	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.

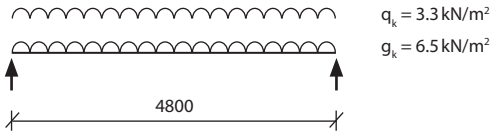


Figure H1.1.1: Simply supported one-way slab

General notes:

This calculation is intended to show a typical basic hand calculation. The worked example uses quick tabulated methods for deflection and crack control checks and anchorage and lap lengths calculations. For alternative and more detailed methods, refer to Worked Example H1.1b.

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:	BS 8500-1	BS 8500-1:2023 [1]
	EC0-1	BS EN 1990-1:2026 [2]
	EC1-1-1	BS EN 1991-1-1:2025 [3]
	EC2-1-1	BS EN 1992-1-1:2023 [4]
	EC2-1-2	BS EN 1992-1-2:2023 [5]
	NA EC0-1	NA to BS EN 1990-1:2023+A1:2026 [6]
	NA EC1-1-1	NA to BS EN 1991-1-1:2026 [7]
	NA EC2-1-1	NA to BS EN 1992-1-1:2026 [8]
	PD 6687	PD 6687:2026 [9]
Guidance documents:	G2. How to	G2 - How to: Actions (2026) [10]
	G3. How to	G3 - How to: Detailing (2026) [11]
	H1. How to	H1 - How to: Solid slabs (2026) [12]
	H8. How to	H8 - How to: Cracking calculations (2026) [13]
	SMDSC	Standard method of detailing structural concrete [14]
	WE H1.1b	Simply supported one-way solid slab (detailed method) [15]

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H1.1.1 Actions

Permanent:			kN/m²	
Self-weight	= 0.2 x 25	=	5.0	EC1-1-1: Table A.1
50 mm screed	= 0.05 x 20	=	1.0	EC1-1-1: Table A.1
Finishes, services		=	0.5	
	Total	g_k =	**6.5 kN/m²**	
Variable:			kN/m²	
Offices, sub-category B1.1		=	2.5	NA EC1-1-1: Table NA.1
Moveable partitions	= max[0.35, (0.4 x 2)]	=	0.8*	EC1-1-1: F. (6.1)
	Total	q_k =	**3.3 kN/m²**	

H1.1.2 Load combinations (and arrangement)

Assuming consequence class CC2b (normal upper class): NA EC0-1: Table NA.1

Consequence factor, k_F = 1.0 NA EC0-1: Table NA.4

For category B: office areas, ψ_0 = 0.7 NA EC0-1: Table NA.2

> *Note:*
>
> *For the structural resistance of the slab, verification case VC1 should be used with either F. (8.12) or with the more adverse of the upper and lower parts of F. (8.13). This WE applies the more economical F. (8.13). The design is also compared with that for loads calculated to F. (8.12), values for which are denoted by the symbol "‡".*

Ultimate load, n:

Using EC0 F. (8.13) upper part with VC1:

$n = \gamma_G g_k + \gamma_{Q,1} \psi_{0,1} q_{k,1}$ EC0-1: Table A.1.4

$n = 1.35 k_F g_k + 1.5 k_F \psi_{0,1} q_{k,1}$ NA EC0-1: Table NA.3

(Alternatively, use How to look-up table) G2. How to: Table G2.8

n = (1.35 x 1 x 6.5) + (1.5 x 1 x 0.7 x 3.3) = 12.2 kN/m²

Using EC0 F. (8.13) lower part with VC1:

$n = \xi \gamma_G g_k + \gamma_{Q,1} q_{k,1}$ EC0-1: Table A.1.4

$n = \xi 1.35 k_F g_k + 1.5 k_F q_{k,1}$ NA EC0-1: Table NA.3

(Alternatively, use How to look-up table) G2. How to: Table G2.8

n = (0.925 x 1.35 x 1 x 6.5) + (1.5 x 1 x 3.3) = 13.1 kN/m² NA EC0-1: Cl. NA.2.2.4

∴ F. (8.13) lower part governs

∴ n = **13.1 kN/m²**

(‡ If F. (8.12) is used with VC1:

n = (1.35 x 1 x 6.5) + (1.5 x 1 x 3.3) = 13.7 kN/m²

i.e. The load calculated to F. (8.12) is 5% more than that calculated to F. (8.13).)

