistance stress	$v_{Rd,c} = 0.12 \cdot k \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} = 0.12 \times 1.825 \times (80 \times 0.00335 \times 30)^{0.33}$	0.438 MPa	
Cl. 10.3.2 Eq. 10.2	Minimum shear resistance stress	$v_{min} = 0.031 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.031 \times 1.825^{1.5} \times 30^{0.5}$	0.419 MPa	
Cl. 10.3.2	Design shear resistance ($\sigma_{cp} = 0$, conservative)	$V_{Rd,c} = \max(v_{Rd,c}, v_{min}) \cdot b_w \cdot d = 0.438 \times 1000 \times 294.0/1000$	128.91 $\geq$ 112.61 kN/m	OK

Shear - Inner walls				OK
Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design shear force at d from face	V_{Ed} - envelope, inner wall 1, d from face	23.44 kN/m	
Cl. 10.3.2	Effective depth	$d = h - c_{nom} - \phi/2 = 300 - 50 - 12/2$	244.0 mm	
Cl. 10.3.2	Size effect factor	$k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/244.0)} \leq 2.0$	1.905	
Cl. 10.3.2	Tension reinforcement ratio	$\rho_1 = A_{sl}/(b_w \cdot d) = 565/(1000 \times 244.0) \leq 0.02$	0.00232	
Cl. 10.3.2 Eq. 10.1	Shear resistance stress	$v_{Rd,c} = 0.12 \cdot k \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} = 0.12 \times 1.905 \times (80 \times 0.00232 \times 30)^{0.33}$	0.405 MPa	
Cl. 10.3.2 Eq. 10.2	Minimum shear resistance stress	$v_{min} = 0.031 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.031 \times 1.905^{1.5} \times 30^{0.5}$	0.447 MPa	
Cl. 10.3.2	Design shear resistance ($\sigma_{cp} = 0$, conservative)	$V_{Rd,c} = \max(v_{Rd,c}, v_{min}) \cdot b_w \cdot d = 0.447 \times 1000 \times 244.0/1000$	108.96 $\geq$ 23.44 kN/m	OK

12. Distribution steel and walls

Distribution reinforcement (along the barrel)

Ma rk	Location	As,main (mm ² /m)	As,re q	Provided	Detailin g	As,pr ov	Rule	Statu s
J	Top slab - Outer (top) face	1676	335	10Ø @ 230 c/c	Design	341	max(0.2×As,main, 0.0005·b·h per face) - IRC 112 cl 16.6.1.1	OK
K	Top slab - Inner (bottom) face	565	175	10Ø @ 250 c/c	Design	314	max(0.2×As,main, 0.0005·b·h per face) - IRC 112 cl 16.6.1.1	OK
L	Bottom slab - Outer (bottom) face	2513	503	12Ø @ 220 c/c	Design	514	max(0.2×As,main, 0.0005·b·h per face) - IRC 112 cl 16.6.1.1	OK
M	Bottom slab - Inner (top) face	870	200	10Ø @ 250 c/c	Design	314	max(0.2×As,main, 0.0005·b·h per face) - IRC 112 cl 16.6.1.1	OK
N	Outer walls - Outer (earth) face	983	246	10Ø @ 250 c/c	Design	314	max(0.25×As,main, 0.0005·b·h per face) - IRC 112 cl 16.3.2	OK
P	Outer walls - Inner (cell) face	565	175	10Ø @ 250 c/c	Design	314	max(0.25×As,main, 0.0005·b·h per face) - IRC 112 cl 16.3.2	OK
Q	Inner walls - Both faces	565	150	10Ø @ 250 c/c	Design	314	max(0.25×As,main, 0.0005·b·h per face) - IRC 112 cl 16.3.2	OK

Axial force in walls (ULS)

Member	N _{Ed,max} (kN/m)	N _{Ed} /(A _c f _{cd})	Remark	Status
Outer walls	320.8	0.068	Axial stress is low - designed as flexural member	OK
Inner walls	380.2	0.095	Axial stress is low - designed as flexural member	OK

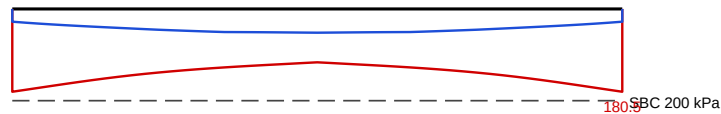
Haunch bars: 12Ø @ 200 c/c at every internal corner (nominal).

13. Base pressure and flotation

Base pressure

Check	Value	Limit	Status
Maximum base pressure, P/A + M/Z (SLS rare)	170.5 kPa	SBC 200 kPa	OK
Minimum base pressure, P/A – M/Z (SLS rare)	17.5 kPa	≥ 0 (no tension)	OK
Peak Winkler spring pressure (SLS rare, info)	180.5 kPa	local, at wall	-
Maximum base pressure, P/A + M/Z (ULS, info)	234.8 kPa	-	-

Bearing is checked with the linear (rigid base) distribution of the resultant of the spring reactions. The Winkler distribution below shows local peaks under the walls.



Base pressure under bottom slab, SLS rare combination (kPa). Red: max, blue: min.

Base pressure distribution (Winkler model)

Check against flotation (cells empty, ground water at 100.500)				OK
Reference	Description	Calculation	Result	
	Head above base	$h = \text{GWL} - \text{base level} = 100.500 - 99.600 = 0.900 \text{ m}$		
	Uplift	$U = \gamma_w h B = 10 \times 0.900 \times 7.000 = 63.00 \text{ kN/m}$		
	Weight of box	$W_c = 208.50 \text{ kN/m}$		
	Weight of fill and surfacing	$W_f = 85.61 \text{ kN/m}$		
	Factor of safety	$\text{FOS} = (W_c + W_f)/U = 4.67 \text{ (required 1.1)}$		

14. Railing / crash barrier

Type	RCC crash barrier (New Jersey profile)
Height above road	0.900 m
Location	Both barrel ends, length 14.00 m each (continued over approach slabs)
Headwall	450 mm thick $\times$ 665 mm high over top slab

Dead load per metre run

Component	Calculation	kN/m
Crash barrier (New Jersey profile)	area $0.2461 \text{ m}^2 \times 25$	6.15
Headwall over top slab	$0.450 \times 0.665 \times 25$	7.48
Total		13.63

In the box analysis the railing load is applied on the top slab as $2 \times 13.63 / 12 \text{ m barrel length} = 2.27 \text{ kPa}$ (case RAIL). Set an edge strip width in the inputs to check the end strip of the barrel instead.

Reinforcement of the barrier/railing is nominal (see BBS and detail sheet). Vehicle collision load on the barrier and its anchorage are not designed; adopt the MoRTH/IRC standard crash barrier details where a containment level is specified.

15. Approach slab and bracket

Approach slab 3.5 m long $\times$ 7.5 m wide $\times$ 300 mm, designed as simply supported between the bracket and the embankment (conservative; soil support ignored).

