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Example

A 450 mm deep and 300 mm wide rectangular beam is required to support office loads of g_k = 30.2 kN/m and q_k = 11.5 kN/m over two 6 m spans. The beams are supported on 300 mm wide supports. Assume that f_{ck} = 30 MPa, f_{yk} = 500 MPa, a 50-year design life and a requirement for a 2 hour resistance to fire in an internal environment.

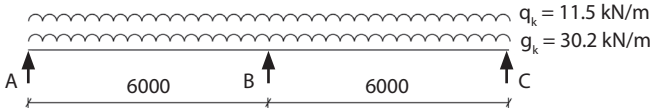


Figure H3.1.1: Continuous rectangular beam

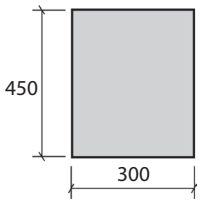


Figure H3.1.2: Section through beam

General notes:

This calculation is intended to show a typical hand calculation for a continuous simply-supported beam using coefficients to determine moments and shears. Simplified curtailment detailing rules are used.

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 1990-1:2023+A1:2026 [2]
	EC2-1-1	BS EN 1992-1-1:2023 [3]
	EC2-1-2	BS EN 1992-1-2:2023 [4]
	NA EC0-1	NA to BS EN 1990-1:2023+A1:2026 [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:

G2. How to	G2 - How to: Actions (2026) [8]
G3. How to	G3 - How to: Detailing (2026) [9]
H3. How to	H3 - How to: Beams (2026) [10]
H8. How to	H8 - How to: Cracking calculations (2026) [11]
SMDSC	Standard method of detailing structural concrete [12]
WE H1.2	WE H1.2: Continuous one-way slab (2026) [13]

H3.1.1 Actions

Permanent: g_k = **30.2 kN/m**
Variable: q_k = **11.5 kN/m**

H3.1.2 Load combinations and arrangement

Load combination:

Assuming consequence class CC2a (normal lower 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

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

$n = (1.35 \times 1 \times 30.2) + (1.5 \times 1 \times 0.7 \times 11.5)$ = 52.8 kN/m

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

$n = \xi \gamma_G g_k + \gamma_{Q,1} q_{k,1}$ — EC0: 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

where:

ξ = 0.925 — NA EC0-1: Cl. NA.2.2.4

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

$n = (0.925 \times 1.35 \times 1 \times 30.2) + (1.5 \times 1 \times 11.5)$ = 55.0 kN/m

$\therefore$ F. (8.13) lower part governs*

$\therefore$ n = **55.0 kN/m**

Arrangement:

Choose to use option b) "all spans and alternate spans" load case for analysis — PD 6687: Cl. 4.3
i.e. use coefficients — H3. How to: Table H3.3

* This could also be determined by checking that $g_k < 4.5\, q_k$
i.e. $30.2 < 4.5 \times 11.5 = 51.75$

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H3.1.3 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 mm

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 25 mm main bars and 8 mm links,

$c_{min,b}$ = 25 mm to main bars

= 25 – 8 = 17 mm to links, say = 20 mm

$\therefore c_{min}$ = max(15 + 0, 20, 10) = 20 mm

Δc_{dev} = allowance in design for deviation

Assuming no measurement of cover,

Δc_{dev} = 10 mm — EC2-1-1: Table 6.7

$\therefore c_{nom}$ = 20 + 10 to links = **30 mm**

Fire:

Check adequecy of section for 2 hour fire resistance (i.e. REI 120)

For b_{min} = 300 mm, average axis distance, a = 35 mm — EC2-1-2: Table 6.7

35 < (30 + 8 + 25/2) = 50.5 mm

i.e. not critical ∴ OK — EC2-1-2: Fig. 6.1

∴ choose c_{nom} = 30 mm to links

H3.1.4 Analysis

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

Using table coefficients: — H3. How to: Table H3.3

Design moments:

Spans:

$M_{Ed} = [(1.25 \times 30.2 \times 0.090) + (1.5 \times 11.5 \times 0.100)] \times 6^2$ = **184.4 kNm**

Support:

$M_{Ed} = 55 \times 0.106 \times 6^2$ = **209.7 kNm**

Shear force:

At outer supports, V = 0.45 x 55 x 6 = **148.4 kN**

At central support, V = 0.63 x 55 x 6 = **207.8 kN**

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H3.1.5 Flexural design

For C30/37:

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

where:

$\eta_{cc} = (f_{ck,ref}/f_{ck})^{1/3} \le 1.0$ — 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)

$\gamma_C = 1.5$ — EC2-1-1: Table 4.3

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

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

(Alternatively, use How to look-up tables for C30/37 and loading time ≥ 3 months) — H3. How to: Table H3.4

a) Flexure in span:

Assuming link diameter = 8 mm

Assuming bar diameter = 25 mm

Effective depth:

$d = h - c - \phi_{link} - \phi/2 = 450 - 30 - 8 - 25/2$ = **399.5 mm**

$K = M_{Ed}/(bd^2 f_{cd})$ — H3. How to: Fig. H3.3

$= 184.4 \times 10^6/(300 \times 399.5^2 \times 20) = 0.193$

Analysis tables assume redistribution at midspan = 3%

Conservatively, use redistribution = 0%

$\therefore \delta_M = 1 + 0 = 1.00$

$K_{lim} = 1.10\delta_M - 0.32 \times \delta_M^2 - 0.45$ — H3. How to: Fig. H3.3

$= 1.10 \times 1 - 0.32 \times 1^2 - 0.45 = 0.334$

(Alternatively, use How to look-up tables for $\delta_M = 1.0$) — H3. How to: Table H3.5

$\therefore K < K_{lim}$ $\therefore$ use K = 0.193

$\therefore$ No compression reinforcement is required

$\therefore z = d/2\,[1 + \sqrt{(1 - 2K)}]$ but $\le 0.95d$ — H3. How to: Fig. H3.3

$= 399.5/2 \times [1 + \sqrt{(1 - 2 \times 0.193)}]$ but $\le (0.95 \times 399.5) = 356$ mm

(Alternatively, use How to look-up tables to determine z/d ** for K = 0.193) — H3. How to: Table H3.6

Tension reinforcement:

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

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

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

$= (184.4 \times 10^6)/(435 \times 356) =$ **1190 mm²**

Try 2H25 + 1H20 B (1296 mm²)

$\therefore \rho = A_s/bd = 1296/(300 \times 399.5) = 1.08\%$

---

* 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.

** Interpolating between K = 0.18 and K = 0.20 in the How to look-up tables.

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Check that the bars fit:

Centre-to-centre bar spacing = [300 – (2 x 30) – (2 x 8) - 25)]/2 = 100 mm

∴ OK

b) Flexure at support:

Assuming top bar diameter = 25 mm

Effective depth to the tension reinforcement:

d = 450 – 30 – 8 – 12.5 = **399.5 mm**

Assume H25 bottom steel, bar diameter = 25 mm

$K = M_{Ed}/(bd^2f_{cd})$ — H3. How to: Fig. H3.3

$= (209.7 \times 10^6)/(300 \times 399.5^2 \times 20)$ = 0.219

Analysis tables assume redistribution at midspan = –15%*

$\therefore \delta_M = 1 + (-0.15)$ = 0.85

$K_{lim} = 1.10\delta_M - 0.32 \times \delta_M^2 - 0.45$ — H3. How to: Fig. H3.3

$= 1.10 \times 0.85 - 0.32 \times 0.85^2 - 0.45$ = 0.258

(Alternatively, use How to look-up tables for $\delta_M = 1.0$) — H3. How to: Table H3.5

$\therefore K < K_{lim}$ $\therefore$ Use K = 0.219

∴ No compression reinforcement is required

$\therefore z = d/2\ [1+\sqrt{(1-2K)}]$ but $\leq 0.95d$ — H3. How to: Fig. H3.3

$= 399.5/2 \times [1+\sqrt{(1-2 \times 0.219)}]$ but $\leq (0.95 \times 399.5)$ = 349 mm

(Alternatively, use How to look-up tables to determine z/d for K = 0.22) — H3. How to: Table H3.6

Tension reinforcement:

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

$A_s = (209.7 \times 10^6)/(435 \times 349)$ = **1367 mm²**

Try 3 no. H25 T (1473 mm²)

$\therefore \rho = A_s/bd = 1473/(300 \times 399.5)$ = 1.23%

H3.1.6 Shear

a) Central support (critical):