---

* Although in this WE actions due to partitions are calculated based on the self-weight of the partitions (in kN/m), it is common practice in the UK to assume 1 kN/m² for partition loads. Note that NA EC1-1-1 Cl. NA.2.10 refers to PD 6688-1-1 for partition loads in offices.

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H1.1.3 Cover

Nominal cover, c_{nom}:

$c_{nom} = c_{min} + \Delta c_{dev}$ — EC2-1-1: F. (6.1)

where:

$c_{min} = \max[(c_{min,dur} + \Sigma\Delta c), c_{min,b}, 10\text{ mm}]$ — EC2-1-1: F. (6.2)

where:

$c_{min,dur}$ = minimum cover due to enviornmental conditions

Assuming XC1 and using C30/37 concrete, — BS 8500-1: Table A.1

$c_{min,dur}$ = 15 mm — BS 8500-1: Table A.3

$\Sigma\Delta c$ = the sum of applicable reductions and additions — EC2-1-1: Cl. 6.5.2.1

= 0

Assuming maximum aggregate size = 20 mm < 32 mm,

$c_{min,b}$ = minimum cover due to bond = bar diameter — EC2-1-1: Table 6.5

Assuming 12 mm main bars,

$c_{min,dur}$ = 12 mm

$\therefore c_{min} = \max[(15 + 0), 12, 10]$ = 15 mm

Δc_{dev} = allowance in design for deviation

Assuming no measurement of cover,

Δc_{dev} = 10 mm — EC2-1-1: Table 6.7

$\therefore c_{nom} = 15 + 10$ = **25 mm**

Fire:

Check adequecy of section for 1 hour fire resistance (i.e. REI 60)

Thickness, $h_{s,min}$ = 80 mm < 200 mm proposed ∴ OK — EC2-1-2: Table 6.9

Axis distance, a_{min} = 20 mm < 25 + 12/2 = 31 i.e. not critical — EC2-1-2: Table 6.9

∴ OK — & Fig. 6.1

∴ choose c_{nom} = 25 mm

H1.1.4 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**

(‡ For the load combination using F. (8.12) with VC1:

M_{Ed} = 39.5 kNm/m

V_{Ed} = 30.6 kN/m)

* 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 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 used - see Section H1.1.6. — EC2-1-1: Fig. 8.6

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H1.1.5 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*) — EC2-1-1: Cl. 5.1.6 (1)

$= 1.00$

$f_{ck,ref}$ $= 40$ 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

Flexure in span:

$K = M_{Ed}/(bd^2 f_{cd})$ — H1. How to: Fig. H1.3

$= (37.6 \times 10^6)/(1000 \times 169^2 \times 20)$ $= 0.066$

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

$\therefore K < K_{lim}$ no compression reinforcement is required. Use K $= 0.066$

$\therefore z = d/2\,[1 + \sqrt{(1 - 2K)}]$ but $\leq 0.95d$ — H1. How to: Fig. H1.3

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

but $\leq (0.95 \times 169)$ $= 161$ 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:

$A_{s,req} = M_{Ed}/f_{yd} z$ — H1. How to: Fig. H1.3

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**

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

Try H12-200 B1 (565 mm²/m)

(‡ For the load combination using F. (8.12) with VC1:
$A_{s,req} = 566$ mm²/m

H12-200 B1 (565 mm²/m) is sufficient (just!)
i.e. no change in reinforcement is required.)

---

* EC2-1-1 accounts for the impact of the age of the concrete on its strength using the factor k_{tc}. When considering load combinations that occur more than 3 months after casting, a k_{tc} value of 1.0 may be used. In typical cases, structures may be subject to construction loading within the first 3 months after casting and so, these should be assessed using the lower k_{tc} value of 0.85. The in-service load combinations, however, may not be applied for at least 3 months and so, the higher k_{tc} value may be adopted for their assessment.