Approach slab - loads per metre width

Load	Intensity (kPa)	Reaction (kN/m)	Moment (kNm/m)
Self weight	7.50	13.12	11.48
Fill over slab	10.80	18.90	16.54
Wearing course	1.43	2.50	2.19
Live load (envelope)	moving	61.36	54.11

Live load on approach slab

Vehicle	Impact	Reaction (kN/m)	Moment (kNm/m)
IRC Class A	47.4 %	56.55	53.04
IRC Class 70R (Tracked)	25.0 %	61.36	53.69
IRC Class 70R (Wheeled)	25.0 %	58.05	54.11

Approach slab - flexure

Mark	Face	MEd	As,req	As,min	Provided	As,prov	wk (mm)	Status
S1	Bottom face (main)	122.8	1290	312	20Ø @ 190 c/c	1653	0.084	OK
S2	Top face (nominal)	0.0	0	317	12Ø @ 200 c/c	565	0.000	OK

S1: Approach slab - bottom face

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design moment (ULS, basic combination)	MEd - envelope at mid-span	122.83 kNm/m	
IRC 6 Tab. B.3	Service moments (rare / quasi-permanent)	M _{rare} ; M _{qp}	84.76 / 30.65 kNm/m	
Cl. 14.3	Effective depth	$d = h - c_{nom} - \phi/2 = 300 - 50 - 20/2$	240.0 mm	
Cl. 6.4.2.8	Design compressive strength	$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_m = 0.67 \times 30 / 1.5$	13.40 MPa	
Cl. 6.2.2	Design yield strength	$f_{yd} = f_{yk} / \gamma_s = 500 / 1.15$	434.8 MPa	
Cl. 8.2	Limiting neutral axis depth	$x_{u,lim} = \epsilon_{cu3} / (\epsilon_{cu3} + f_{yd}/E_s) \cdot d = 0.0035 / (0.0035 + 0.00217) \times 240.0$	148.0 mm	
Cl. 8.2	Limiting moment of resistance	$M_{Rd,lim} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{u,lim} \cdot (d - \lambda \cdot x_{u,lim}/2)$	286.9 kNm/m	OK
Cl. 8.2, Fig. 6.5	Neutral axis depth ($\lambda = 0.8$, $\eta = 1.0$)	MEd = $\eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x \cdot (d - \lambda \cdot x/2) \rightarrow$ solve for x	52.3 mm	OK
Cl. 8.2	Tension steel required	$A_{s,req} = \eta \cdot f_{cd} \cdot b \cdot \lambda \cdot x_{fcd} / f_{yd} = 1.0 \times 13.40 \times 1000 \times 0.8 \times 52.3 / 434.8$	1290 mm ² /m	
Cl. 16.5.1.1 Eq. 16.1	Minimum tension steel	$A_{s,min} = \max(0.26 \cdot f_{ctm} / f_{yk} \cdot b \cdot d, 0.0013 \cdot b \cdot d) = \max(0.26 \times 2.50 / 500 \times 1000 \times 240, 0.0013 \times 1000 \times 240)$	312 mm ² /m	
	Steel provided	20Ø @ 190 c/c: $A_s = \pi \cdot \phi^2/4 \times 1000/s = \pi \times 20^2/4 \times 1000/190$	1653 mm ² /m $\geq$ 1290	OK
Cl. 16.5.1.1	Maximum steel	$A_s \leq 0.025 \cdot b \cdot h = 0.025 \times 1000 \times 300$	7500 mm ² /m	OK
Cl. 16.6.1.1	Maximum bar spacing	$s \leq \min(2h, 250 \text{ mm}) = \min(2 \times 300, 250)$	190 $\leq$ 250 mm	OK
Cl. 16.2	Minimum clear spacing	$s - \phi \geq \max(\phi, d_g + 5) = \max(20, 20 + 5)$	170 $\geq$ 25 mm	OK
Cl. 12.2, 6.4.2.5	Modular ratio (long-term)	$\alpha_e = E_s/E_{c,eff} = E_s \cdot (1 + \phi)/E_{cm} = 200000 \times 2.00/31000$	12.90	
Cl. 12.2	Cracked section neutral axis	$x = d \cdot [-\alpha_e \cdot \rho + \sqrt{(\alpha_e \cdot \rho)^2 + 2 \cdot \alpha_e \cdot \rho}]$, $\rho = A_s/(b \cdot d) = 0.00689$	82.1 mm	
Cl. 12.2	Cracked moment of inertia	$I_{cr} = b \cdot x^3/3 + \alpha_e \cdot A_s \cdot (d - x)^2$	716.4 $\times 10^6$ mm ⁴	
Cl. 12.2.1	Concrete stress, rare combination	$\sigma_c = M \cdot x / I_{cr} \leq 0.48 \cdot f_{ck} = 0.48 \times 30$	9.71 $\leq$ 14.40 MPa	OK
Cl. 12.2.2	Steel stress, rare combination	$\sigma_s = \alpha_e \cdot M \cdot (d - x) / I_{cr} \leq 0.8 \cdot f_{yk} = 0.8 \times 500$	241 $\leq$ 400 MPa	OK
Cl. 12.3.4	Steel stress, quasi-permanent	$\sigma_s = \alpha_e \cdot M_{qp} \cdot (d - x) / I_{cr}$	87.2 MPa	
Cl. 12.3.4	Effective tension area	$h_{c,eff} = \min[2.5(h - d), (h - x)/3, h/2]$; $\rho_{p,eff} = A_s/(b \cdot h_{c,eff})$	72.6 mm; 0.0228	
Cl. 12.3.4	Mean strain difference	$\epsilon_{sm} - \epsilon_{cm} = [\sigma_s - k_t \cdot f_{ctm} / \rho_{p,eff} \cdot (1 + \alpha_e \cdot \rho_{p,eff})] / E_s \geq 0.6 \cdot \sigma_s / E_s$ ($k_t = 0.4$, $\alpha_e = 6.45$)	0.000262	
Cl. 12.3.4	Maximum crack spacing	$s_{r,max} = 3.4 \cdot c + 0.425 \cdot k_1 \cdot k_2 \cdot \phi / \rho_{p,eff} = 3.4 \times 50 + 0.425 \times 0.8 \times 0.5 \times 20 / 0.0228$	319.4 mm	
Cl. 12.3.4, Table 12.1	Crack width	$w_k = s_{r,max} \cdot (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k,max}$ (moderate exposure)	0.084 $\leq$ 0.3 mm	OK

Shear - approach slab at support

OK

Reference	Description	Calculation	Result	
IRC 6 Tab. B.2	Design shear force at d from face	VEd - envelope, at support (conservative)	139.65 kN/m	
Cl. 10.3.2	Effective depth	$d = h - c_{nom} - \phi/2 = 300 - 50 - 20/2$	240.0 mm	
Cl. 10.3.2	Size effect factor	$k = 1 + \sqrt{(200/d)} = 1 + \sqrt{(200/240.0)} \leq 2.0$	1.913	
Cl. 10.3.2	Tension reinforcement ratio	$\rho_1 = A_{sl}/(b \cdot w \cdot d) = 1653/(1000 \times 240.0) \leq 0.02$	0.00689	
Cl. 10.3.2 Eq. 10.1	Shear resistance stress	$v_{Rd,c} = 0.12 \cdot k \cdot (80 \cdot \rho_1 \cdot f_{ck})^{0.33} = 0.12 \times 1.913 \times (80 \times 0.00689 \times 30)^{0.33}$	0.585 MPa	
Cl. 10.3.2 Eq. 10.2	Minimum shear resistance stress	$v_{min} = 0.031 \cdot k^{1.5} \cdot f_{ck}^{0.5} = 0.031 \times 1.913^{1.5} \times 30^{0.5}$	0.449 MPa	

