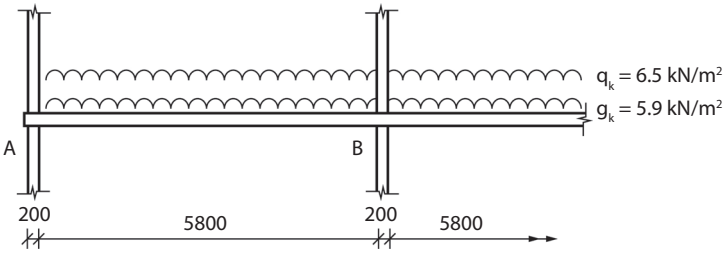
	Project details				
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Example

A 200 mm thick continuous 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 200 mm wide load-bearing block walls at 6 m centres. Assume that f_{ck} = 30 MPa, f_{yk} = 500 MPa and the design life is 50 years. A fire resistance of 1 hour is required.



Note:
The structure consists of four spans and the area of each slab bay is > 30 m².

Figure H1.2.1: Continuous solid slab

General notes:

This calculation is intended to show in detail the provisions of designing a continuous solid slab to Eurocode 2 Part 1-1. It also demonstrates how to curtail longitudinal reinforcement using the shift rule to Clause 12.3.2 of the code. The slab loading and durability and fire requirements are similar to that of WE H1.1, to which this worked example refers.

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]
	EC1-1-1	BS EN 1991-1-1:2025 [2]
	EC2-1-1	BS EN 1992-1-1:2023 [3]
	NA EC0-1	NA to BS EN 1990-1:2023+A1:2026 [4]
	NA EC1-1-1	NA to BS EN 1991-1-1:2026 [5]
	NA EC2-1-1	NA to BS EN 1992-1-1:2026 [6]
	PD 6687	PD 6687:2026 [7]

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Guidance documents:

G3. How to	G3 - How to: Detailing (2026) [8]
H1. How to	H1 - How to: Solid slabs (2026) [9]
H8. How to	H8 - How to: Cracking calculations (2026) [10]
SMDSC	Standard method of detailing structural concrete (2021) [11]
WE H1.1	Simply supported one-way solid slab (2026) [12]

H1.2.1 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**	

H1.2.2 Load combinations (and arrangement)

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

Using the most adverse of the upper and lower part of Formula (8.13) of EC0: EC0-1: Table A.1.4 and NA EC0-1: Table NA.3

Ultimate load, n = **13.1 kN/m²**

H1.2.3 Cover

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

Cover for durability:

c_{nom} = **25 mm**

This is also adequate for a 1 hour fire resistance

H1.2.4 Analysis

Slab span = 6.0 m — EC2-1-1: Cl.7.2.3 (5)

$q_k < 1.25\, g_k$ ∴ OK

$q_k < 5$ kN/m² ∴ OK

Slab has four spans and each span > 30 m²

∴ Analysis tables can be used for bending moments and shear forces — H1. How to: Table H1.3

Design moment:

End span:

M_{Ed} = 0.086 x 13.1 x 6² = **40.5 kNm/m** — H1. How to: Table H1.3

1st internal support:

M_{Ed} = 0.086 x 13.1 x 6² = **40.5 kNm/m** — H1. How to: Table H1.3

Internal span & support:

M_{Ed} = 0.063 x 13.1 x 6² = **29.7 kNm/m** — H1. How to: Table H1.3

* 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 refers to partitions allowance for offices which is presented in PD 6688-1-1. — NA EC1-1-1: Cl. NA.2.10

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Shear force:

End support:

V_{Ed} = 0.4 x 13.1 x 6 = **31.4 kN/m** H1. How to: Table H1.3

1st internal support:

V_{Ed} = 0.6 x 13.1 x 6 = **47.1 kN/m** H1. How to: Table H1.3

H1.2.5 Flexural design

For C30/37:

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

where:

η_{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

Effective depth:

$d = h - c_{nom} - \phi/2$ $= 200 - 25 - 12/2$ = 169 mm

a) End span and 1st internal support

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

$= 40.5 \times 10^6/(1000 \times 169^2 \times 20)$ = 0.07

The analysis tables used assume 20%** redistribution $\therefore \delta_M$ = 0.8

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

$\therefore K_{lim} = 1.10 \times 0.8 - 0.32 \times 0.8^2 - 0.45$ = 0.229

(Alternatively, use How to look-up tables for $\delta_M = 0.8$) H1. How to: Fig. H1.6

$K < K_{lim}$

$\therefore$ use K = 0.07

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

$= (169/2) \times [1 + \sqrt{(1 - 2 \times 0.07)}]$ = 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 = 0.07 < 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

$\therefore A_{s,req}$ $= (40.5 \times 10^6)/(435 \times 161)$ = **580 mm²/m** H1. How to: Fig. H1.3

$\therefore \rho$ $= A_{s,req}/(bd) = 580/(1000 \times 169)$ = 0.34%

Try H10-125 B1 and T1 (628 mm²/m)

---

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

** The tables assume 20% moment redistribution to the hogging moments at the supports but not at the spans. Consequently, assuming 20% redistribution to both supports and spans is conservative for the spans.

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b) Internal spans and supports

K	$= M_{Ed}/(bd^2f_{cd})$			H1. How to: Fig. H1.3
	$= 29.7 \times 10^6/(1000 \times 169^2 \times 20)$	=	0.05	
K_{lim}	= (from Section H1.2.5 a))	=	0.23	
$K < K_{lim}$				
∴ use K		=	0.05	
∴ z	$= d/2\ [1+\sqrt{(1-2K)}]$ but $\leq 0.95d$			H1. How to: Fig. H1.3
	$= (169/2) \times [1+\sqrt{(1-2 \times 0.05)}]$ but ≤ 161	=	161 mm*	

Tension reinforcement:

$A_{s,req}$	$= M_{Ed}/f_{yd}z$ $= (29.7 \times 10^6)/(435 \times 161)$	=	**425 mm²/m**	H1. How to: Fig. H1.3
∴ ρ	$= A_{s,req}/(bd)$ $= 425/(1000 \times 169)$	=	0.25%	
			Try H12-250 B1 and T1 (452 mm²/m)	87.5

H1.2.6 Shear

Design shear force, V_{Ed}:

At distance d from the face of the end support,				EC2-1-1: Fig. 8.6
V_{Ed}	$= 31.4 - [(0.169 + 0.1) \times 13.1]$	=	29.2 kN/m	
τ_{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)
At distance d from the face of the first internal support,				EC2-1-1: Fig. 8.6
V_{Ed}	$= 47.1 - [(0.169 + 0.1) \times 13.1]$	=	44.9 kN/m	
τ_{Ed}	$= V_{Ed}/(b_w \times 0.9d) = (44.9 \times 1000)/(1000 \times 0.9 \times 169)$	=	**0.29 MPa****	EC2-1-1: F. (8.18)

Shear resistance

The shear resistance neglecting shear reinforcement is: H1. How to: Fig. H1.4

$\tau_{Rdc,min} = 11/\gamma_V \sqrt{[(f_{ck}/f_{yd}) \times (d_{dg}/d)]}$ EC2-1-1: F. (8.20)

where:

Assuming D_{lower}	=	20 mm	NA EC2-1-1: 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

∴ $\tau_{Ed} < \tau_{Rdc,min}$

∴ Detailed verification of the shear resistance may be omitted EC2-1-1: Cl. 8.2.1 (1a)

∴ Section has sufficient shear capacity

H1.2.7 Deflection: end span

a) End span

Using the simplified deflection control by span/depth ratio: EC2-1-1: Cl. 9.3.2

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

* Since the moment at internal spans and supports is less than the end span moment, z = 161 mm by inspection.

** If the design shear force at the face of the support is used, the design shear stress at the end support and first internal support is 0.21 MPa and 0.31 MPa respectively, hence < 0.95 MPa i.e. the section's shear capacity is sufficient.

