	Project details				
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Example

A 200 mm thick C30/37 slab is required to support screed, finishes, an office variable action of 2.5 kN/m² and demountable partitions at 2 kN/m. The slab is supported on load-bearing block walls. Assume a 50-year design life and a requirement for 1 hour resistance to fire. Also, assume f_{yk} = 500 MPa.

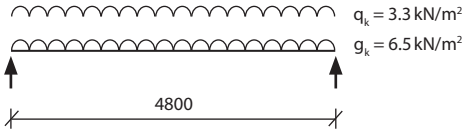


Figure H1.1b.1: Simply supported one-way slab

General notes:

The same 200 mm thick slab in Worked Example H1.1 is redesigned using the alternative simplified explicit verification method for deflections (Clause 9.3.3), the refined crack width calculation method (Clause 9.2.3), and anchorage and lap lengths using Formula (11.3) of Eurocode 2 Part 1-1.

All calculations are carried out in a spreadsheet format where the outcome is not rounded and it is the unrounded value that is used in any further calculation.

This worked example is based on the versions of the codes of practice, guidance documents and publications as noted in the List of References. The user is responsible for ensuring that they use the latest version of the documents.

Abbreviations

General:	Cl.	Clause
	F.	Formula
	Fig.	Figure
	WE	Worked example
Codes of practice:	EC0-1	BS EN 1990-1:2023+A1:2026 [1]
	EC2-1-1	BS EN 1992-1-1:2023 [2]
	NA EC0-1	NA to BS EN 1990-1:2023+A1:2026 [3]
	NA EC2-1-1	NA to BS EN 1992-1-1:2026 [4]
	PD 6687	PD 6687:2026 [5]
Guidance documents:	H1. How to	H1 - How to: Solid slabs (2026) [6]
	H7. How to	H7 - How to: Deflection calculations (2026) [7]
	H8. How to	H8 - How to: Cracking calculations (2026) [8]
	SMDSC	Standard method of detailing structural concrete [9]
	WE H1.1	Simply supported one-way solid slab [10]

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H1.1b.1 Actions

As before (from WE H1.1 Section H1.1.1):

Permanent load: g_k = **6.5 kN/m²**

Variable load: q_k = **3.3 kN/m²**

H1.1b.2 Load combinations (and arrangement)

As before (from WE H1.1 Section H1.1.3):

Using the most adverse of the upper and lower part of F. (8.13) of EC0-1:

Ultimate load, n = **13.1 kN/m²**

H1.1b.3 Cover

As before (from WE H1.1 Section H1.1.2):

Cover for durability:

c_{nom} = **25 mm**

This is also adequate for a 1 hour fire resistance

H1.1b.4 Analysis

Design moment:

$M_{Ed} = 13.1 \times 4.8^2/8$ = **37.6 kNm/m**

Shear force:

At face of support:

$V = 13.1 \times 4.8/2$ = 31.4 kN/m

At distance 'd' away from the face of the support*: EC2-1-1: Fig. 8.6

$V_{Ed} = V - dn$

where:

d = effective depth = $h - c - \phi/2 = 200 - 25 - 12/2$ = **169 mm**

$V_{Ed} = 31.4 - (0.169 \times 13.1)$ = **29.2 kN/m**

H1.1b.5 Flexural design

For C30/37:

$f_{cd} = \eta_{cc} k_{tc} f_{ck}/\gamma_C$ EC2-1-1: F. (5.3)

where:

$\eta_{cc} = (f_{ck,ref}/f_{ck})^{1/3} \leq 1.0$ EC2-1-1: F. (5.4)

k_{tc} (assuming that the design loading is expected after 3 months from casting**) = 1.00 EC2-1-1: Cl. 5.1.6 (1)

$f_{ck,ref}$ = 40 MPa EC2-1-1: Cl. 5.1.6 (1)

γ_C = 1.5 EC2-1-1: Table 4.3

$\therefore \eta_{cc} = (40/30)^{1/3}$ but ≤ 1.0 = 1.00

$\therefore f_{cd} = 1 \times 1 \times 30/1.5$ = 20.0 MPa

(Alternatively, use How to look-up tables for C30/37 and loading time ≥ 3 months) H1. How to: Table H1.5

* EC2-1-1 allows the use of a reduced design shear force that is calculated at 'd' away from the support for members without shear reinforcement. In this WE, the shear force reduction equals 7%. However, it is worth noting that in this case, the design would have been unchanged if the full shear force at the face of the support were designed for (see Section H1.1b.6).