Reference	Description	Calculation	Result	
Cl. 10.3.2	Design shear resistance ($\sigma_{cp} = 0$, conservative)	$VR_{d,c} = \max(vR_{d,c}, v_{min}) \cdot b_w \cdot d = 0.585 \times 1000 \times 240.0 / 1000$	140.35 $\geq$ 139.65 kN/m	OK
Distribution steel (S3 bottom / S4 top)		10Ø @ 230 c/c / 10Ø @ 250 c/c		
Bracket (corbel) on outer wall				OK
Reference	Description	Calculation	Result	
Geometry	Bracket (corbel) dimensions	projection = 300 mm; hc = 450 mm at wall, 300 mm at free end; bearing at ac = projection/2	ac/hc = 0.33	
IRC 6 Tab. B.2	Characteristic reactions per m	approach slab 13.12 + fill 18.90 + surfacing 2.50 + bracket 2.81; live load 61.36	kN/m	
IRC 6 Tab. B.2	Design vertical load	$VEd = 1.35 \times (13.12 + 2.81) + 1.35 \times 18.90 + 1.75 \times 2.50 + 1.5 \times 61.36$	143.44 kN/m	
EN 1992-1-1 J.3	Horizontal force at bearing	$HEd = 0.2 \cdot VEd$ (restraint / friction)	28.69 kN/m	
Cl. 14.3	Effective depth and lever arm	$d = hc - c_{nom} - \phi/2 = 450 - 50 - 12/2$; $z = 0.8 \cdot d$	d = 394.0; z = 315.2 mm	
Strut-and-tie	Tie force	$F_{td} = VEd \cdot ac/z + HEd \cdot (z + hc - d)/z = 143.44 \times 150/315.2 + 28.69 \times (315.2 + 56.0)/315.2$	102.05 kN/m	
Cl. 6.2.2	Tie steel required	$A_{s,req} = F_{td}/f_{yd} = 102.05 \times 1000/434.8$	235 mm²/m	
Cl. 16.5.1.1	Minimum steel	$A_{s,min} = \max(0.26 \cdot f_{ctm}/f_{yk} \cdot b \cdot d, 0.0013 \cdot b \cdot d)$	512 mm²/m	
	Tie steel provided	R1: 12Ø @ 200 c/c	565 mm²/m	OK
Cl. 10.3.3	Strength reduction factor	$v = 0.6 \cdot (1 - f_{ck}/310) = 0.6 \times (1 - 30/310)$	0.542	
Cl. 10.3.3	Maximum shear (strut crushing)	$VR_{d,max} = 0.5 \cdot v \cdot f_{cd} \cdot b \cdot d = 0.5 \times 0.542 \times 13.40 \times 1000 \times 394.0 / 1000$	1430.6 $\geq$ 143.4 kN/m	OK
EN 1992-1-1 J.3	Horizontal links in upper 2/3-d	$\Sigma A_{s,link} \geq 0.5 \cdot A_{s,main}$ (ac $\leq$ 0.5 hc); R2: 2 layers of 10Ø @ 250 c/c	628 $\geq$ 283 mm²/m	OK
Cl. 16.6.1.1	Maximum tie spacing	$s \leq \min(2 \cdot hc, 250)$	200 mm	OK
Bracket reinforcement		R1 main tie 12Ø @ 200 c/c; R2 links 2 layers of 10Ø @ 250 c/c		
Dowels		25 mm dowels @ 500 c/c, 500 mm long (debonded half)		
Bearing		Bitumen felt strip on bracket seat; 20 mm joint filler between slab and wall		
Effect on box		Reaction applied at the top of each outer wall with eccentricity e = 0.325 m (cases APP and APPLL)		

16. Bar bending schedule

Mark	Member	Description	Shape	Dimensions (mm)	Di	Spa	N	Cut length (m)	Total length (m)	kg/m	Weight (kg)
A	Top slab	Negative bar - outer (top) face, U-bar lapped with wall bars (alternate bars, continuous)	U (inverted)	a=6900, b=1000	16	240	50	8.836	441.8	1.580	698.2
A1	Top slab	Extra negative bar over inner supports, 700 mm each side of wall face, ends cranked 45°	crank	a=1700, c=100	16	240	50	1.900	95.0	1.580	150.1
A2	Top slab	Extra negative corner bar, 700 mm from wall face, leg into wall, slab end cranked 45°	L_crank	a=1000, b=700, c=100	16	240	100	1.768	176.8	1.580	279.4
B	Top slab	Positive bar - inner (bottom) face, continuous, ends bent 90° up into walls	U	a=6750, b=196	12	200	60	7.094	425.6	0.889	378.3
C	Bottom slab	Negative bar - outer (bottom) face, U-bar lapped with wall bars (alternate bars, continuous)	U	a=6900, b=1250	20	250	48	9.320	447.4	2.469	1104.6
C1	Bottom slab	Extra negative bar over inner supports, 850 mm each side of wall face, ends cranked 45°	crank_u	a=2000, c=100	20	250	48	2.200	105.6	2.469	260.7
C2	Bottom slab	Extra negative corner bar, 850 mm from wall face, leg into wall, slab end cranked 45°	L_up_crank	a=1150, b=850, c=100	20	250	96	2.060	197.8	2.469	488.3
D	Bottom slab	Positive bar - inner (top) face, continuous, ends bent 90° down into walls	U (inverted)	a=6750, b=244	12	130	92	7.190	661.5	0.889	588.0

M a r k	Member	Description	Shape	Dimensions (mm)	Di a	Spa cing	N os	Cut length (m)	Total length (m)	kg/ m	Weight (kg)
E	Outer walls	Vertical bar - outer (earth) face	straight	a=3650	12	115	20 8	3.650	759.2	0.8 89	674.8
F	Outer walls	Vertical bar - inner (cell) face, ends bent 90° into slabs	C	a=3512, b=169	12	200	12 0	3.802	456.2	0.8 89	405.5
G	Inner walls	Vertical bar - both faces, ends bent 90° into slabs	C	a=3512, b=150	12	200	12 0	3.764	451.7	0.8 89	401.5
H	Haunches	Haunch bar, straight, parallel to haunch, across the corner to far-face steel	diag	a=950	12	200	48 0	0.950	456.0	0.8 89	405.3
J	Top slab	Distribution - top face	straight	a=11900	10	230	31	11.900	368.9	0.6 17	227.7
K	Top slab	Distribution - bottom face	straight	a=11900	10	250	26	11.900	309.4	0.6 17	191.0
L	Bottom slab	Distribution - bottom face	straight	a=11900	12	220	32	11.900	380.8	0.8 89	338.5
M	Bottom slab	Distribution - top face	straight	a=11900	10	250	26	11.900	309.4	0.6 17	191.0
N	Outer walls	Horizontal - outer face	straight	a=11900	10	250	30	11.900	357.0	0.6 17	220.4
P	Outer walls	Horizontal - inner face	straight	a=11900	10	250	26	11.900	309.4	0.6 17	191.0
Q	Inner walls	Horizontal - both faces	straight	a=11900	10	250	26	11.900	309.4	0.6 17	191.0
S1	Approach slab	Main bar - bottom face (along traffic), 2 slabs	straight	a=3400	20	190	78	3.400	265.2	2.4 69	654.8
S2	Approach slab	Main bar - top face (along traffic), 2 slabs	straight	a=3400	12	200	76	3.400	258.4	0.8 89	229.7
S3	Approach slab	Distribution - bottom face, 2 slabs	straight	a=7400	10	230	30	7.400	222.0	0.6 17	137.0
S4	Approach slab	Distribution - top face, 2 slabs	straight	a=7400	10	250	28	7.400	207.2	0.6 17	127.9
R1	Brackets	Main tie - closed loop on bracket outline, top leg bent down in wall, 2 nos	bracket	a=521, b=220, c=209, d=273, e=247	12	200	76	1.374	104.4	0.8 89	92.8
R2	Brackets	Horizontal hairpin links, 2 layers, legs @ 250, 2 nos	U	a=491, b=250	10	250	60	1.212	72.7	0.6 17	44.9
R3	Brackets	Longitudinal bars at corners of R1 loop (4 per bracket)	straight	a=7400	10	200	8	7.400	59.2	0.6 17	36.5
D W	Brackets	Dowel bracket to approach slab, debonded (sleeved) in the slab half	straight	a=460	25	500	30	0.460	13.8	3.8 58	53.2
C B1	Crash barrier	Vertical U-bar, legs anchored 300 mm into headwall/slab, 2 ends	U (inverted)	a=95, b=1160	12	200	14 0	2.367	331.4	0.8 89	294.6
C B2	Crash barrier	Longitudinal bars, 10 per barrier, 2 ends	straight	a=13920	10	200	20	14.550	291.0	0.6 17	179.6
H W 1	Headwall	Vertical bars both faces, 300 mm into slab and barrier, 2 ends	straight	a=1265	12	200	14 0	1.265	177.1	0.8 89	157.4
H W 2	Headwall	Horizontal bars both faces, 2 ends	straight	a=6900	10	200	16	6.900	110.4	0.6 17	68.1
		Total									9,462.1