Shear at central support (from Section H3.1.4) = 207.8 kN

Effective depth, d (from Section H3.1.5b) = 399.5 mm

V_{Ed} may be calculated at z cotθ away from the support** — EC2-1-1: Cl. 8.2.3 (2)

Assume cotθ = 2.5

$V_{Ed} = V – (z \cot\theta)n$; where z = 0.9d

= 207.8 – [(0.9 x 399.5 x 2.5/1000) x 55] = 158.3 kN/m

$\tau_{Ed} = V_{Ed}/(b_w \times 0.9d) = (158.3 \times 1000)/(300 \times 0.9 \times 399.5)$ = 1.47 MPa — EC2-1-1: F. (8.18)

* This WE uses moment coefficient tables, which are based on 15% redistribution at supports. This degree of redistribution should be reflected in the K_{lim}. i.e. a K_{lim} value corresponding to $\delta_M = 0.85$ should be used.

** Where applied actions are uniformly distributed, shear may be checked at z cotθ from the face of the support. — EC2-1-1: Cl. 8.2.3 (2)

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Calculate the struts angle:

$\theta = \max [21.8, 0.5 \sin^{-1} \{\tau_{Ed}/(0.25 f_{cd})\}]$

$\theta = \max [21.8, 0.5 \times \sin^{-1} \{1.47/(0.25 \times 20)\}]$ — H3. How to: Fig. H3.8

$\theta = \max [21.8, 17.1]$ = 21.8°

$\therefore$ cotθ assumption is correct

Area of shear reinforcement:

$A_{sw}/s = \tau_{Ed}\, b_w/f_{ywd} \cot\theta$ — H3. How to: Fig. H3.8

$= (1.47 \times 300)/[435 \times \cot(21.8)]$ = **0.41 mm²/mm**

Try H8-225 (A_{sw}/s = 0.45 mm²/mm)

$\therefore \rho_w = A_{sw}/(b_w s) = 0.45/300$ = 0.150% — EC2-1-1: F. (8.43)

Check minimum shear reinforcement:

$\therefore \rho_{w,min} = A_{sw,min}/(s\, b_w \sin\alpha) \geq 0.08 (\sqrt{f_{ck}})/f_{yk}$ — EC2-1-1: F. (12.4)

where:

$\alpha = 90°$ i.e. sin (90) = 1.0

$0.08 (\sqrt{f_{ck}})/f_{yk} = (0.08 \times \sqrt{30}/500)$ = **0.088%**

(Alternatively, use How to look-up tables for f_{ck} = 30 MPa) — H3. How to: Table H3.10

0.0015 > 0.00088

$\therefore$ minimum shear reinforcement OK

Check maximum longitudinal spacing of stirrups:

$s_{l,max} = 0.75d (1 + \cot \alpha) = 0.75\, d (1 + \cot(90))$ — EC2-1-1: Table 12.1

$= 0.75\, d = 0.75 \times 399.5$ = **300 mm**

$\therefore$ max longitudinal spacing of stirrups OK

Check maximum transverse spacing of shear legs:

$s_{tr,max} = \min (0.75d, 600) = \min (300, 600)$ = **300 mm** — EC2-1-1: Table 12.1

c.f. 300 – (30 x 2) – 8 = 232 mm

$\therefore$ max transverse spacing of shear legs OK

$\therefore$ Use H8-225 (A_{sw}/s = 0.45 mm²/mm)

b) End supports:

Shear at end supports (from Section H3.1.4) = 148.4 kN

Effective depth, d (from Section H3.1.5b) = 399.5 mm

V_{Ed} may be calculated at z cotθ away from the support — EC2-1-1: Cl. 8.2.3 (2)

Assume cotθ = 2.5

$V_{Ed} = V - (z \cot\theta) n$; where z = 0.9d

V_{Ed} = 148.4 – [(0.9 x 399.5 x 2.5/1000) x 55] = 99.0 kN/m

$\tau_{Ed} = V_{Ed}/(b_w \times 0.9d) = (99.0 \times 1000)/(300 \times 0.9 \times 399.5)$ = 0.92 MPa — EC2-1-1: F. (8.18)

Calculate the struts angle:

$\theta = \max [21.8, 0.5 \sin^{-1} \{\tau_{Ed}/(0.25 f_{cd})\}]$

$\theta = \max [21.8, 0.5 \times \sin^{-1} \{0.92/(0.25 \times 20)\}]$ — H3. How to: Fig. H3.8