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∴ The three conditions for using EC2-1-1 Table 9.3 are met				
ω_r	$= A_{s,req} f_{yd} / b d f_{cd}$			
	$= 580 \times 435 / (1000 \times 169 \times 20)$	=	0.07	EC2-1-1: Table 9.3
Calculate characteristic live load (LL)/characteristic total load (TL)				EC2-1-1: Table 9.3
LL	(from Section H1.2.1)	=	3.3 kN/m²	
TL	(from Section H1.2.1) $= 3.3 + 6.5$	=	9.8 kN/m²	
∴ LL/TL		=	34%	
For an end span of a continuous one-way spanning solid slab with LL/TL = 34% and mechanical reinforcement ratio = 0.07, the limiting span-to-effective-depth ratio l/d:				
For $\omega_r = 0.05$*, interpolating between LL/TL = 30% & 45%		=	29.0	H1.How to: Table H1.11
For $\omega_r = 0.1$, interpolating between LL/TL = 30% & 45%		=	22.7	EC2-1-1: Table 9.3
Interpolating between the above for LL/TL = 34%		=	25.9	
Check span-to-effective-depth ratio				
l/d = 6000/169		=	35.5	
25.9 < 35.5		∴ Provide additional reinforcement		EC2-1-1: Cl. 9.3.2 (1)
Increasing factor $A_{s,prov}/A_{s,req}$	$= 35.5/25.9$	=	1.37	
1.37 < 1.5 (limit)				NA to EC2-1-1: Cl. NA.4.49
∴ $A_{s,prov} = 1.37 \times 580$		=	**795 mm²/m**	
		∴ Increase reinforcement to H12-125 B1 (905 mm²/m)		
b) Internal span				
As before, the three conditions for using EC2-1-1: Table 9.3 are met				
ω_r	$= A_{s,req} f_{yd} / b d f_{cd}$			EC2-1-1: Table 9.3
	$= 425 \times 435 / (1000 \times 169 \times 20)$	=	0.05	
For an internal span of a continuous one-way spanning solid slab with LL/TL = 34% and mechanical reinforcement ratio = 0.05, the limiting span-to-effective-depth ratio l/d:				
Interpolating between LL/TL = 30% and 45%		=	29.0	H1.How to: Table H1.11
29 < 35.5		∴ Provide additional reinforcement		EC2-1-1: Cl. 9.3.2 (1)
Increasing factor $A_{s,prov}/A_{s,req}$	$= 35.5/29$	=	1.23	
1.23 < 1.5 (limit)				NA to EC2-1-1: Cl. NA.4.49
∴ $A_{s,prov} = 1.23 \times 425$		=	**520 mm²/m**	
		∴ Increase reinforcement to H12-200 B1 (565 mm²/m)		

* Table 9.3 of EC2-1-1 provides span-to-depth (l/d) limiting ratios for three mechanical tension reinforcement ratios, ω_r of 0.1, 0.2 and 0.3. The permitted ratios may be interpolated for intermediate ω_r values. However, for low reinforcement ratios ($\omega_r < 0.1$), which is common in slabs, very conservative l/d ratios may result if the limiting values in the table were to be extrapolated. For improved design efficiency for reinforced concrete elements with low reinforcement ratios, refer to the tabulated l/d values in the additional column for $\omega_r = 0.05$ in the H1. How to guide.

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H1.2.8 Summary of design

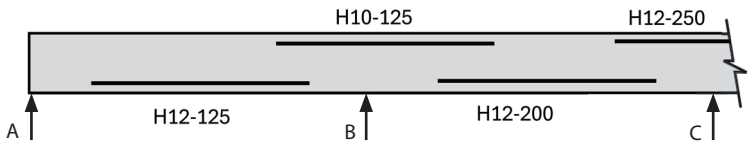


Figure H1.2.2: Continuous slab: summary

H1.2.9 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: Cl. NA.4.60

where:

$f_{ctm} = 0.3f_{ck}^{2/3}$ (for $t > t_{ref} = 28$) — EC2-1-1: Table 5.1

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

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

$\therefore$ Minimum area of reinforcement OK

ii) Secondary (transverse) reinforcement

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

$\therefore$ Minimum secondary reinforcement = 0.2×580 = **116 mm²/m**

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

Check edges:

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

U-bar length $\geq \max(2h, l_{bd})$ — EC2-1-1: Fig. 12.4

where:

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

Minimum reinforcement is used. Thus:

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

$\therefore$ Table 11.1 values for a stress of 435 MPa are conservative

* The theoretical maximum spacing for strength design can be considerably greater than 250 mm (see Section H1.2.9 a) iii) "maximum spacing"). However, a smaller maximum reinforcement pitch is selected in this WE to ensure that site operatives can safely walk over reinforcement.

h < 300 mm ∴ good bond — EC2-1-1: Fig. 11.4

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

l_{bd}/ϕ = 38 — EC2-1-1: Table 11.1

∴ l_{bd} = 38 x 10 for a stress of 435 MPa = 380 mm*

(Alternatively, use How to look-up tables for "straight" bars, "good" bond and 10 mm diameter) — G3. How to: Table G3.2

∴ U-bar length ≥ max(2 x 200, 380) = **400 mm**

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

iii) Maximum spacing

Maximum spacing of bars for concrete in tension:

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

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

Longitudinal (flexural) reinforcement: 175 < 400 mm

Transverse (secondary) reinforcement: 250 < 400 mm

∴ Maximum bar spacing OK

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:

$w_{lim,cal}$ = 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**)

Use Table 4 for a slab with $f_{ck} \ge 25$ MPa and $c \le 30$ mm

The quasi-permenant stress:

$\sigma_s = f_{yd} (A_{s,req}/A_{s,prov})$ (SLS load/ULS load) $(1/\delta_M)$ — H8. How to: Fig H8.3

ψ_2 for category B office areas (from Section H1.2.7) = 0.3 — NA EC0-1: Table NA.2

The quasi-permanent load: — EC0-1: Table A.1.6 & F. (8.31)

SLS load† = n_{QP} = 6.5 + (0.3 x 3.3) = 7.5 kN/m²

	$A_{s,prov}$ (mm²)	$A_{s,req}$ (mm²)	δ_M	σ_s (MPa)
End span	905	580	1.0	160
1st interior support	628	580	0.8	287
Interior span	565	425	1.0	187
Interior suport	452	425	0.8	293

largest stress: **293 MPa**

For h = 200 mm and a steel stress of 300 MPa > 293 MPa

Maximum ϕ = 20 mm at maximum spacing, s = 250 mm — PD 6687: Table 4

∴ H12 at a maximum spacing of 250 mm is sufficient to limit crack widths to 0.4 mm

∴ No further checks are required

∴ Cracking OK

* There is no benefit in refining the anchorage length calculation to account for a stress lower than 435 MPa (i.e. by multiplying l_{bd} by a factor of $(333/435)^{1}$) because the minimum U-bar length is governed by 2h, which is greater than l_{bd} for σ_{sd} = 435 MPa.

** 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 H1.2.9 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

vi) End supports: effects of partial fixity

Main bottom reinforcement:

Curtail main bars 50 mm from or at face of support

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

$0.25A_{s,req,span} = 0.25 \times 580$ = **145 mm²/m** — EC2-1-1: Table 12.2

and extending by lap length, l_{sd} to main bars (see H1.2.9 d))

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:

$0.25\ A_{s,req,span} = 0.25 \times 580$ = 145 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 6000 mm = **1200 mm** — EC2-1-1: Table 12.2

∴ Use H10-250 (349 mm²) B1 T1 as U-bars

extending 1200 mm at top

b) Curtailment

i) End span, bottom reinforcement

Method A

The bottom reinforcement provided at supports, A_s = 349 mm²/m

Curtailment of reinforcement may take place a_l from where the moment of resistance of the section with the remaining steel would be adequate to resist the applied bending moment.

The equivalent bending moment resistance is:

$M_{Ed} = A_s f_{yd} z$

$= 349 \times 435 \times 161/10^6$ = 24.4 kNm/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 H1.2.9 a) i).