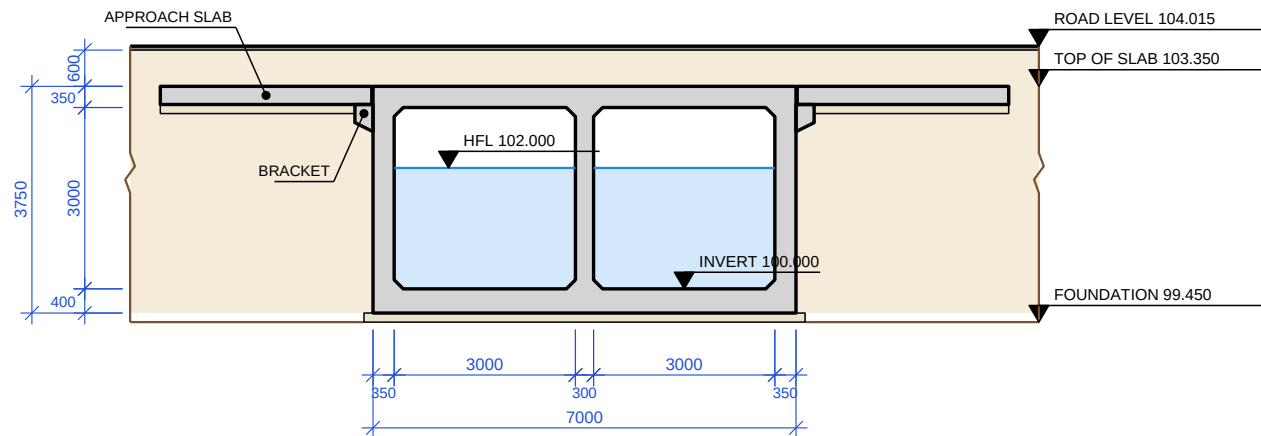
Steel abstract

Diameter	Weight (kg)	Weight (t)
10 mm	1,806.2	1.806
12 mm	3,966.5	3.967
16 mm	1,127.7	1.128
20 mm	2,508.4	2.508
25 mm	53.2	0.053
Total	9,462.1	9.462

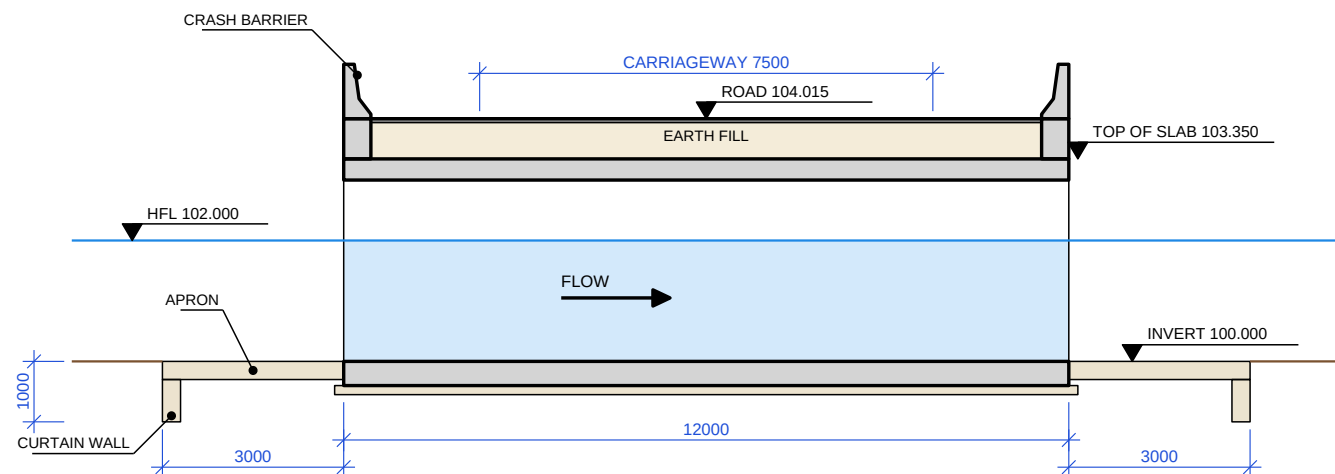
17. Bill of quantities

Rates are as entered by the user (INR). Quantities are computed from the general arrangement; verify against site levels before tendering.

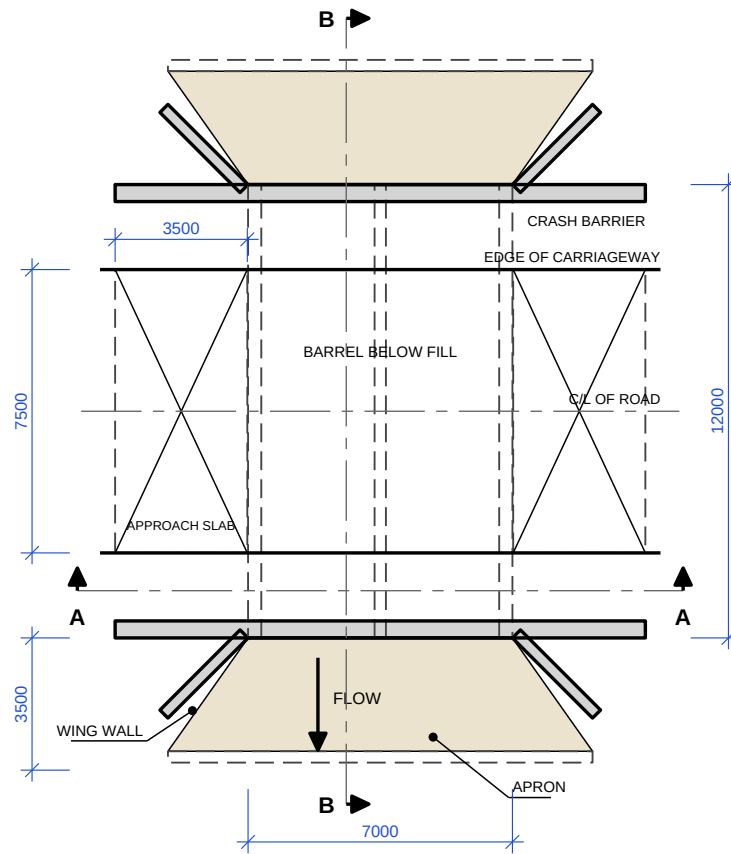
No	Item	Unit	Quantity	Rate	Amount	Basis
1	Earthwork in excavation for foundation in all soils (incl. dewatering)	m ³	88.692	350.00	31,042	8.50×13.20×0.55 + aprons + curtain walls
2	PCC M15 levelling course below base slab	m ³	13.469	6,500.00	87,545	7.30×12.30×0.15
3	RCC M30 in box barrel (slabs, walls, haunches)	m ³	100.080	9,500.00	950,760	net section 8.340 m ² × 12.00 m
4	PCC M15 in aprons (inlet & outlet)	m ³	16.418	6,500.00	106,719	2 × 9.12×3.00×0.30
5	PCC M15 in curtain / cut-off walls	m ³	6.746	6,500.00	43,846	2 × 11.24×0.30×1.00
6	RCC M30 in wing walls (nominal)	m ³	8.550	9,500.00	81,225	4 × 3.00×0.30×2.38 (avg ht)
7	RCC M30 in approach slabs (2 nos)	m ³	15.750	9,500.00	149,625	2 × 3.5×7.5×0.30
8	PCC M15 below approach slabs	m ³	7.155	6,500.00	46,508	2 × 3.18×7.5×0.15
9	RCC M30 in brackets on outer walls (2 nos)	m ³	1.687	9,500.00	16,031	2 × 0.1125 m ² × 7.5
10	Bitumen felt bearing strip with 20 mm joint filler at bracket seat	m	15.000	450.00	6,750	2 × 7.5
11	RCC M30 in crash barrier, both ends	m ³	6.890	9,500.00	65,458	2 × 0.2461 m ² /m × 14.00
12	RCC M30 in headwalls over top slab	m ³	4.190	9,500.00	39,800	2 × 0.450×0.665×7.00
13	Formwork to crash barrier and headwalls	m ²	87.640	750.00	65,730	2 ends × 2 faces × 14.00 × 1.56
14	Reinforcement steel (HYSD) as per BBS (barrel, approach slabs, brackets, railing)	t	9.462	85,000.00	804,276	9,462 kg from BBS
15	Formwork / shuttering to barrel (cells, outer walls, ends)	m ²	391.145	750.00	293,359	cells 2×11.65×12.00 + outer faces + ends
16	Selected granular backfill behind walls and over top slab	m ³	185.400	450.00	83,430	2×1.5×3.75×12.00 + 7.00×0.60×12.00
17	Wearing course 65 mm over culvert portion	m ²	52.500	1,200.00	63,000	7.00×7.50
	Total				2,935,106	