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H1.1.6 Shear

V_{Ed} (from Section H1.1.4)	=	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:			
$\tau_{Rdc,min} = 11/\gamma_V \sqrt{[(f_{ck}/f_{yd}) \times (d_{dg}/d)]}$			EC2-1-1: F. (8.20)
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
$\therefore \tau_{Rdc,min} = 11/1.4 \times \sqrt{[(30/435) \times (36/169)]}$	=	**0.95 MPa**	
(Alternatively, use How to look-up tables for d = 169 mm and C30/37)			H1. How to: Table H1.8
$\therefore \tau_{Ed} < \tau_{Rdc,min}$			
∴ Detailed verification of the shear resistance may be omitted			EC2-1-1: Cl. 8.2.1 (1a)
∴ Section has sufficient shear capacity			

(‡ For the load combination using F. (8.12) with VC1:
V_{Ed} = 30.6 kN/m
τ_{Ed} = 0.20 MPa < 0.95 MPa
i.e. the section has sufficient shear capacity.)

H1.1.7 Deflection

Using the simplified deflection control by span/depth ratio:			EC2-1-1: Cl. 9.3.2 & H1. How to: Fig. H1.5
The following conditions are satisfied:			EC2-1-1: Cl. 9.3.2 (1) & Table 9.3
i) The load is uniformly distributed			
ii) A deflection limit of l/250 is appropriate			
iii) ψ_2 for category B office areas	=	0.3	NA EC0-1: Table NA.2
∴ The three conditions for using EC2-1-1 Table 9.3 are met			
$\omega_r = (A_{s,req} f_{yd})/(bdf_{cd})$			
$= (539 \times 435)/(1000 \times 169 \times 20)$	=	0.07	EC2-1-1: Table 9.3
Calculate the characteristic live load (LL)/characteristic total load (TL):			EC2-1-1: Table 9.3
LL (from Section H1.1)	=	3.3 kN/m^2	
TL (from Section H1.1) = 3.3 + 6.5	=	9.8 kN/m^2	
∴ LL/TL	=	34%	
For a simply supported one-way spanning solid slab with LL/TL = 34% and mechanical reinforcement ratio = 0.07, limiting span-to-effective-depth ratio l/d:			H1. How to:
For ω_r = 0.05**, interpolating between LL / TL = 30% and 45%	=	22.7	Table H1.11
For ω_r = 0.1, interpolating between LL / TL = 30% and 45%	=	17.5	EC2-1-1: Table 9.3
Interpolating between the above for LL / TL = 34%	=	20.7	

* As noted in Section H1.1.4, 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.
** Table 9.3 of EC2-1-1 is limited to ω_r = 0.1. For low reinforcement ratios (i.e. ω_r < 0.1), refer to the tabulated values in the H1. How to guide.

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Check span-to-effective-depth ratio

$l/d = 4800/169$ = 28.4

$20.7 < 28.4$ ∴ Provide additional reinforcement — EC2-1-1: Cl. 9.3.2 (1)

Increasing factor $A_{s,prov}/A_{s,req} = 28.4/20.7$ = 1.37

$1.37 < 1.5$ (limit) — NA EC2-1-1: Cl. NA.4.49

∴ $A_{s,prov} = 1.37 \times 539$ = 740 mm²/m

∴ Increase reinforcement to H12-150 B1 (754 mm²/m)

(‡ For the load combination using F. (8.12) with VC1:

Limiting span-to-effective-depth ratio by interpolation = 20.3

Increasing factor $A_{s,prov}/A_{s,req} = 28.4/20.3$ = 1.40

$1.4 < 1.5$ (limit) ∴ $A_{s,prov} = 1.4 \times 566$ = 792 mm²/m

∴ Increase reinforcement further to H12-125 B1 i.e. 905 mm²/m. This shows that there is a slight benefit in using the more economical combination F. (8.12) with VC1.)

H1.1.8 Summary of design

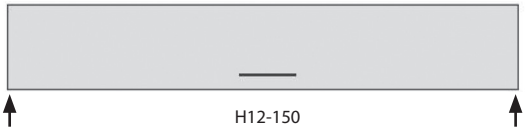


Figure H1.1.2: Simply supported slab - summary

(‡ For the load combination using F. (8.12) with VC1, increase main reinforcement to H12-125.)