For a slab without shear reinforcement:

$a_l = d$ = 169 mm — EC2-1-1: F. (12.6)

For a simply-supported span supporting a UDL of n (see Figure H1.2.3 b):

The reaction at support A:

$R_A = 0.4nl$

At distance X from the end support, the moment is:

$M_{Ed,X} = R_A X - nX^2/2$

$\therefore M_{Ed,X} = 0.4nlX - nX^2/2$

$\therefore$ When $M_{Ed,X}$ = 24.4 kNm/m:

$24.4 = 0.4\ nlX - nX^2/2$

$24.4 = 0.4 \times 13.1 \times 6 \times X - 13.1 \times 0.5\ X^2$

$0 = 6.54\ X^2 - 31.38\ X + 24.4$

$\therefore X = [31.38 +/- \sqrt{\{-31.38^2 - (4 \times 6.54 \times 24.4)\}}]/[2 \times 6.54]$

= 0.97 m

OR 3.83 m

Shift rule: — EC2-1-1: Cl. 12.3.2 (1) & Fig. 12.1

At end support, curtailment is required at $X - a_l = 970 - 169$ = **805 mm**

Say, 800 mm from face of support A

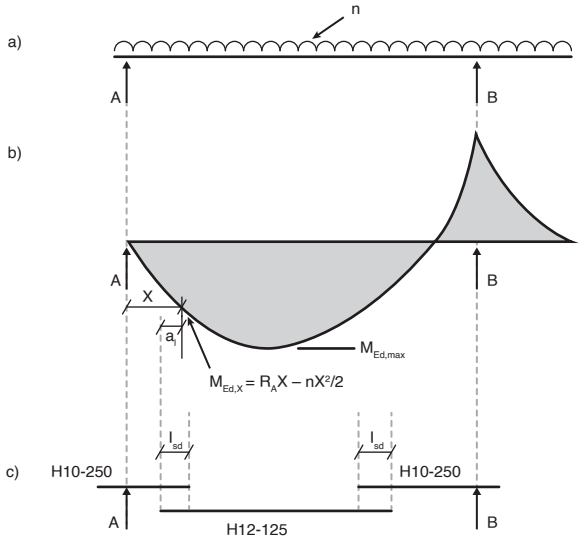


Figure H1.2.3: Curtailment of bottom reinforcement (Method A):
a) Load arrangement,
b) Bending moment $M_{Ed,X}$,
c) Curtailment of bottom reinforcement

Similarly, in end span at first internal support, X from support B
= 6.0 – 3.83 = 2.17 m
Shift rule:
At first internal support, curtailment is required at 2170 – 169 = **2005 m** EC2-1-1: Cl. 12.3.2 (1) & Fig. 12.1

Say, 2 m from face of support B

Method B (alternative method)
A common alternative method to curtail reinforcement is to stagger bottom rebar such that 100% is provided at midspan and 50% is provided at the supports (see Figure H1.2.4).

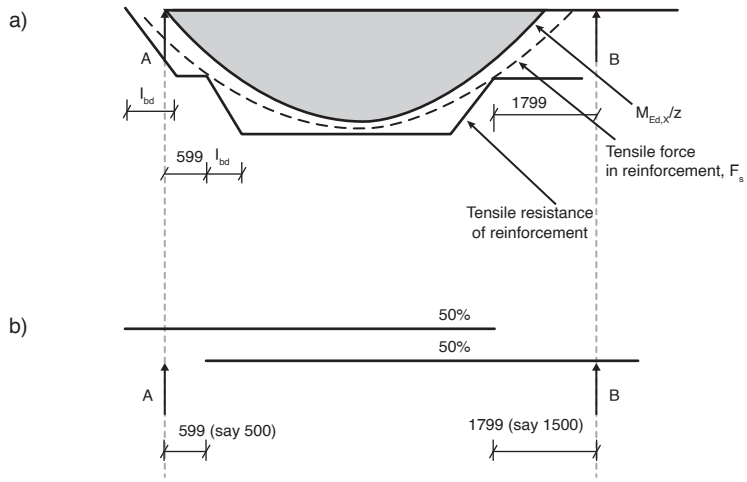


Figure H1.2.4: Curtailment of bottom reinforcement (alternative method): a) Tensile force in the bottom reinforcement, b) curtailment of bottom reinforcement