** Refer to k_{tc} explanation in WE H1.1 Section H1.1.5.

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Flexure in span:

$K = M_{Ed}/(bd^2f_{cd})$

$= 37.6 \times 10^6/(1000 \times 169^2 \times 20)$ = 0.066 — H1. How to: Fig. H1.3

0% redistribution, $\therefore \delta_M$ = 1.0

$K_{lim} = 1.10\delta_M - 0.32\delta_M^2 - 0.45$ — H1. How to: Fig. H1.3

$\therefore K_{lim} = 1.10 \times 1 - 0.32 \times 1^2 - 0.45$ = 0.334

(Alternatively, use How to look-up tables for $\delta_M = 1.0$) — H1. How to: Table H1.6

$\therefore K < K_{lim}$

$\therefore$ use K = 0.066

$\therefore z = d/2\,[1+\sqrt{(1-2K)}] \le 0.95d$

$= (169/2) \times [1 + \sqrt{(1 - 2 \times 0.066)}]$ = 163 mm

but $\le (0.95 \times 169)$ = 161 mm

$\therefore z$ = 161 mm

(Alternatively, use How to look-up tables to determine z/d for K = 0.066 < 0.095) — H1. How to: Table H1.7

Tension reinforcement:

$A_{s,req} = M_{Ed}/f_{yd}z$

where:

f_{yk} (given) = 500 MPa

$f_{yd} = 500/1.15$ = 435 MPa — EC2-1-1: Table 4.3

$\therefore A_{s,req} = (37.6 \times 10^6)/(435 \times 161)$ = **539 mm²/m** — H1. How to: Fig. H1.3

$\therefore \rho = A_{s,req}/(bd) = 539/(1000 \times 169)$ = 0.32%

Try H12-200 B1 (565 mm²/m)

H1.1b.6 Shear

V_{Ed} (from Section H1.1b.4) = 29.2 kN/m

$\tau_{Ed} = V_{Ed}/(b_w \times 0.9d) = (29.2 \times 1000)/(1000 \times 0.9 \times 169)$ = 0.19 MPa* — EC2-1-1: F. (8.18) & H1. How to: Fig. H1.4

The shear resistance neglecting shear reinforcement is: — EC2-1-1: F. (8.20)

$\tau_{Rdc,min} = 11/\gamma_V \sqrt{[(f_{ck}/f_{yd}) \times (d_{dg}/d)]}$

where: — NA EC2-1-1:

Assuming D_{lower} = 20 mm — Cl. NA.4.29

$d_{dg} = 16 + 20$ = 36 mm — EC2-1-1: Cl. 8.2.1 (4)

γ_V = 1.4 — EC2-1-1: Table 4.3

$\tau_{Rdc,min} = 11/1.4 \times \sqrt{[(30/435) \times (36/169)]}$ = **0.95 MPa**

(Alternatively, use How to look-up tables for d = 169 mm and C30/37) — H1. How to: Table H1.8

$\therefore \tau_{Ed} < \tau_{Rdc,min}$

$\therefore$ Detailed verification of the shear resistance may be omitted — EC2-1-1: Cl. 8.2.1 (1a)

$\therefore$ Section has sufficient shear capacity

* If the design shear force at the face of the support is used, the design shear stress is 0.21 MPa < 0.95 MPa i.e. the section's shear capacity is sufficient, as noted in Section H1.1b.4.

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H1.1b.7 Deflection

Using the simplified explicit verification method for deflections: — H7. How to: Fig. H7.4

Geometrical properties:

The distance from the top fibres to the neutral axis of the uncracked section:

$x_u = [A_c(h/2) + (\alpha_e - 1)(A_{s,prov}d)]/[A_c + (\alpha_e - 1)A_{s,prov}]$ — H7. How to: Panel H7.2

where:

$A_c = bh = 1000 \times 200 = 200{,}000$ mm²/m

$\alpha_e = \alpha_{e,ef} = E_s/E_{c,eff}$

where:

$E_s = 200{,}000$ MPa — EC2-1-1: Cl. 5.2.4 (3)

$E_{c,eff} = 1.05\, E_{cm}/[1 + \phi(t, t_0)]$ — EC2-1-1: F. (9.1)

where:

$E_{cm} = k_E f_{cm}^{1/3}$ — EC2-1-1: F. (5.1)

$f_{cm} = f_{ck} + 8 = 30 + 8 = 38$ MPa — EC2-1-1: Table 5.1

Assume quartzite aggregates, $\therefore k_E = 9{,}500$ — EC2-1-1: Cl. 5.1.4 (2)

$\therefore E_{cm} = 9500 \times 38^{1/3} = 31{,}939$ MPa

(Alternatively, use How to look-up tables for C30/37) — H7. How to: Table H7.2

> *Note:*
>
> *Table 5.2 of EC2-1-1 can be used to assess the creep coefficient $\phi(t, t_0)$ if the compressive stress under the quasi-permanent combinations < $0.4f_{cm}(t_0)$. A check to confirm that this condition is met is included at the end of the calculation.*

Notional size $h_n = 2A_c/u$ — EC2-1-1: Table 5.2

Assume one slab face is exposed to drying

$\therefore$ Perimeter of exposed part, u = 1000 mm

$\therefore$ Notional size h_n:

$= 2 \times (1000 \times 200)/1000 = 400$ mm

Relative humidity for indoor slab, RH = 50% — EC2-1-1: Table 5.2

For Class CR, use $t_0 = 3$ days* and RH = 50%:

$\phi(t, t_0) = \phi(50y, 3)$ for $f_{ck,28} = 35$ MPa** = 2.6 — EC2-1-1: Table 5.2

Correction exponent A = 0.76 — EC2-1-1: Table 5.2

$\therefore \phi(t, t_0) = \phi(50y, 3)$

$= 2.6 \times (35/30)^{0.76} = 2.9$

$\therefore E_{c,eff} = 1.05 \times 31939/(1 + 2.9) = 8{,}545$ MPa

$\therefore \alpha_e = 200000/8545 = 23.4$

$\therefore x_u = [200000 \times (200/2) + (23.4 - 1)(565 \times 169)]/[200000 + (23.4 - 1) \times 565]$

$= \mathbf{104.1\ mm}$

The second moment of area of the gross concrete cross-section:

$I_g = bh^3/12 = 1000 \times 200^3/12 = \mathbf{667E + 06\ mm^4}$ — H7. How to: Panel H7.2

Long-term deflections: — EC2-1-1: Cl. 9.3.3 (1)

$\delta = k_l(\delta_{loads} + k_s\delta_{ecs})$ — EC2-1-1: F. (9.23)

---

* For simplicity, it is assumed that the entire quasi-permanent action is applied at the same time as the self-weight. This can be further refined for the creep calculation by considering the slab self-weight to be applied at 3 days and the remaining quasi-permanent action to be applied at 90 days.

** The tabulated values for creep coefficients in Table 5.2 of EC2-1-1 correspond to C35/40 concrete. For different concrete grades, these values should be multiplied by $(35/f_{ck})^A$, where A is the correction exponent also found from Table 5.2.

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where:				
$k_l = \zeta I_g/I_{cr} + (1 - \zeta)$				EC2-1-1: F. (9.25)
where:				
$\therefore \zeta = 1 - 0.5\ (M_{cr}/M_k)^2$				EC2-1-1: Cl. 9.3.3 (1)
The cracking moment ignoring reinforcement:				
$M_{cr} = f_{ctm} bh^2/6$*				
where:				
$f_{ctm} = 0.3 f_{ck}^{2/3} = 0.3 \times 30^{2/3}$	=	2.9 MPa		EC2-1-1: Table 5.1
$\therefore M_{cr} = 2.9 \times 1000 \times 200^2/6/10^6$	=	19.3 kNm		
[Alternatively, use the more accurate formula:				H7. How to: Fig. H7.4
$M_{cr} = f_{ctm} I_g/(h - x_u) = 2.9 \times 667E + 06/(200 - 104.1)/10^6$				
	=	20.1 kNm]		
Characteristic load = 6.5 + 3.3	=	9.8 kN/m²		
The bending moment under the characteristic combination:				
$M_k = 9.8 \times 4.8^2/8$	=	28.2 kNm		
$\therefore M_k > M_{cr}$ i.e. the section is cracked				
$\therefore \zeta = 1 - 0.5\ (19.3/28.2)^2$	=	0.8		
$I_g/I_{cr} = 1/[2.7\ (\alpha_{e,ef}\ \rho)^{0.6}\ (d/h)^3]$				EC2-1-1: F. (9.26)
where:				
$\rho = A_{s,prov}/(bd) = 565/(1000 \times 169)$	=	0.33%		
$\therefore I_g/I_{cr} = 1/[2.7 \times (23.4 \times 0.33/100)^{0.6} \times (169/200)^3]$				
	=	2.83		
$\therefore k_l = 0.8 \times 2.83 + (1 - 0.8)$	=	2.4		
The linear elastic deflection (determined for the uncracked conditions due to quasi-permanent combination of actions):				
$\delta_{loads} = 5/384\ wL^4/(E_{c,eff} I_g)$				H7. How to: Fig. H7.4
where:				
w = quasi-permanent load n_{QP} (from before)	=	7.5 kN/m²		
$\therefore \delta_{loads} = 5/384 \times 7.5 \times 4800^4/(8545 \times 6.67E + 08)$	=	9.1 mm		
For a cracked section:				NA to EC2-1-1: Cl. NA.4.50
The shrinkage curvature can be ignored i.e. δ_{ecs}	=	0.0 mm		
$\therefore \delta = 2.4 \times (9.1 + 0)$	=	**21.8 mm**		
Allowable deflection is L/250 = 4800/250	=	**19.2 mm**		
$\therefore \delta >$ allowable deflection				
$\therefore$ Increase reinforcement to H12-150 B1 (754 mm²/m)				
Recalculate deflections with the increased reinforcement:				
$x_u = [200000 \times (200/2) + (23.4 - 1)(754 \times 169)]/[200000 + (23.4 - 1) \times 754]$				
	=	**105.4 mm**		
$\rho = A_{s,prov}/(bd) = 754/(1000 \times 169)$	=	0.45%		
$\therefore I_g/I_{cr} = 1/[2.7 \times (23.4 \times 0.45/100)^{0.6} \times (169/200)^3]$	=	2.38		EC2-1-1: F. (9.26)
$\therefore k_l = 0.8 \times 2.38 + (1 - 0.8)$	=	2.1		EC2-1-1: F. (9.25)
δ_{loads} (from before)	=	9.1 mm		
As before, δ_{ecs}	=	0.0 mm		NA to EC2-1-1: Cl. NA.4.50
$\therefore \delta = 2.1 \times (9.1 + 0)$	=	**18.7 mm**		EC2-1-1: F. (9.23)
$\therefore \delta <$ allowable deflection (19.2 mm)				
$\therefore$ Deflection OK				