$\theta = \max [21.8, 10.6]$ = 21.8°

$\therefore$ cotθ assumption is correct

Area of shear reinforcement:

$A_{sw}/s = \tau_{Ed}\, b_w/f_{ywd} \cot\theta$ — H3. How to: Fig. H3.8

$= (0.92 \times 300)/[435 \times \cot(21.8)]$ = **0.25 mm²/mm**

Use H8-225 (A_{sw}/s = 0.45 mm²/mm)*

$\therefore \rho_w = A_{sw}/(b_{ws}) = 0.45/300$ = 0.15% — EC2-1-1: F. (8.43)

$\therefore$ Minimum area and maximum spacing of stirrups OK

* H10-300 (i.e. A_{sw}/s = 0.34 mm²/mm), which corresponds to the maximum longitudinal spacing, is sufficient. However, changing spacing of links would appear to be of limited benefit.

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H3.1.7 Deflection

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

$\therefore$ The three conditions for using EC2-1-1:2023 Table 9.3 are met

$\omega_r = A_{s,req} f_{yd} / b d f_{cd}$ — EC2-1-1: Table 9.3

$= (1190 \times 435)/(300 \times 399.5 \times 20)$ = 0.22

Calculate characteristic live load (LL)/characteristic total load (TL) — EC2-1-1: Table 9.3

LL (from Section H3.1.1) = 11.5 kN/m^2

TL (from Section H3.1.1) = 11.5 + 30.2 = 41.7 kN/m^2

$\therefore$ LL/TL = 11.5/41.7 = 28%

For an end span of a continuous beam with LL/TL = 28% and mechanical reinforcement ratio = 0.22, limiting span-to-effective-depth ratio l/d:

For $\omega_r = 0.2$, extrapolating between LL/TL = 30% and 45% = 16.5 — EC2-1-1: Table 9.3

For $\omega_r = 0.3$, extrapolating between LL/TL = 30% and 45% = 15.7 — EC2-1-1: Table 9.3

Interpolating between the above for LL/TL = 28% = 16.4

Modification factor = $(500/f_{yk})(A_{s,prov}/A_{s,req})$ — NA EC2-1-1: Cl. NA.4.49

$= (500/500) \times (1296/1190)$ = 1.1

Modified limiting l/d ratio = 1.1 x 16.4 = **17.8**

Check span-to-effective-depth ratio

l/d = 6000/399.5 = **15.0**

15 < 17.8 (limit) $\therefore$ OK

$\therefore$ Deflections OK

H3.1.8 Summary of design

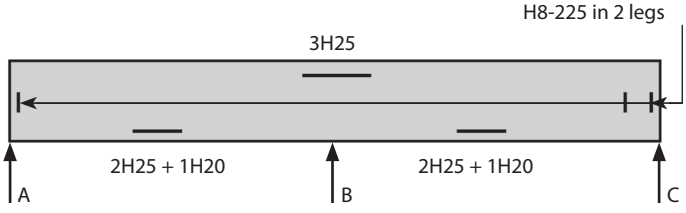


Figure H3.1.3: Continuous rectangular beam: summary

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H3.1.9 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 reinforcement area 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 $f_{ck} \leq 50$ MPa) — EC2-1-1: Table 5.1

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

Using the smaller d = min (399.5, 399.5) = 399.5 mm

$A_s > (300 \times 450^2 \times 2.9)/(5.7 \times 399.5 \times 500)$ = **155 mm²**

(Alternatively, use How to look-up tables for f_{ck} = 30 MPa and beam depth of 450 mm)* — H3. How to: Table. H3.9

∴ Minimum area of reinforcement OK

b) Crack control

i) Crack width check

The beam is intenal and thus, crack width limits for durability are not relevant — EC2-1-1: Cl. 9.2.1 (2) & (3)

For an exposure class of XC1:

Crack width limit, $w_{lim,cal}$ = 0.4 mm** — EC2-1-1: Table 9.1

And use quasi-permanent combination of actions — EC2-1-1: Table 9.1

Note:

Crack widths are only checked at the bottom surface of the beam because the slab and finishes overlie the beam and thus, appearance is not an issue at the top surface.