In theory, 50% curtailment of reinforcement may take place a_l from where the moment of resistance of the section with the remaining 50% would be adequate to resist the applied bending moment. In practice, it is usual to determine the curtailment distance as being a_l from where $M_{Ed} = M_{Ed,max}/2$.
For a simply-supported span supporting a UDL of n:
The reaction at support A:
$M_{Ed,max} = 0.086nl^2$
$R_A = 0.4nl$
At distance X from the end support, the moment is:
$M_{Ed,X} = R_AX - nX^2/2$
$\therefore M_{Ed,X} = 0.4nlX - nX^2/2$
$\therefore$ When $M_{Ed,X} = M_{Ed,max}/2$
$0.086nl^2/2 = 0.4nlX - nX^2/2$
Assuming $X = xl$:
$0.043nl^2 = 0.4nlxl - n(xl)^2/2$

Crossing out nl^2 from both sides

$\therefore 0.043 = 0.4x - x^2/2$

$\therefore 0 = x^2/2 - 0.4x + 0.043$

$\therefore x = [0.4 +/- \sqrt{\{-0.4^2 - (4 \times 0.5 \times 0.04)\}}]/[2 \times 0.5]$

$= 0.13l$

OR $0.67l$

$\therefore$ At end support, 50% moment occurs at 0.13 x span

$= 0.13 \times 6000 = 768$ mm

Shift rule: EC2-1-1: Cl. 12.3.2 (1) & Fig. 12.1

As before, $a_l = 169$ mm

At end support, curtailment is required at $768 - 169 =$ **599 mm**

Say, 500 mm from face of support A

Similarly, in the end span at first internal support, X from support A

$= 0.67 \times 6000 = 4032$ mm

Shift rule:

At first internal support, curtailment is required at $4032 + 169$ from support A $= 4201$ mm EC2-1-1: Cl. 12.3.2 (1) & Fig. 12.1

or $6000 - 4201$ from support B = **1799 mm**

Say, 1.5 m from face of support B

ii) First internal support, top reinforcement

Presuming 50% curtailment of reinforcement is required, this may take place a_l from where the moment of resistance of the section with the remaining 50% would be adequate. However, it is usual to determine the curtailment distance as being a_l from where $M_{Ed} = M_{Ed,max}/2$.

Thus, for the first internal support supporting a UDL of n:

The maximum moment at support B:

$M_{Ed,maxT} = 0.086nl^2$

The reaction at support B:

$R_B = 0.6nl$

At distance Y from the end support, the moment is:

$M_{Ed,Y} = M_{Ed,maxT} - R_B Y + nY^2/2$

$\therefore M_{Ed,Y} = 0.086nl^2 - 0.6nlY + nY^2/2$

$\therefore$ When $M_{Ed,Y} = M_{Ed,maxT}/2$

$0.086nl^2/2 = 0.086nl^2 - 0.6nlY + nY^2/2$

Assuming $Y = yl$ (i.e. Y is in terms of the span)

$0.043nl^2 = 0.086nl^2 - 0.6nyl^2 + nl^2y^2/2$

$0.043 = 0.086 - 0.6y + y^2/3$

$0 = 0.5y^2 - 0.6y + 0.043$

$\therefore y = [-0.6 +/- \sqrt{\{0.6^2 - (4 \times 0.5 \times 0.043)\}}]/[2 \times 0.5] = 0.08$

OR 1.12

i.e. $Y = yl = 6y = 0.46$ m

OR 6.74 m

Shift rule:

In end span at support B, curtailment is required at $460 + 169$ EC2-1-1: Cl. 12.3.2 (1) & Fig. 12.1

$= 628$ mm

Check: 0.5 x support width + $a_l + l_{bd}$ EC2-1-1: Fig. 12.1

$= 0.5 \times 200 + 169 + 380 = 649$ mm

Use distance from support centreline = max(628, 649) = **649 mm**

50% of reinforcement may be curtailed at, say 600 mm from either face of support B