H1.1.9 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

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

where:

$f_{ctm} = 0.3f_{ck}^{2/3}$ (for $f_{ck} \le 50$ MPa)

∴ $f_{ctm} = 0.3 \times 30^{2/3}$ = 2.9 MPa — EC2-1-1: Table 5.1

f_{yk} (given) = 500 MPa — EC2-1-1: Table 5.4

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

(Alternatively, use How to look-up table for minimum percentage reinforcement) — H1. How to: Table H1.14

∴ Minimum area of reinforcement OK

(‡ Also applicable for the load combination using F. (8.12) with VC1.)

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ii) Secondary (transverse) reinforcement

Provide minimum secondary reinforcement of $0.2A_{s,req,span}$ — EC2-1-1: Table 12.2 (3)

$\therefore$ Minimum secondary reinforcement = 0.2 x 539 = **108 mm²/m**

$\therefore$ Use H10-250* (314 mm²/m) B2

(‡ For the load combination using F. (8.12) with VC1:
Minimum secondary reinforcement = 113 mm²/m i.e. H10-250 is sufficient.)

iii) Maximum spacing

Maximum spacing of bars for concrete in tension:

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

= min[(3 x 200), 400] = **400 mm** — EC2-1-1: Table 12.2

Longitudinal (flexural) reinforcement: 150 < 400 mm

Transverse (secondary) reinforcement: 250 < 400 mm

$\therefore$ Maximum bar spacing OK

(‡ Also applicable for the load combination using F. (8.12) with VC1.)

iv) Crack control:

Note:
For a more detailed calculation to calculate crack widths, refer to WE H1.1b.

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

For appearance, limit crack widths to 0.4 mm:

i.e. $w_{lim,cal}$ = 0.4 mm — EC2-1-1: Table 9.1

Use the simplified crack control tables in PD 6687 for $w_{lim,cal}$ = 0.3 mm (< 0.4 mm i.e. conservative**)

First find the quasi-permenant stress, σ_s:

$\sigma_s = f_{yd} (A_{s,req}/A_{s,prov})$ (SLS load/ULS load) $(1/\delta_M)$ — H8. How to

ψ_2 for category B office areas (from Section H1.1.6) = 0.3 — NA EC0-1: Table NA.2

The quasi-permanent load:

SLS load† = n_{QP} = 6.5 + (0.3 x 3.3) = 7.5 kN/m² — EC0-1: Table A.1.6 & F. (8.31)

σ_s = 435 x (539/754) x (7.5/13.1) (1/1) = 178 MPa

Using PD 6687 Table 4 for c ≤ 30 mm — NA EC2-1-1: Cl. NA.3.12

For h = 200 mm: maximum ϕ = 25 mm at maximum spacing s = 250 mm — PD 6687: Table 4

$\therefore$ H12-150 is sufficient to limit crack widths to 0.4 mm

$\therefore$ No further checks are required

$\therefore$ Cracking OK

(‡ H12-125 is sufficient to limit crack widths to 0.4 mm for the load combination using F. (8.12) with VC1.)

---

* The theoretical maximum spacing for strength design can be considerably greater than 250 mm (see also Section H1.1.9 a) iii) "maximum spacing"). However, a smaller spacing is selected in this WE to make reinforcement safe for site operatives to walk on.

** The simplified crack control tables in PD 6687 are used to limit crack widths to 0.3 mm. The crack width limit for this WE is 0.4 mm and hence, these tables are conservative.

† Depending on the office layout and how the office is managed, it may be sensible to consider the partition loading as a permanent action when calculating the quasi-permanent load combination. For the purpose of this WE, this is not deemed to be the case.

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v) Minimum area of longitudinal tension (flexural) reinforcement

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

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

where:

$f_{ct,eff} = f_{ctm}$ (from Section H1.1.9 a) i)) = 2.90 MPa — EC2-1-1: Cl. 9.1 (3)

$k_h = 0.8 - 0.6$ [min(b, h) – 0.3]; $k_h \leq 0.8$, $k_h \geq 0.5$ — 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 $k_h \leq 0.8$, $k_h \geq 0.5$

$\therefore k_h$ = 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

(‡ Also applicable for the load combination using F. (8.12) with VC1.)