At midspan:

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 H3.1.7) = 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} = 30.2 + (0.3 x 11.5) = 33.7 kN/m²

σ_s = 435 x (1190/1296) x (33.7/55) (1/1) = 244 MPa

* The minimum percentage of reinforcement in the How to table is based on a number of parameters, some of which (e.g. c_{nom} = 25 mm) are different to the ones in this WE. Hence, the table should be used with caution.

** It is worth noting that although the crack width limit of 0.4 mm is a code requirement for aesthetics, there may be project-specific requirements for a smaller crack width limit e.g. an element in a monumental/ prestige public building or an element where the viewing distance is small. In such cases, the smaller crack width limit overrides.

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Using PD 6687 Table 5 for $30 < c \leq 40$ and $f_{ck} \geq 25$ MPa:
Conservatively, for a stress of 250 MPa, h = 500 mm
(> 450 mm) and good bond, the maximum spacing
for H25 to achieve 0.3 mm (< 0.4 mm) crack width* = 125 mm — PD 6687: Table 5
The bar spacing (centre-to-centre)
= [300 – (2 x 30 + 2 x 8 + 25)]/2 = 100 mm
100 < 125 ∴ OK

∴ Crack width OK

Note:

The quasi-permanent stress may alternatively be estimated by calculating the neutral axis position and subsequently calculating the force in the steel reinforcement from an assumed elastic stress block. Both methods used to calculate the stress in the reinforcement steel give very similar results (< 2% difference).

Alternative method to calculate the quasi-permanent stress:
Assuming a fully-cracked transformed singly-reinforced section
(i.e. conservatively ignoring the top reinforcement):
The neutral axis from the top fibres:

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

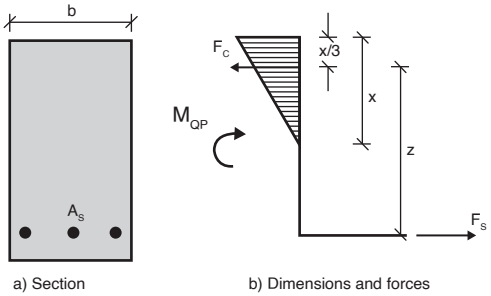
where:

d	(at midspan from Section H3.1.5 a))	=	399.5 mm
A_s	(from Section H3.1.5 a))	=	1296 mm^2
α_e		=	15 — EC2-1-1: Cl. 9.1 (5)
$(A_s) \times (\alpha_e)$		=	19440 mm^2

$x = \{[(19440)^2 + 2 \times 300 \times (19440 \times 399.5)]^{0.5} - (19440)\}/300$
$= [(377913600 + 4659768000)^{0.5} - 19440]/300$ = 171.8 mm

ψ_2 for category B office areas (from before) = 0.3 — NA EC0-1: Table NA.2
The quasi-permanent moment:
$M_{QP} = [(30.2 \times 0.090) + (0.3 \times 11.5 \times 0.100)] \times 6^2$ = 110.3 kNm — H3. How to:Table H3.3

Assuming an elastic stress block (see Figure H3.1.4):



a) Section b) Dimensions and forces

Figure H3.1.4: SLS elastic stress block

* Note that assuming a smaller f_{ck}, and a larger element thickness h, as well as designing to a lower crack width limit is conservative.

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The tensile force and stress in the steel:

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

$= 110.3/(399.5 - 171.8/3) \times 1000$ = 322.2 kN

$\sigma_s = 322.2 \times 1000/1296$ = 249 MPa

ii) Minimum area of longitudinal tension (flexural) reinforcement

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

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

where:

$f_{ct,eff} = f_{ctm}$ (from Section H3.1.9 a) i)) = 2.90 MPa EC2-1-1: Cl. 9.1 (3)

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

where:

b (in metres) = 0.3 m

h (in metres) = 0.45 m

$k_h = 0.8 - 0.6 \times [(\min(0.3, 0.45) - 0.3)$ = 0.80

Check that $k_h \le 0.8$, $k_h \ge 0.5$

$\therefore k_h$ = 0.80

$A_c = bh = 300 \times 450$ = 135,000 mm²

$\therefore A_{s,min,w1} = 0.2 \times 0.8 \times 2.9 \times 135000/500$ = **125 mm²/m**

∴ Minimum area of reinforcement OK

c) Curtailment of main bars

i) Curtailment of bottom bars:

100% of bottom bars = (2 x H25) + (1 x H20) bars = 1296 mm²

At support A:

Curtail 75% main bars 0.08 l from end support G3. How to:

25% = 0.25 x 1296 = **324 mm²** Fig. G3.9c

0.08 l = 0.08 x 6000 = **480 mm** G3. How to:

2 no. H16 (402 mm²) say, 450 mm from support A Fig. G3.9c

At support B:

Curtail 70% main bars 0.30l – a_l from face of support G3. How to:

30% = 0.30 x 1296 = **389 mm²** Fig. G3.9b

a_l** = 1.125 d = 1.125 x 399.5 = 449 mm

$\therefore$ 0.30 l – a_l = 0.3 x 6000 – 449 = **1351 mm**

2 no. H16 (402 mm²) say, 1350 mm from support B

---

* For beams and slab, this check is typically less critical than the minimum reinforcement rules for bending in Section H3.1.9 a) i).

** For beams with shear reinforcement, either the additional force in the longitudinal reinforcement due to shear is calculated, or alternatively, EC2-1-1 Cl. 12.3.2 allows for the curtailment length of the longitudinal reinforcement to be shifted (i.e. extended) by the shift distance a_l. This shift distance is defined as $a_l = z\,(\cot\theta)/2$ in F. (12.5) and can be conservatively simplified as 1.125 d (refer to G3. How to).

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ii) Curtailment of top bars:

100% of top bars = 3 x H25 = 1473 mm²

<u>At support B:</u>

Curtail 40% main bars $0.15\,l + a_l$ but $\geq l_{bd}$ — G3. How to:

60% = 0.6 x 1473 = **884 mm²** — Fig. G3.9a

$a_l = 1.125\,d = 1.125 \times 399.5$ = 449 mm

$\therefore\ 0.15\,l + a_l = 0.15 \times 6000 + 449$ = 1349 mm

Check anchorage length, l_{bd}, for top bars:

The diameter of the curtailed bars, ϕ = 25 mm

Check if $c_d < 1.5\phi$:*

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

$= \min(0.5 \times 75, 38, 38, 3.75 \times 25)$ = 37.5 mm

$\therefore\ c_d = 1.5\phi = 1.5 \times 25 = 37.5$

$\therefore$ Table 11.1 can be used

For ϕ = 25 mm, good bond, σ_{sd} = 435 MPa and f_{ck} = 30 MPa:

l_{bd}/ϕ = 49 — EC2-1-1: Table 11.1

Top bars are in poor bond for h > 300 mm — EC2-1-1: Fig. 11.4

Factor allowing for poor bond = 1.2 — EC2-1-1: Cl. 11.4.2 (2)

$\therefore\ l_{bd} = 1.2 \times 49 \times 25$ (for 435 MPa) = 1470 mm**

(Alternatively, use How to look-up tables for "straight" bars, "poor" bond and 25 mm diameter) — G3. How to: Table G3.2

$\therefore$ Use curtailment distance = **1470 mm**

<u>2 no. H25 (982 mm²) say, 1475 mm from support B</u>

Curtail 75% main bars $0.30\,l + a_l$ — G3. How to:

25% = 0.25 x 1473 = **368 mm²** — Fig. G3.9a

$0.30\,l + a_l = (0.30 \times 6000) + 449$ = 2249 mm

<u>2 no. H16 (402 mm²) say, 2250 mm from support B</u>

d) Minimum reinforcement

Provide minimum longitudinal bottom reinforcement at end supports of:

$0.25\,A_{s,req,span}$ = 0.25 x 1190 = **298 mm²/m** — EC2-1-1: Table 12.1

Provide minimum longitudinal bottom reinforcement at internal supports of:

$0.25\,A_{s,req,span}$ = **298 mm²/m** — EC2-1-1: Table 12.1

<u>Use min. of 2 no. H16 (402 mm²) at all supports (A, B and C)</u>

---

* Note that the anchorage lengths in Table 11.1 of EC2-1-1 assume $c_d = 1.5\phi$. It is not uncommon to have situations where $c_d < 1.5\phi$ (e.g. when large bar diameters are used in internal beams) and in such cases, the anchorage lengths in Table 11.1 may be unconservative. The look-up tables presented in G3. How to take this into account and assume a minimal cover of 25 mm. Alternatively, adjustments may be made to F. (11.3) by multiplying the tabulated values by $(1.5\phi/c_d)^{0.5}$. Note that NA EC2-1-1 limits c_d to 3.0ϕ (see Cl. NA.4.53). — G3. How to: Table G3.2