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100% curtailment may take place a_l from where there is no hogging moment			EC2-1-1: Fig. 12.1
$M_{Ed,Y} = 0$			
$\therefore\ 0 = 0.086nl^2 - 0.6nlY + nY^2/2$			
Assuming Y = yl (i.e. Y is in terms of the span)			
$0 = 0.086\,nl^2 - 0.6\,nyl^2 + nl^2y^2/2$			
$0 = 0.086 - 0.6\,y + y^2/3$			
$0 = 0.5y^2 - 0.6\,y + 0.086$			
$\therefore\ y = [-0.6 +/- \sqrt{\{0.6^2 - (4 \times 0.5 \times 0.086)\}}]/[2 \times 0.5]$	=	0.17	
	OR	1.03	
i.e. Y = yl = 6 y	=	1.00 m	
	OR	6.20 m	
Shift rule:			EC2-1-1: Cl. 12.3.2 (1)
In end span at support B, curtailment is required at 1000 + 169			& Fig. 12.1
	=	**1169 mm**	
100% of reinforcement may be curtailed at, say 1100 mm from either face of support B			
iii) Support B bottom steel at support			
At the support, 25% of span steel is required			
$0.25A_{s,req,span} = 0.25 \times 580$	=	**145 mm²/m**	EC2-1-1: Table 12.2
For convenience use H10-250 (349 mm²)			
c) Anchorage at end support			
The steel at supports should be anchored to resist a force:			
$F_E = V_{Ed}\, a_l/z$			
$= 29.2 \times 169/161$	=	30.7 kN	
The design stress in the bar is thus:			
$\sigma_{sd} = F_E/A_{s,prov}$			
$= 30.7 \times 1000/349$	=	88 MPa	
Anchorage length, l_{bd} is calculated from Table 11.1 of EC2-1-1*:			
h < 300 mm ∴ good bond			EC2-1-1: Fig. 11.4
For $\phi \le 12$ mm, good bond, and $f_{ck} = 30$ MPa:			
l_{bd}/ϕ for a stress of 435 MPa	=	38	EC2-1-1: Table 11.1
Bar diameter of end U-bars	=	10 mm	
$\therefore\ l_{bd} = 38 \times 10$ for a stress of 435 MPa	=	380 mm	
n_σ	=	1	NA EC2-1-1: Cl. NA.2.43
Factor to account for a lower stress = $(88/435)^1$	=	0.20*	
$c_d = \min(0.5\,c_s, c_x, c_y, 3.75\phi)$	=	25 mm	
$\therefore\ c_d > 1.5\,\phi$ i.e. Table 11.1 is conservative			
Factor to account for larger $c_d = (15/25)^{0.5}$	=	0.77 *	
$\therefore\ l_{bd} = 0.2 \times 0.77 \times 380 = 60$ but $\ge 10\,\phi = 100$	=	**100 mm**	
(Alternatively, use How to look-up tables for "straight" bars, "good" bond and 10 mm diameter)			G3. How to: Table G3.2
By inspection, using U-bars is OK			

* The anchorage lengths in Table 11.1 assume good bond conditions, $\sigma_{sd} = 435$ MPa and $c_d = 1.5\,\phi$. The tabulated anchorage lengths are conservative for cases with lower stressed bars ($\sigma_{sd} < 435$ MPa) and/or larger concrete covers ($c_d > 1.5\,\phi$). Adjustments may be made to F. (11.3) by multiplying the tabulated values by $(\sigma_{sd}/435)$ and $(1.5\,\phi/c_d)^{0.5}$. Note that the NA EC2-1-1 uses $n_\sigma = 1$ in F. (11.3).

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d) Laps

i) Lap H10-250 U-bars with H12-125 straight bars at the bottom end span:

$l_{sd} = k_{ls} l_{bd} \geq 15\phi$ — EC2-1-1: Table 11.3 & Cl. 11.5.2 (2)

where:

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

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

where:

bottom bars ∴ good bond condition (and h < 300 mm) — EC2-1-1: Fig. 11.4

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

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

Smaller bar diameter = 10 mm

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

σ_{sd} = the design stress at ULS at the position from where the lap is measured

V_{Ed} at end support = 31.4 kN

Assuming lap starts 800 mm from face of support (curtailment Method A):