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 Table 11.1

> *Note:*
> *For a more detailed calculation using F. (11.3), refer to WE H1.1b.*

$\sigma_{sd} = f_{yd} A_{s,min}/A_s$ = 435 x 241/314 = 333 MPa — EC2-1-1: Cl. 12.2 (6)

∴ Table 11.1 values for a stress of 435 MPa are conservative

For $\phi < 12$ mm, $f_{ck} = 30$ MPa, and good bond conditions:

l_{bd}/ϕ = 38 — EC2-1-1: Table 11.1

$\therefore l_{bd} = 38 \times 10$ = 380 mm**

∴ minimum U-bar length = max[(2 x 200), 380] = 400 mm

∴ Use H10-250† (314 mm²) U-bars at edges extending 400 mm from ends

(‡ Also applicable for the load combination using F. (8.12) with VC1.)

ii) Reinforcement at ends (B1, T1)

Curtailment main bottom reinforcement:

Curtail main bars 50 mm from or at face of support

Provide minimum longitudinal bottom reinforcement at end supports (B1) of:

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

say, H10-300‡ (262 mm²/m)

---

* 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 in Section H1.1.9 a) i).

** There is no benefit in refining the anchorage length calculation to account for a stress lower than 435 MPa because the minimum U-bar length is governed by 2h, which is greater than l_{bd} for $\sigma_{sd} = 435$ MPa.

† The selected 250 mm U-bar spacing matches the slab secondary reinforcement spacing.

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

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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:

ϕ = minimum diameter of lapped bars = 10 mm

k_{ls} = 1.0 — NA EC2-1-1: Cl. NA.2.44

l_{bd} is the anchorage length calculated from EC2-1-1 Table 11.1

> *Note:*
>
> *For a more detailed calculation using F. (11.3), refer to WE H1.1b.*

Using Table 11.1 of EC2-1-1

$\sigma_{sd} = f_{yd} A_{s,min}/A_s = 435 \times 241/754$ = 139 MPa — EC2-1-1: Cl. 12.2 (6)

For $\phi < 12$ mm, $f_{ck} = 30$ MPa, and good bond conditions:

l_{bd}/ϕ for a stress of 435 MPa = 38 — EC2-1-1: Table 11.1

$\therefore l_{bd} = 38 \times 10$ for a stress of 435 MPa = 380 mm

n_σ = 1.0 — NA EC2-1-1: Cl. NA.2.43

Factor to account for a lower stress*

$= (139/435)^1$ = 0.32

$\therefore l_{bd} = 0.32 \times 380 = 121$ but $\geq 10\phi = (100)$ = 121 mm

$\therefore l_{sd} = (1 \times 121) \geq (15 \times 10) = (150)$ = **150 mm**

Top reinforcement:

Assuming clamping from the wall above, 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 x 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**

(‡ Also applicable for the load combination using F. (8.12) with VC1 but provide H10-250 U-bars to match the spacing of the main reinforcement.)

c) Detailing summary

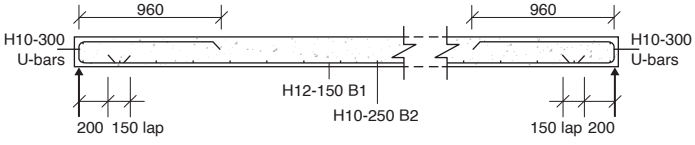


Figure H1.1.3: Reinforcement detailing summary

(‡ For the load combination using F. (8.12) with VC1, increase main reinforcement to H12-125 and U-bars to H10-250†.)

* The tabulated anchorage lengths in EC2-1-1 Table 11.1 are derived from F. (11.3), where the anchorage length l_{bd} is a function of $(\sigma_{sd}/435)^{n\sigma}$. In the table, σ_{sd} is taken as 435 MPa i.e. $(\sigma_{sd}/435)^{n\sigma} = 1$. In this example, for a lower stress, the tabulated anchorage length is thus multiplied by $(\sigma_{sd}/435)^1$.

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

† The 250 mm U-bar spacing is a multiple of the main bar spacing.

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

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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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