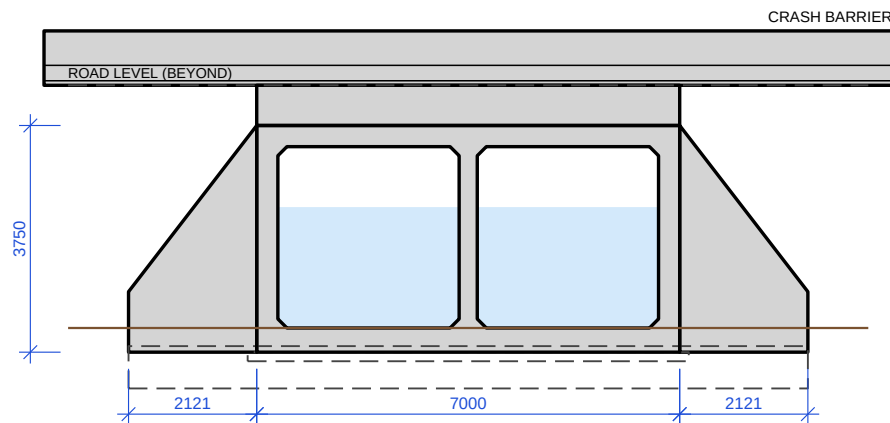
CROSS SECTION A-A
SCALE 1:125



LONGITUDINAL SECTION B-B
SCALE 1:125



PLAN
SCALE 1:200



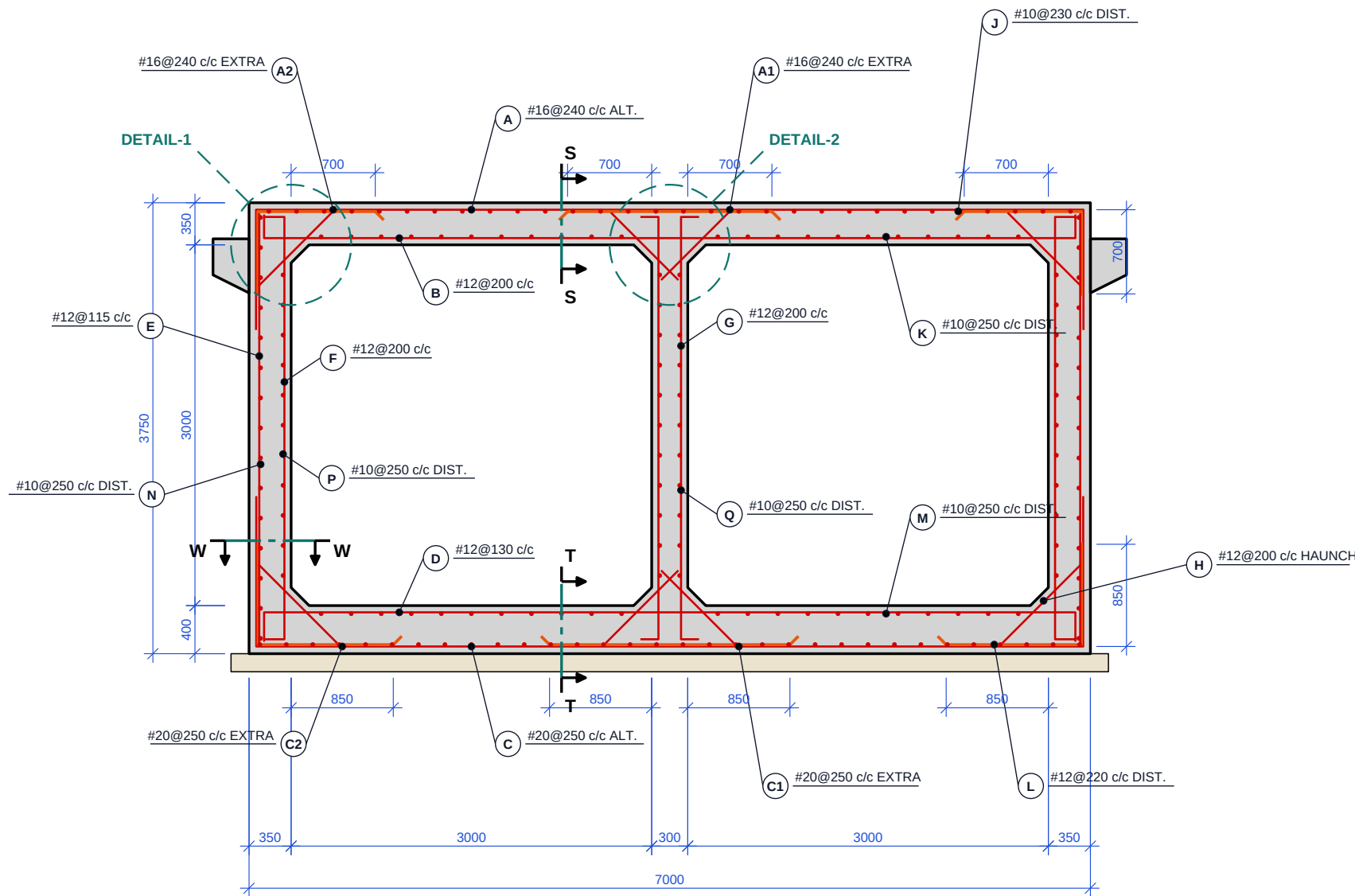
ELEVATION (OUTLET END)
SCALE 1:125

NOTES

1. All dimensions are in mm and levels in metres unless noted otherwise.
2. Concrete: barrel M30, levelling course / apron / curtain walls M15 (PCC).
3. Reinforcement: HYSD bars Fe500 conforming to IS 1786. Clear cover 50 mm.
4. Live load: IRC Class A (2 lane), 70R tracked, 70R wheeled; fill over top slab 600 mm.
5. Skew angle 0 deg (drawings shown square to the barrel). Haunches 150x150 at all internal corners.
6. Wing walls, aprons and curtain walls are nominal; design separately if required.
7. Weep holes / filter media behind walls as per MoRTH specifications.
8. Crash barrier at both barrel ends, 900 mm above road.
9. Approach slabs 3.5 m long on brackets cast with outer walls (see details).