Distance to support centreline = 900 mm

$M_{Ed} = 31.4 \times 0.9 - (13.1 \times 0.9^2/2)$ = 15.2 kNm

$\sigma_{sd} = M_{Ed}/(A_s z) = 15.2 \times 10^6/(349 \times 161)$ = 271 MPa

Factor to account for a lower stress = $(271/435)^1$ = 0.62*

As before (see Section H1.2.9 c)), $c_d > 1.5\phi$ = 25 mm

Factor to account for larger $c_d = (15/25)^{0.5}$ = 0.77 *

∴ $l_{bd} = 0.62 \times 0.77 \times 380 = 183$ but $\geq 10\phi = 100$ = 183 mm

(Alternatively, use How to look-up tables for "straight" bars, "good" bond and 10 mm diameter) — G3. How to: Table G3.2

∴ $l_{sd} = 1 \times 183 \geq 15\phi$ = **183 mm**

Provide say, 200 mm lap between U-bars and straight bars at support A

ii) Lap H10-250 bars with H12-125 bars and H10-250 bars with H10-125 bars at first internal support:

$l_{sd} = k_{ls} l_{bd} \geq 15\phi$ — EC2-1-1: Table 11.3 & Cl. 11.5.2 (2)

where:

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

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

ϕ is the smaller bar diamater = 10 mm

Factor to account for larger c_d (as before from Section H1.2.9d) i)) = 0.77

∴ $l_{bd} = 0.77 \times 38 \times 10$ but $\geq 10\phi = 100$ = 294 mm

(Alternatively, use How to look-up tables for "straight" bars, "good" bond and 10 mm diameter) — G3. How to: Table G3.2

∴ $l_{sd} = 1 \times 294 \geq 15\phi$ = **294 mm**

Provide say, 300 mm laps at bottom internal support

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* As noted in the previous section, the anchorage lengths in EC2-1-1 Table 11.1 are conservative for cases where $\sigma_{sd} < 435$ MPa and where $c_d > 1.5\phi$ and adjustments may be made to F. (11.3) by multiplying the tabulated values by $(\sigma_{sd}/435)$ for lower stresses and $(1.5\phi/c_d)^{0.5}$ to account for enhanced concrete confinement.

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iii) Lap H10-250 U-bars with H10-250 straight bars at the bottom edge:

$l_{sd} = k_{ls}\, l_{bd} \geq 15\phi$ — EC2-1-1: Table 11.3 & Cl. 11.5.2 (2)

where:

k_{ls}	=	1.0	NA EC2-1-1: Cl. NA.2.44
As before (from Section H1.2.9 a) iii)):			
σ_{sd} (for minimum reinforcement)	=	333 MPa	
l_{bd} = 38 x 10 for a stress of 435 MPa	=	380 mm	EC2-1-1: Table 11.1
Factor to account for a lower stress = $(333/435)^{1}$	=	0.77	
$c_d > 1.5\phi = 25 + 10 \leq 3.0\phi = 3.0 \times 10$	=	30 mm	NA EC2-1-1: NA.4.53
Factor to account for lower $c_d = (15/30)^{0.5}$	=	0.71	
$\therefore\ l_{bd} = 0.77 \times 0.71 \times 380$ but $\geq 10\phi = 100$	=	206 mm	
(Alternatively, use How to look-up tables for "straight" bars, "good" bond and 10 mm diameter)			G3. How to: Table G3.2

$\therefore\ l_{sd} = 1 \times 206 \geq 15\phi$ = **206 mm**

Provide say, 210 mm lap between U-bars and straight bars at edges

Note:

Adopting lap lengths equal to the anchorage lengths tabulated in Table 11.1 of EC2-1-1 of 38ϕ = 380 mm throughout would have been conservative and would have removed the need to calculate the lap length for each case because:
a) the slab is thin (200 mm) and thus, all laps are in good bond condition
b) small bars (10 mm) are used and thus, $c_d > 1.5\phi$ in all cases
c) a stress of 435 MPa is conservative.

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H1.2.10 Summary of reinforcement details

* For convenience, H12-200 is changed to H10-125 (for the spacing to match adjacent spans).
Note: Bottom reinforcement detailing is to Method A in Section H1.2.9. bi).

Figure H1.2.5: Continuous solid slab: reinforcement details

Figure H1.2.6: Section A-A – reinforcement detail at edge

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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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