* This simplified equation gives an approximation for the cracking moment and ignores the reinforcement contribution by assuming $(h - x_u) = 0.5$ h.

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Check that the condition for Table 5.2 of EC2-1-1 is met i.e. check that the compressive stress under the quasi-permanent (QP) combinations < $0.4f_{cm}(t_0)$: — EC2-1-1: Cl. 5.1.5 (3)

First find the compressive stress under QP combinations

ψ_2 for category B office areas = 0.3 — NA EC0-1: Table NA.2

The quasi-permanent load:

n_{QP}* = 6.5 + 0.3 x 3.3 = 7.5 kN/m^2 — EC0-1: Table A.1.6 & F. (8.31)

The quasi-permanent moment:

$M_{QP} = 7.5 \times 4.8^2/8$ = 21.6 kNm/m

Assuming a fully cracked transformed singly-reinforced section, the neutral axis from the top fibres:

$x = \{[(A_s \alpha_e)^2 + 2bA_s d\alpha_e]^{0.5} - (A_s \alpha_e)\}/b$ — H7. How to: Panel H7.2

Assume a value for α_e = 20**

$x = \{[(565 \times 20)^2 + (2 \times 1000 \times 565 \times 169 \times 20]^{0.5} - (565 \times 20)\}/1000$ = 51.5 mm

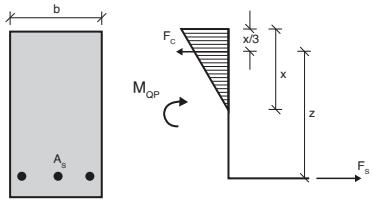


Figure H1.1b.2: SLS elastic stress block

Assuming an elastic stress block as shown in Fig. H1.1b.2:

The compressive force in the concrete, F_c:

$F_c = F_s = M_{QP}/z = M_{QP}/(d - x/3)$

$= 21.6/(169 - 51.5/3) \times 1000$ = 142.1 kN

$\therefore$ The concrete compressive stress at the extreme fibre:

$\sigma_c = 2F_c/(xb)$

$= 2 \times 142.1 \times 1000/(51.5 \times 1000)$ = **5.5 MPa**

The mean concrete compressive strength, $f_{cm}(t)$:

$f_{cm}(t) = \beta_{cc}(t) f_{cm}$ — EC2-1-1: F. (B.1)

$\beta_{cc}(t) = \exp[s_c(1 - \sqrt{(t_{ref}/t)})\sqrt{(28/t_{ref})}]$; $t \le t_{ref}$ — EC2-1-1: F. (B.2)

Assume Class CR (rapid hardening)

s_c (for f_{ck} < 35 MPa and Class CR) = 0.3 — EC2-1-1: Table B.2

Assume that the slab formwork is struck at 3 days

$\therefore t_0$ = 3 days

t_{ref} = 28 days — EC2-1-1: Cl. 5.1.3 (2)

$\therefore \beta_{cc}(t_0) = \exp[0.3(1-\sqrt{(28/3)})\sqrt{(28/28)}]$ at 3 days = 0.54

$\therefore f_{cm}(t_0) = 0.54 \times 38$ at 3 days = 20.5 MPa

$\therefore 0.4f_{cm}(t_0) = 0.4 \times 20.5$ at 3 days = **8.2 MPa**

5.5 < 8.2 $\therefore$ OK

$\therefore$ EC2-1-1 Table 5.2 can be used†

---

* Depending how the office is managed, it may be sensible to consider partition loading as a permanent action when calculating quasi-permanent actions for deflections. This is not the case in this WE.

** A modular ratio of 20 is used as an initial estimate to calculate the concrete compressive stress. This is later found to equal 23.4 > 20, which corresponds to a larger neutral axis depth 'x' and thus, a smaller concrete compressive stress. Hence, the estimation is conservative.