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BOX CULVERT				
GENERAL ARRANGEMENT				
CLIENT			STRUCTURE	
-			2 x 3 m x 3 m RCC BOX	
DESIGNED	CHECKED	DATE	SCALE	SHEET
-	-	31-10-2026	AS SHOWN	1 of 5



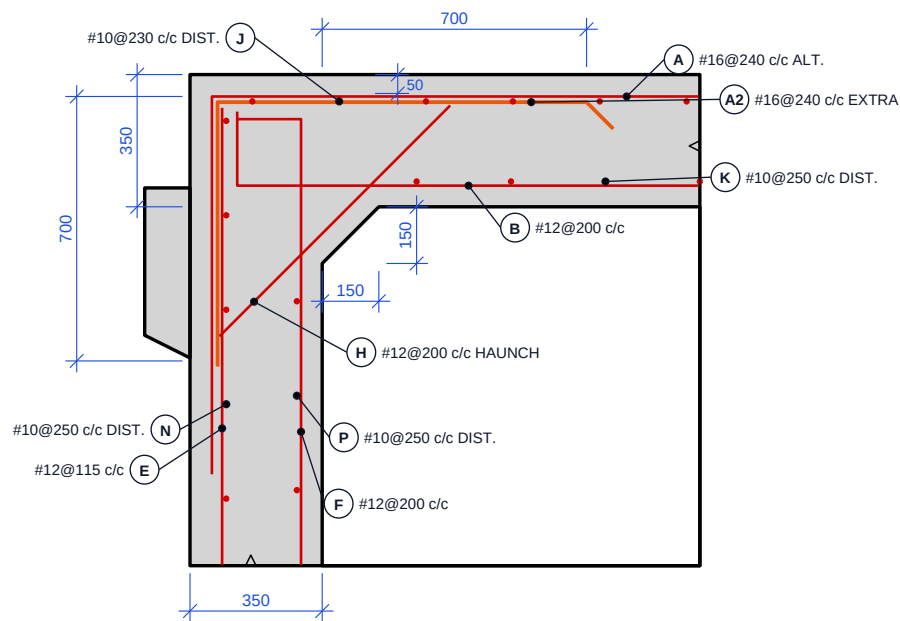
TYPICAL CROSS SECTION - REINFORCEMENT
SCALE 1:50

REINFORCEMENT SCHEDULE		
MARK	LOCATION	BARS
A	Top slab: outer (top) face, U-bar lapped with wall bars (alter	#16@240 c/c
A1	Top slab: Extra negative bar over inner supports, 700 mm each	#16@240 c/c
A2	Top slab: Extra negative corner bar, 700 mm from wall face, le	#16@240 c/c
B	Top slab: inner (bottom) face, continuous, ends bent 90° up in	#12@200 c/c
C	Bottom slab: outer (bottom) face, U-bar lapped with wall bars	#20@250 c/c
C1	Bottom slab: Extra negative bar over inner supports, 850 mm ea	#20@250 c/c
C2	Bottom slab: Extra negative corner bar, 850 mm from wall face,	#20@250 c/c
D	Bottom slab: inner (top) face, continuous, ends bent 90° down	#12@130 c/c
E	Outer walls: outer (earth) face	#12@115 c/c
F	Outer walls: inner (cell) face, ends bent 90° into slabs	#12@200 c/c
G	Inner walls: both faces, ends bent 90° into slabs	#12@200 c/c
H	Haunches: Haunch bar, straight, parallel to haunch, across the	#12@200 c/c
J	Top slab: top face	#10@230 c/c
K	Top slab: bottom face	#10@250 c/c
L	Bottom slab: bottom face	#12@220 c/c
M	Bottom slab: top face	#10@250 c/c
N	Outer walls: outer face	#10@250 c/c
P	Outer walls: inner face	#10@250 c/c
Q	Inner walls: both faces	#10@250 c/c

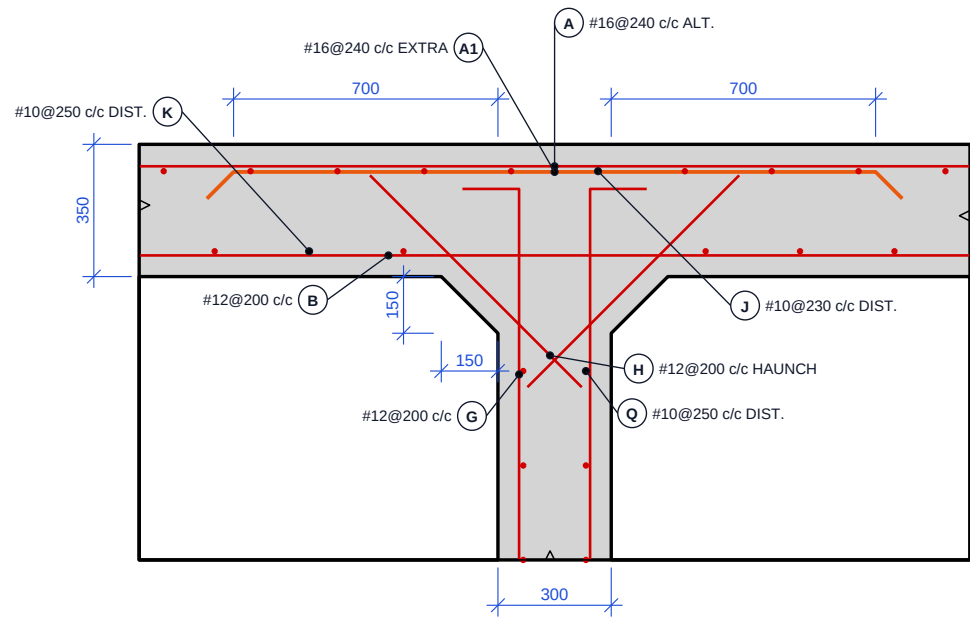
- NOTES
- Clear cover to main reinforcement 50 mm on all faces.
 - Concrete M30; steel Fe500. Main bars placed in the outer layer.
 - Outer-face U-bars A and C lap with wall bars E; lap length = 1000 mm for mark A (#16).
 - Negative steel: A/C continuous (alternate) + A1/C1 over inner supports + A2/C2 at corners (orange); lengths from support faces.
 - Positive steel B and D continuous. Distribution bars J to Q run along the barrel.
 - Laps staggered, max 50% at a section. B, D, F, G anchored with 90° bends; extra bars cranked 45°.
 - See Reinforcement Details - 2 for DETAIL-1, DETAIL-2 and sections S-S, T-T, W-W.

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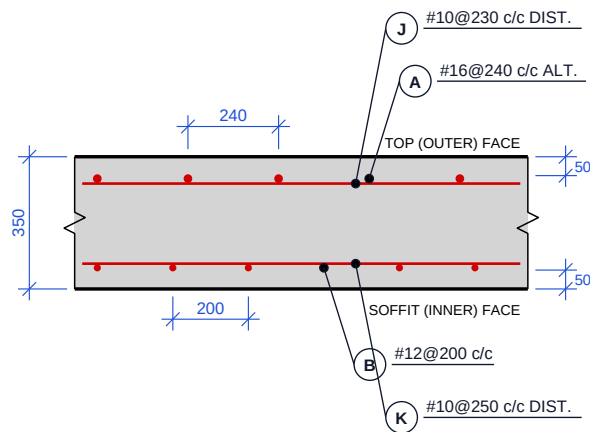
BOX CULVERT				
REINFORCEMENT DETAILS				
CLIENT			STRUCTURE	
-			2 x 3 m x 3 m RCC BOX	
DESIGNED	CHECKED	DATE	SCALE	SHEET
-	-	31-10-2026	AS SHOWN	2 of 5



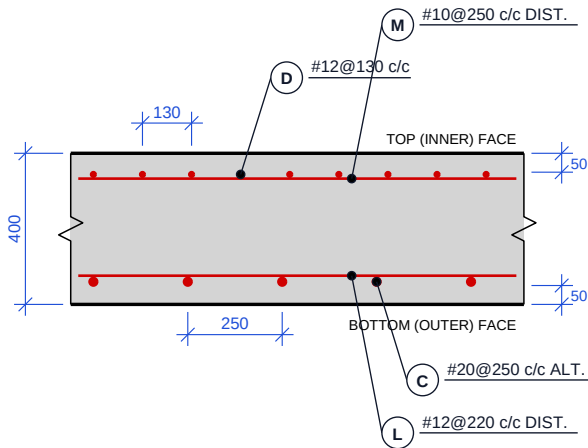
DETAIL-1: OUTER CORNER AND HAUNCH
SCALE 1:20