** The calculated anchorage length is conservative because it corresponds to a steel stress of 435 MPa.

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e) Lap lengths

i) Lap length between H16 U-bar at beam ends (at supports A and C) with main steel:

Top lap:

The smaller lapped bar diameter	=	16 mm	
$c_d = 30 + 8$	=	38	
$\therefore c_d > 1.5\phi = 1.5 \times 16 = 24$			
$\therefore$ Anchorage values in Table 11.1 are conservative			
For $\phi = 16$ mm, good bond, $\sigma_{sd} = 435$ MPa and $f_{ck} = 30$ MPa:			
l_{bd}/ϕ	=	42	EC2-1-1: Table 11.1
Top bars are in poor bond for $h > 300$ mm			EC2-1-1: Fig. 11.4
Factor allowing for poor bond	=	1.2	EC2-1-1: Cl. 11.4.2 (2)
$\therefore l_{bd} = 1.2 \times 42 \times 16$	=	**806 mm***	
$l_{sd} = k_{ls} l_{bd} \geq 15\phi$			EC2-1-1: Table 11.3
where:			
k_{ls}	=	1.0	NA EC2-1-1: NA.2.44
$\therefore l_{sd} = 1 \times 806 > 15 \times 16$	=	**806 mm**	
(Alternatively, use How to look-up tables for "straight" bars, "poor" bond and 16 mm diameter)			G3. How to: Table G3.2
		Say, 810 mm	

Bottom lap:

The smaller lapped bar diameter	=	16 mm	
As before, $c_d > 1.5\phi$			
Bottom bars are in good bond conditions			EC2-1-1: Fig. 11.4
For $\phi = 16$ mm, good bond, $\sigma_{sd} = 435$ MPa and $f_{ck} = 30$ MPa:			
l_{bd}/ϕ	=	42	EC2-1-1: Table 11.1
$\therefore l_{bd} = 42 \times 16$	=	672 mm	
$\therefore l_{sd} = 1 \times 672 > 15 \times 16$	=	**672 mm****	
(Alternatively, use How to look-up tables for "straight" bars, "good" bond and 16 mm diameter)			G3. How to: Table G3.2

For simplicity, make U-bar with equal (top and bottom) legs i.e. use 810 mm lap for both top and bottom steel

ii) Lap length between top H25 and H16 bars

Same as lap length with U-bars

$\therefore l_{sd}$ (for poor bond and H16) = **806 mm**

Lap lengths at support B for top reinforcement, say 810 mm

iii) Lap length between bottom H25 and H16 bars

Same as lap length with U-bars

$\therefore l_{sd}$ (for good bond and H16) = **672 mm**

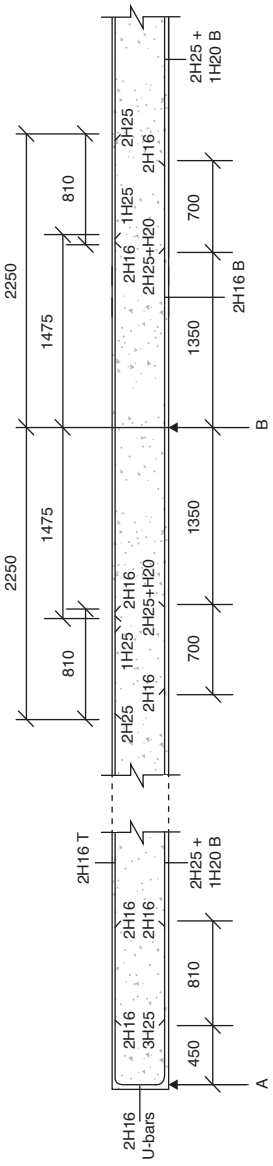
Lap lengths at support B for bottom reinforcement, say 700 mm

* The calculated anchorage length is conservative because it corresponds to a steel stress of 435 MPa and smaller confinement.

** The calculated anchorage length can be further reduced by estimating the reinforcement stress at the lap and by taking into account the concrete confinement (see WE H1.2).

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f) Reinforcement details



Notes:
Links are omitted for clarity
Only one half of the beam is shown – the beam reinforcement arrangement is symmetrical about support B

Figure H3.1.6: Continuous rectangular beam: reinforcement details

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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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