† In situations where Table 5.2 of EC2-1-1 cannot be used, Annex B.5 should be used instead.

Note:

In this case, there is no benefit in using the simplified explicit verification method for deflections over the deflection control span/depth ratio method (see WE H1.1) and both methods require H12-150. The simplified explicit verification method requires at least 715 mm²/m (for a 19.2 mm deflection) i.e. 3% less than the 740 mm²/m required by the span/depth ratio method. There may be cases where the simplified explicit verification method for deflections results in more economical designs. Additionally, the simplified explicit verification method for deflections has the benefit of explicitely calculating deflections, and these can be directly compared with values obtained from finite element models.

H1.1b.8 Summary of design

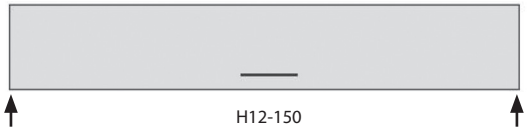


Figure H1.1b.3: Simply supported slab – summary

H1.1b.9 Detailing checks

Note:

It is presumed that both the designer and the detailer play an important role in ensuring accurate detailing and the interaction of both parties is crucial for the project success (refer to SMDSC). This section includes checks which are typically expected from the designer but responsibilities might vary from project to project.

a) Minimum areas

i) Minimum reinforcement rules for bending

$A_s > bh^2f_{ctm}/(5.7df_{yk})$ — NA EC2-1-1: Cl. NA.4.60

where:

f_{yk} = 500 MPa — EC2-1-1: Table 5.4

$f_{ctm} = 0.3f_{ck}^{2/3}$ (for $f_{ck} \le 50$ MPa) — EC2-1-1: Table 5.1

$\therefore f_{ctm} = 0.3 \times 30^{2/3}$ = 2.90 MPa

$A_s > 1000 \times 200^2 \times 2.9/(5.7 \times 169 \times 500)$ = **241 mm²**

∴ Minimum area of reinforcement OK

ii) Secondary (transverse) reinforcement

Provide minimum secondary reinforcement of $0.2A_{s,req,span}$ — EC2-1-1: Table 12.2

∴ Minimum secondary reinforcement = 0.2×539 = **108 mm²/m**

∴ Use H10-250* (314 mm²/m) B2

iii) Maximum spacing

Maximum spacing of bars for concrete in tension:

$s_{slab,max} = 3h \le 400$ mm

$= \min(3 \times 200, 400)$ = **400 mm** — EC2-1-1: Table 12.2

Longitudinal (flexural) reinforcement: 150 < 400 mm

Transverse (secondary) reinforcement: 250 < 400 mm

∴ Maximum bar spacing OK

* The theoretical maximum spacing for strength design can be considerably greater than 250 mm (see Section H1.1b.9 a) iii) "maximum spacing"). However, a smaller spacing is selected in this WE to make reinforcement safe for site operatives to walk on.

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iv) Crack control:

Slab is internal and hence, crack width limits for durability are not relevant

For appearance, limit crack widths to 0.4 mm: — EC2-1-1: Table 9.1

$w_{lim,cal}$ = 0.4 mm

Check that the crack widths do not exceed the crack width limit = 0.4 mm using the refined method for the control of cracking EC2-1-1 Cl. 9.2.3:

The difference in mean strains is:

$\varepsilon_{sm} - \varepsilon_{cm} = 1/E_s\,[\sigma_s - \{k_t\,(f_{ct,eff}/\rho_{p,eff})(1 + \alpha_e\rho_{p,eff})\}] \geq (1 - k_t)(\sigma_s/E_s)$ — EC2-1-1: F. (9.11)

where:

E_s (from Section H1.1b.7) = 200,000 MPa — EC2-1-1: Cl. 5.2.4 (3)

α_e = 15* — EC2-1-1: Cl. 9.1 (5)

Assuming a fully-cracked transformed singly-reinforced section:

The neutral axis from the top fibres:

$x = \{[(A_s\alpha_e)^2 + 2bA_sd\alpha_e]^{0.5} - (A_s\alpha_e)\}/b$ — H8. How to: Panel H8.3

$x = \{[(754 \times 15)^2 + (2 \times 1000 \times 754 \times 169 \times 15]^{0.5} - (754 \times 15)\}/1000$

= 51.5 mm

The quasi-permanent moment:

M_{QP} (from Section H1.1b.7) = 21.6 kNm/m

Assuming an elastic stress block (see Fig. H1.1b.2):

The tensile force and stress in the steel:

$F_c = F_s = M_{QP}/z = M_{QP}/(d - x/3)$

$= 21.6/(169 - 51.5/3) \times 1000$ = 142.1 kN

$\sigma_s = 142.1 \times 1000/754$ = 188 MPa

For the long-term and stabilised cracking stage,

k_t = 0.4 — EC2-1-1: Cl. 9.2.3 (3)

For a single layer of bars:

$h_{c,eff} = \min(a_y + 5\phi, 10\phi, 3.5a_y, h - x, h/2)$ — EC2-1-1: Fig. 9.3

where:

$a_y = c + \phi/2 = 25 + 12/2$ = 31 mm — EC2-1-1: Fig. 9.3a

$\therefore h_{c,eff} = \min(31 + 5 \times 12, 10 \times 12, 3.5 \times 31, 200 - 51.5, 200/2)$ — EC2-1-1: Fig. 9.3

$= \min(91, 120, 108.5, 148.5, 100)$ = 91 mm

$b_{eff} = (1000/150) \times \min[(10 \times 12), 150]$ = 800 mm — EC2-1-1: Fig. 9.3b

$A_{c,eff} = b_{eff}h_{c,eff} = 800 \times 91$ = 72,800 mm² — EC2-1-1: Cl. 9.2.3 (3)

$\therefore \rho_{p,eff} = A_s/A_{c,eff} = 754/72800$ = 1.0% — EC2-1-1: F. (9.12)

$f_{ct,eff} = f_{ctm}$ (from Section H1.1b.7) = 2.90 MPa

$\varepsilon_{sm} - \varepsilon_{cm} = 1/E_s\,[\sigma_s - \{k_t\,(f_{ct,eff}/\rho_{p,eff})(1 + \alpha_e\rho_{p,eff})\}] \geq (1 - k_t)(\sigma_s/E_s)$ — EC2-1-1: F. (9.11)

$\therefore \varepsilon_{sm} - \varepsilon_{cm} = 1/200000 \times [188 - \{0.4 \times (2.9/0.000104) \times (1 + 15 \times 0.000104)\}] \times 10^6$

= 296 με

But $\varepsilon_{sm} - \varepsilon_{cm} \geq (1 - 0.4) \times (188/200000) \times 10^6$ = 565 με

$\therefore \varepsilon_{sm} - \varepsilon_{cm}$ = 565 με

The calculated mean crack spacing is:

$s_{r,m,cal} = 1.5c + (k_fk_b/7.2)(\phi/\rho_{p,eff}) \leq (1.3/k_w)(h - x)$ — EC2-1-1: F. (9.15)

* This is an approximate value. Alternatively, use the modular ratio calculated for deflections (see H1.1b.7).

where:

$k_{fl} = (h - h_{c,eff})/h = (200 - 91)/200$ = 0.55 — EC2-1-1: F. (9.16)

For a slab < 300 mm thick, assume good bond — EC2-1-1: Cl. 11.4.2 (4b) & Fig. 11.4

$\therefore k_b$ = 0.9 — EC2-1-1: F. (9.18)

k_w = 1.7 — EC2-1-1: Cl. 9.2.3 (2)

$\therefore s_{r,m,cal} = (1.5 \times 25) + (0.55 \times 0.9/7.2) \times (12/0.0104)$ = 116 mm

But $s_{r,m,cal} \le (1.3/1.7) \times (200 - 51.5)$ = 114 mm

$\therefore s_{r,m,cal}$ = **114 mm**

The calculated surface crack width is:

$w_{k,cal} = k_w k_{1/r} s_{rm,cal} (\varepsilon_{sm} - \varepsilon_{cm})$ — EC2-1-1: F. (9.8)

where:

$k_{1/r} = (h - x)/(h - a_{y,i} - x)$ — EC2-1-1: F. (9.9)

$= (200 - 51.5)/(200 - 31 - 51.5)$ = 1.26

$\therefore w_{k,cal} = k_w k_{1/r} s_{rm,cal} (\varepsilon_{sm} - \varepsilon_{cm})$

$= 1.7 \times 1.26 \times 114 \times 565 \times 10^{-6}$ = **0.14 mm**

$\therefore w_{k,cal} < w_{lim}$

∴ Crack width OK

ii) Minimum area of longitudinal tension (flexural) reinforcement

For pure bending, the minimum area of reinforcement to avoid yielding* is:

$A_{s,min,w1} = 0.2\, k_h f_{ct,eff} Ac/f_{yk}$ — EC2-1-1: F. (9.2)

where:

$f_{ct,eff} = f_{ctm}$ (from Section H1.1b.7) = 2.90 MPa — EC2-1-1: Cl. 9.1 (3)

$k_h = 0.8 - 0.6$ (min{b, h} − 0.3); $k_h \le 0.8$, $k_h \ge 0.5$ — EC2-1-1: F. (9.5)

where:

b (in metres) = 1 m

h (in metres) = 0.2 m

$k_h = 0.8 - 0.6 \times [(\min(1.0, 0.2) - 0.3]$ = 0.86

But $k_h \le 0.8$, $k_h \ge 0.5$

$\therefore k_h$ = 0.80

$A_c = b_h = 1000 \times 200$ = 200,000 mm²

$\therefore A_{s,min,w1} = 0.2 \times 0.8 \times 2.9 \times 200000/500$ = **185 mm²/m**