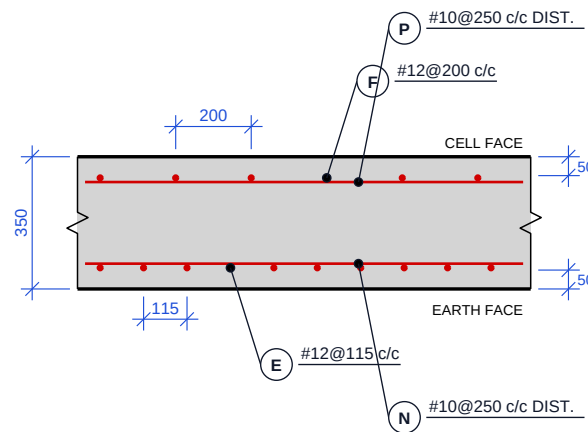
DETAIL-2: INNER WALL JUNCTION AND HAUNCHES
SCALE 1:20



SECTION S-S (TOP SLAB)
SCALE 1:20



SECTION T-T (BOTTOM SLAB)
SCALE 1:20



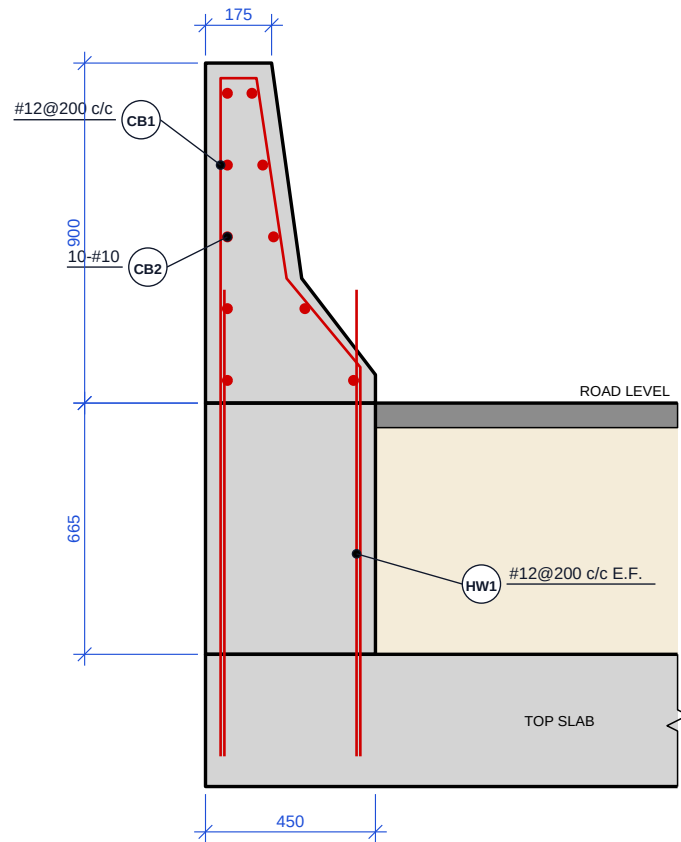
SECTION W-W (OUTER WALL)
SCALE 1:20

NOTES

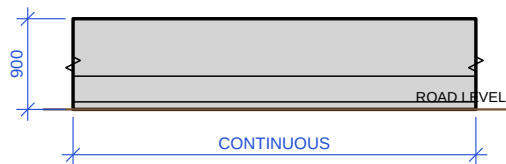
1. Clear cover 50 mm to the outermost bars on all faces; main bars in the outer layer.
2. Bar marks as per the reinforcement schedule and BBS. Anchorage bends 90°; extra bars cranked 45°.
3. Negative (hogging) slab steel: alternate continuous bars (A, C) with extra bars (A1, A2, C1, C2) over the supports; lengths dimensioned from the support faces.
4. Haunch 150 x 150 at all internal corners; haunch bars H straight, parallel to the haunch at cover, extended across the corner to the far-face reinforcement of slab and wall (MOST standard arrangement).
5. Sections S-S, T-T and W-W are taken at mid-span / mid-height as marked on the cross section.

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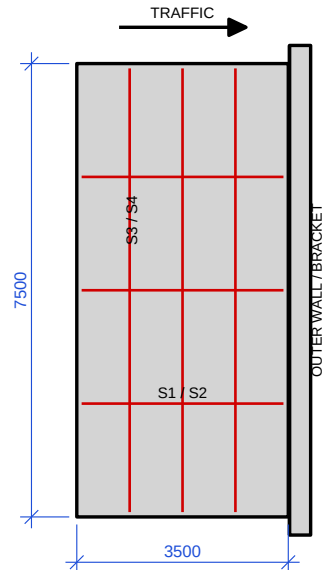
BOX CULVERT				
REINFORCEMENT DETAILS - 2				
CLIENT			STRUCTURE	
-			2 x 3 m x 3 m RCC BOX	
DESIGNED	CHECKED	DATE	SCALE	SHEET
-	-	31-10-2026	AS SHOWN	3 of 5



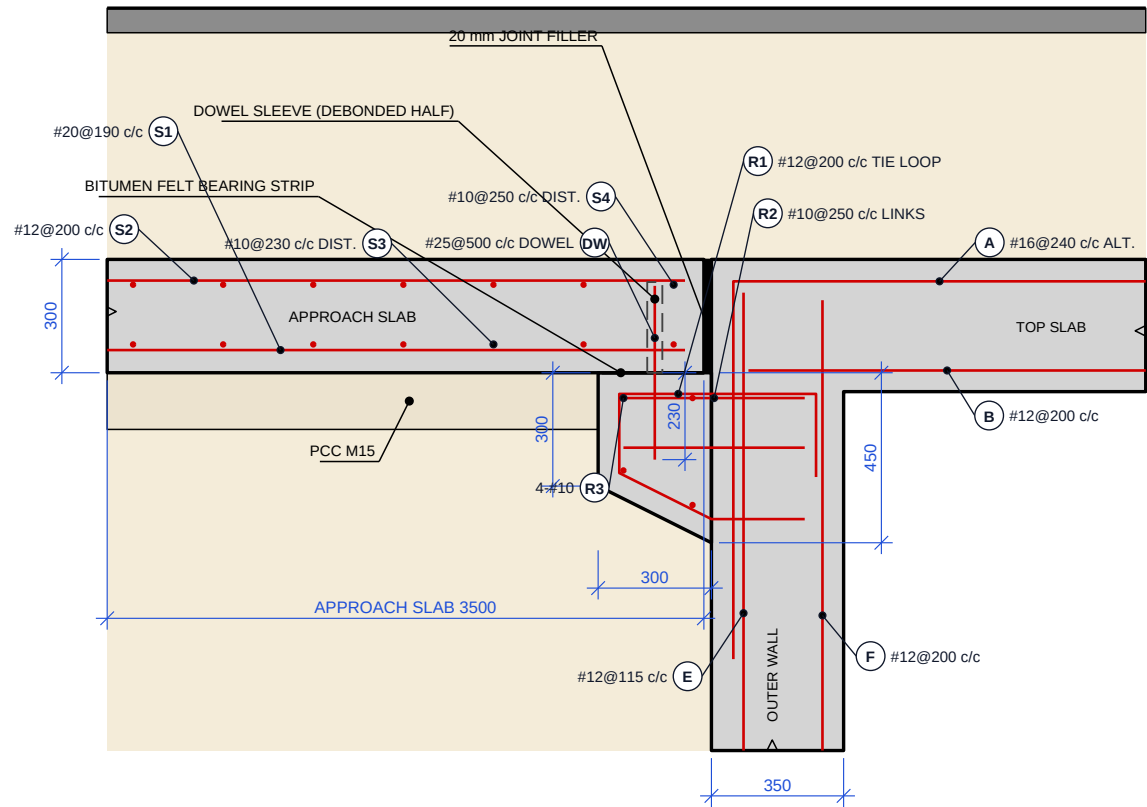
CRASH BARRIER - SECTION
SCALE 1:20



CRASH BARRIER - ELEVATION
SCALE 1:75



APPROACH SLAB - PLAN
SCALE 1:125



BRACKET AND APPROACH SLAB - SECTION
SCALE 1:20

SCHEDULE OF REINFORCEMENT

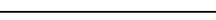
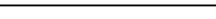
MARK	LOCATION	BARS
S1	Approach slab: Main bar - bottom face (along traffic), 2 slabs	#20@190 c/c
S2	Approach slab: Main bar - top face (along traffic), 2 slabs	#12@200 c/c
S3	Approach slab: Distribution - bottom face, 2 slabs	#10@230 c/c
S4	Approach slab: Distribution - top face, 2 slabs	#10@250 c/c
R1	Brackets: Main tie - closed loop on bracket outline, top leg bent down in wall, 2 nos	#12@200 c/c
R2	Brackets: Horizontal hairpin links, 2 layers, legs @ 250, 2 nos	#10@250 c/c
R3	Brackets: Longitudinal bars at corners of R1 loop (4 per bracket)	#10@200 c/c
DW	Brackets: Dowel bracket to approach slab, debonded (sleeved) in the slab half	#25@500 c/c
CB1	Crash barrier: Vertical U-bar, legs anchored 300 mm into headwall/slab, 2 ends	#12@200 c/c
CB2	Crash barrier: Longitudinal bars, 10 per barrier, 2 ends	#10@200 c/c
HW1	Headwall: Vertical bars both faces, 300 mm into slab and barrier, 2 ends	#12@200 c/c
HW2	Headwall: Horizontal bars both faces, 2 ends	#10@200 c/c

Design: IRC 112:2020, IRC 6:2017, IRC SP 13 | Generated by BoxCulvert Designer

BOX CULVERT				
RAILING & APPROACH SLAB				
CLIENT			STRUCTURE	
-			2 x 3 m x 3 m RCC BOX	
DESIGNED	CHECKED	DATE	SCALE	SHEET
-	-	31-10-2026	AS SHOWN	4 of 5

- NOTES**
- Crash barrier reinforcement is nominal; collision design per IRC 6 / MoRTH standard drawings where a containment level is specified.
 - Cover to railing / barrier reinforcement 40 mm.
 - Approach slab 3.5 m × 7.5 m × 300 mm on 150 mm PCC M15 over compacted granular fill.
 - Bracket designed as corbel: VEd = 143.4 kN/m, main tie R1 #12@200 c/c (closed loop, MOST type).
 - Dowels debonded (sleeved) over half length to allow rotation of the approach slab.

BAR BENDING SCHEDULE - BOX CULVERT (BARREL LENGTH 12 m)

MARK	MEMBER	DESCRIPTION	SHAPE	DIMENSIONS (mm)	DIA	SPACING	NOS	CUT LEN (m)	TOTAL (m)	kg/m	WEIGHT (kg)
A	Top slab	Negative bar - outer (top) face, U-bar lapped with wall bars (alternate bars, co		a=6900, b=1000	16	240	50	8.836	441.8	1.580	698.2
A1	Top slab	Extra negative bar over inner supports, 700 mm each side of wall face, ends cran		a=1700, c=100	16	240	50	1.900	95.0	1.580	150.1
A2	Top slab	Extra negative corner bar, 700 mm from wall face, leg into wall, slab end cranke		a=1000, b=700, c=100	16	240	100	1.768	176.8	1.580	279.4
B	Top slab	Positive bar - inner (bottom) face, continuous, ends bent 90° up into walls		a=6750, b=196	12	200	60	7.094	425.6	0.889	378.3
C	Bottom slab	Negative bar - outer (bottom) face, U-bar lapped with wall bars (alternate bars,		a=6900, b=1250	20	250	48	9.320	447.4	2.469	1104.6
C1	Bottom slab	Extra negative bar over inner supports, 850 mm each side of wall face, ends cran		a=2000, c=100	20	250	48	2.200	105.6	2.469	260.7
C2	Bottom slab	Extra negative corner bar, 850 mm from wall face, leg into wall, slab end cranke		a=1150, b=850, c=100	20	250	96	2.060	197.8	2.469	488.3
D	Bottom slab	Positive bar - inner (top) face, continuous, ends bent 90° down into walls		a=6750, b=244	12	130	92	7.190	661.5	0.889	588.0
E	Outer walls	Vertical bar - outer (earth) face		a=3650	12	115	208	3.650	759.2	0.889	674.8
F	Outer walls	Vertical bar - inner (cell) face, ends bent 90° into slabs		a=3512, b=169	12	200	120	3.802	456.2	0.889	405.5
G	Inner walls	Vertical bar - both faces, ends bent 90° into slabs		a=3512, b=150	12	200	120	3.764	451.7	0.889	401.5
H	Haunches	Haunch bar, straight, parallel to haunch, across the corner to far-face steel		a=950	12	200	480	0.950	456.0	0.889	405.3
J	Top slab	Distribution - top face		a=11900	10	230	31	11.900	368.9	0.617	227.7
K	Top slab	Distribution - bottom face		a=11900	10	250	26	11.900	309.4	0.617	191.0
L	Bottom slab	Distribution - bottom face		a=11900	12	220	32	11.900	380.8	0.889	338.5
M	Bottom slab	Distribution - top face		a=11900	10	250	26	11.900	309.4	0.617	191.0
N	Outer walls	Horizontal - outer face		a=11900	10	250	30	11.900	357.0	0.617	220.4
P	Outer walls	Horizontal - inner face		a=11900	10	250	26	11.900	309.4	0.617	191.0
Q	Inner walls	Horizontal - both faces		a=11900	10	250	26	11.900	309.4	0.617	191.0
S1	Approach slab	Main bar - bottom face (along traffic), 2 slabs		a=3400	20	190	78	3.400	265.2	2.469	654.8
S2	Approach slab	Main bar - top face (along traffic), 2 slabs		a=3400	12	200	76	3.400	258.4	0.889	229.7
S3	Approach slab	Distribution - bottom face, 2 slabs		a=7400	10	230	30	7.400	222.0	0.617	137.0
S4	Approach slab	Distribution - top face, 2 slabs		a=7400	10	250	28	7.400	207.2	0.617	127.9
R1	Brackets	Main tie - closed loop on bracket outline, top leg bent down in wall, 2 nos		a=521, b=220, c=209, d=273, e=247	12	200	76	1.374	104.4	0.889	92.8
R2	Brackets	Horizontal hairpin links, 2 layers, legs @ 250, 2 nos		a=491, b=250	10	250	60	1.212	72.7	0.617	44.9
R3	Brackets	Longitudinal bars at corners of R1 loop (4 per bracket)		a=7400	10	200	8	7.400	59.2	0.617	36.5
DW	Brackets	Dowel bracket to approach slab, debonded (sleeved) in the slab half		a=460	25	500	30	0.460	13.8	3.858	53.2
CB1	Crash barrier	Vertical U-bar, legs anchored 300 mm into headwall/slab, 2 ends		a=95, b=1160	12	200	140	2.367	331.4	0.889	294.6
CB2	Crash barrier	Longitudinal bars, 10 per barrier, 2 ends		a=13920 (+1 lap)	10	200	20	14.550	291.0	0.617	179.6
HW1	Headwall	Vertical bars both faces, 300 mm into slab and barrier, 2 ends		a=1265	12	200	140	1.265	177.1	0.889	157.4
HW2	Headwall	Horizontal bars both faces, 2 ends		a=6900	10	200	16	6.900	110.4	0.617	68.1

STEEL ABSTRACT

DIAMETER (mm)	WEIGHT (kg)
#10	1,806.2
#12	3,966.5
#16	1,127.7
#20	2,508.4
#25	53.2
TOTAL	9,462.1 kg (9.462 t)

NOTES

1. Cutting lengths include bend deductions of 2ϕ per 90° bend.
2. Lap length per IRC 112 cl 15.2.5 (α6 = 1.5); stock length 12 m.
3. Wing walls excluded. Railing / barrier reinforcement is nominal.

Design: IRC 112:2020, IRC 6:2017, IRC SP 13 | Generated by BoxCulvert Designer

BOX CULVERT				
BAR BENDING SCHEDULE				
CLIENT			STRUCTURE	
-			2 x 3 m x 3 m RCC BOX	
DESIGNED	CHECKED	DATE	SCALE	SHEET
-	-	31-10-2026	NTS	5 of 5