∴ Minimum area of reinforcement OK

b) Curtailment

i) Reinforcement at edges (B2, T2)

Provide longitudinal and transverse reinforcement at unsupported edges to accommodate torsional moments near edges as follows:

U-bar length ("1" in EC2-1-1 Fig. 12.4) ≥ max(2h, l_{bd}) — EC2-1-1: Fig. 12.4

where:

l_{bd} is the anchorage length calculated from EC2-1-1 F. (11.3)**

$l_{bd} = k_{lb} k_{cp} \phi (\sigma_{sd}/435)^{n\sigma} (25/f_{ck})^{0.5} (\phi/20)^{1/3} (1.5\phi/c_d)^{0.5} \ge 10\phi$ — EC2-1-1: F. (11.3)

where:

$c_d = \min(0.5c_s, c_x, c_y, 3.75\phi)$

$c_s = 250 - 10$ = 240 mm — EC2-1-1: Fig. 11.3

c_x = (assumed) = 50 mm — EC2-1-1: Fig. 11.3

$c_y = 25 + 12$ = 37 mm — EC2-1-1: Fig. 11.3

$c_d = \min(0.5 \times 240, 50, 37, 3.75 \times 10)$ = 37 mm

* Crack width equations assume that reinforcement does not yield. This is ensured by providing $A_{s,min,w1}$. Typically, this check is less critical than the minimum reinforcement rules for bending.

** Alternatively, use Table 11.1 of EC2-1-1 (see WE H1.1).

mpa The Concrete Centre	Project details **H1.1b Simply supported one-way solid slab (detailed method)**	Calculated by **MM**	Job no. **H1**
		Checked by **CIEG**	Sheet no. **10/11**
		Client **TCC**	Date **Nov 2026**

	It is prudent to limit c_d to $3.0\phi = 3.0 \times 10$	=	30 mm	NA EC2-1-1: Cl. NA.4.53
	$\therefore c_d = \min(37, 30)$	=	30 mm	
	$h < 300$ mm $\therefore$ good bond			EC2-1-1: Fig. 11.4
	$\therefore k_{cp}$	=	1.0	EC2-1-1: Cl. 11.4.2 (3)
	$\sigma_{sd} = f_{yd} A_{s,min}/A_s = 435 \times 241/314$	=	333 MPa	EC2-1-1: Cl. 12.2 (6)
	k_{lb}	=	50	EC2-1-1: Cl. 11.4.2 (3)
	n_σ	=	1.0	NA EC2-1-1: Cl. NA.2.43
	Ratio limit of $(\phi/20) \geq 0.6$			EC2-1-1: Cl. 11.4.2 (3)
	$\therefore \phi/20 = 10/20 = 0.5$. Thus, use $\phi/20$	=	0.6	
	Ratio limit of $(25/f_{ck}) \geq 0.3$ $\therefore$ 25/30	=	0.83	
	$\therefore l_{bd} = 50 \times 1 \times 10 \times (333.1/435)^1 \times 0.83^{0.5} \times 0.6^{1/3} \times (1.5 \times 10/30)^{0.5}$			EC2-1-1: F. (11.3)
	$\geq 10 \times 10$	=	208 mm	
	$\therefore$ minimum U-bar length = max(2 x 200, 208)	=	**400 mm***	
	$\therefore$ Use H10-250 (314 mm²) U-bars at edges extending 400 mm from ends			
ii) Reinforcement at ends (B1, T1)				
	Main bottom reinforcement:			
	Curtail main bars 50 mm from or at face of support			
	Provide minimum longitudinal bottom reinforcement at end supports (B1) of:			
	$0.25A_{s,req,span} = 0.25 \times 539$	=	**135 mm²/m**	EC2-1-1: Table 12.2
			say, H10-300 (262 mm²/m)**	
	and extending by lap length, l_{sd} to main bars			
	$l_{sd} = k_{ls} l_{bd} \geq 15\phi$			EC2-1-1: Table 11.3 & Cl. 11.5.2 (2)
	where:			
	k_{ls}	=	1.0	NA EC2-1-1: Cl. NA.2.44
	ϕ = minimum diameter of lapped bars	=	10 mm	
	l_{bd} is the anchorage length calculated from EC2-1-1: F. (11.3)†			
	$l_{bd} = k_{lb} k_{cp} \phi (\sigma_{sd}/435)^{n\sigma} (25/f_{ck})^{0.5} (\phi/20)^{1/3} (1.5\phi/c_d)^{0.5} \geq 10\phi$			EC2-1-1: F. (11.3)
	where:			
	$c_d = \min(0.5c_s, c_x, c_y, 3.75\phi)$			
	$c_s = 150 - (2 \times 10)$	=	130 mm	EC2-1-1: Fig. 11.10 & NA EC2-1-1: Cl. NA.4.55
	c_x = (assumed)	=	50 mm	EC2-1-1: Fig. 11.3
	$c_y = c_{nom}$	=	25 mm	EC2-1-1: Fig. 11.3
	$c_d = \min(0.5 \times 130, 50, 253.75 \times 10)$	=	25 mm	
	It is prudent to limit c_d to $3.0\phi = 3.0 \times 10$	=	30 mm	NA EC2-1-1: Cl. NA.4.53
	$\therefore c_d = \min(25, 30)$	=	25 mm	
	$h < 300$ mm $\therefore$ good bond			EC2-1-1: Fig. 11.4
	$\therefore k_{cp}$	=	1.0	EC2-1-1: Cl. 11.4.2 (3)
	$\sigma_{sd} = f_{yd} A_{s,min}/A_s = 435 \times 241/754$	=	139 MPa	EC2-1-1: Cl. 12.2 (6)
	k_{lb}	=	50	EC2-1-1: Cl. 11.4.2 (3)
	n_σ	=	1.0	NA EC2-1-1: Cl. NA.2.43
	Ratio limit of $(\phi/20) \geq 0.6$			EC2-1-1: Cl. 11.4.2 (3)
	$\therefore \phi/20 = 10/20 = 0.5$. Thus, use $\phi/20$	=	0.6	
	Ratio limit of $(25/f_{ck}) \geq 0.3$			
	$\therefore$ 25/30	=	0.83	

* In this case, there is no benefit in calculating anchorage lengths using EC2-1-1 F. (11.3) because the minimum U-bar length is governed by twice the slab thickness.

** The 300 mm spacing is a multiple of the 150 mm main reinforcement spacing.

† Alternatively, use Table 11.1 of EC2-1-1 (see WE H1.1).

$\therefore l_{bd} = 50 \times 1 \times 10 \times (139/435)^{1} \times 0.83^{0.5} \times 0.6^{1/3} \times (1.5 \times 10/25)^{0.5}$ EC2-1-1: F. (11.3)

$\geq 10 \times 10 = 100$ mm

$\therefore l_{sd} = 1 \times 100$ but $\geq 15 \times 10 = 150 =$ **150 mm***

Top reinforcement:

Assuming clamping from the wall above, provide minimum top reinforcement at end supports (T1) of:

Provide minimum top reinforcement at end supports (T1) of:

$0.25A_{s,req,span} = 0.25 \times 539 =$ 135 mm²/m EC2-1-1: Table 12.2

but $> A_{s,min} =$ **241 mm²/m** EC2-1-1: Table 12.2

and extending 0.2 x length of span from the end support

$= 0.2 \times 4800$ mm $=$ **960 mm** EC2-1-1: Table 12.2

$\therefore$ Use H10 - 300 (262 mm²) B1 T1 as U-bars extending 960 mm at top and lapping 150 mm with main bottom steel

c) Detailing summary

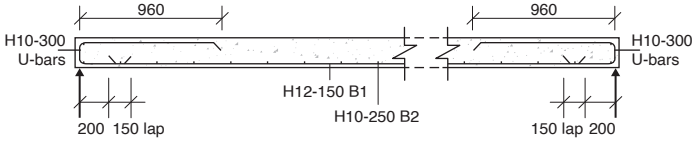


Figure H1.1b.4: Reinforcement detailing summary

List of references

[1] British Standards Institution. (2026) BS EN 1990-1:2023+A1:2026 Eurocode – Basis of structural and geotechnical design – New structures. London, BSI.

[2] British Standards Institution. (2023) BS EN 1992-1-1:2023 Eurocode 2 – Design of concrete structures. Part 1-1: General rules and rules for buildings, bridges and civil engineering structures. London, BSI.

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[6] MPA The Concrete Centre. (2026) How to design concrete structures using Eurocode 2 (second generation). H1 - How to: Solid slabs. London, TCC.

[7] DRAFT: MPA The Concrete Centre. (2026) How to design concrete structures using Eurocode 2 (second generation). H7 - How to: Deflection calculations. London, TCC.

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[10] MPA The Concrete Centre. (2026) Worked Examples to Eurocode 2 (second generation). WE H1.1– Simply supported one-way solid slab. London, TCC.

* In this case, there is no benefit in calculating anchorage lengths using EC2-1-1 F. (11.3) because the lap length is governed by fifteen times the bar size.

Revision number	Revision date	Notes
1.0	Nov 2026	First publication

At the time of publication, work on the second generation of Eurocodes and supporting documents is ongoing. Where possible, this worked example has been developed based on the standards that have been published to date, but in some instances, the relevant documents have not yet been finalised and so anticipated values are included and clearly noted as such. Amendments are planned for all of the Eurocodes ahead of the March 2028 date of adoption and users are advised to ensure they are referring to the latest version of a standard or publication.

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