

Ribbed slab

One-way ribbed slab, one rib as a T-beam · EN 1992-1-1 §5.3.2, §6.1, §6.2, §7.4, §9.2 · UK NA / PN-EN

Reference	NAE-RC-021 · version 1.0
Standards	BS EN 1992-1-1:2004 with UK NA · PN-EN 1992-1-1:2008 with Polish NA (NDP sources listed on the NDP sheet)
National Annex	Selectable: UK NA to BS EN 1992-1-1 or Polish NA to PN-EN 1992-1-1
Issued	29 October 2026

What it does

Design of one rib of a one-way ribbed slab as a T-beam: design moments and shears per rib for a simply supported or continuous span, the effective flange width, span bending with the flange in compression, support bending with the rib in compression, shear in the rib with or without links, longitudinal shear between the rib and the topping, deflection by the span to depth ratio and the detailing of the rib and the topping.

How to use

1. Open the Calculation sheet and select the National Annex.
2. Fill in the shaded input cells. Everything else is calculated.
3. Read the Results block; every step below it quotes its clause. Print or save as PDF for your calculation package.

Scope and limitations

Uniformly loaded one-way ribbed slab, constant rectangular ribs (tapered ribs: enter the smallest width and add the extra concrete to g_k), solid zones and trough formers not modelled. Simply supported spans from statics; continuous spans from the coefficients for beams of Concise Eurocode 2 Table 15.3 (all and alternate spans loaded, 15 % redistribution at the supports, $\delta = 0.85$), which apply only where $Q_k \leq G_k$, the load is substantially uniform and there are at least three spans (two-span option: support 0.106, shear 0.63) with the shortest ≥ 0.85 the longest: $Q_k \leq G_k$ is checked and the rest must be confirmed with the input provided, otherwise every coefficient result fails. The coefficient method is a UK simplification of a redistributed linear analysis (5.5); its use under the PL annex is the designer's decision. Interior spans take 0.55 for the shears (the five-span centre span value). Effective flange width by 5.3.2.1 with l_0 of Figure 5.2 (0.85 l end span, 0.7 l interior, 0.15 ($l_1 + l_2$) $\approx 0.3 l$ at a support). Rectangular stress block ($\lambda = 0.8$, $\eta = 1$), no compression bars; where the block runs below the topping the flange and the rib are taken separately. The top bars are designed in the rib width (rib in compression), lie under both layers of the mesh and are spread evenly over the effective flange width at the support (9.2.1.2(2)); at an outer support they are designed for 25 % of the span moment (9.2.1.2(1), UK NA value, applied to both annexes: the recommended value is 0.15), always with $A_{s,min}$. Shear at the support centreline (conservative): without links VRd,c (6.2.2) with the bars in tension there, assumed anchored beyond the section; minimum links are omitted (6.2.1(4)) only where the slab meets 5.3.1(6), transverse ribs included, so that the load can redistribute transversely; otherwise, and where $VR_d > VR_{d,c}$, two-leg links by the variable strut method (6.2.3) with the longitudinal and transverse spacing limits of 9.2.2. Longitudinal shear in the topping (6.2.4): the compression flange in the span, from the outstand force of the stress block at the largest moment and at 0.75 of it (parabolic moment, $\Delta x = l_0/4$), and the tension flange at each continuous support, from the support moment and shear of the coefficients (contraflexure point, Δx half of that distance), with $\theta_f = 45^\circ$ and the relaxation of 6.2.4(6) not used (both conservative); the mesh is the transverse steel. Deflection by (7.16) with p on the concrete above the tension bars (Concise EC2 10.5.2), the flanged-section factor (0.8 for $b_{eff}/b_w \geq 3$, interpolated below), the 7/1 and steel stress factors. Concrete classes up to C50/60, $f_{yk} \leq 500$ MPa; spans of at least 3 h (shorter: a deep beam, 5.3.1(3)). Enter the design span: for Table 15.3 the largest of the spans. The design of transverse ribs, the topping spanning between the ribs under point loads, crack widths, fire (Table 5.9 / 5.11 of EN 1992-1-2) and openings are separate checks. The second-generation EN 1992-1-1:2023 is not yet implemented.

Further reading and independent cross-check

- The Concrete Centre, How to design concrete structures using Eurocode 2: Slabs (ribbed slabs) and Beams (flanged beams).
 - The Concrete Centre, Concise Eurocode 2, Table 15.3 and section 10.5.
 - The Concrete Centre, RC Spreadsheets v4, TCC32 Ribbed slabs (licensed product) can be used as an independent cross-check.
- NA Engineering is not affiliated with The Concrete Centre or CIRIA; their publications are referenced for information only.

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Revision history

v1.0	29 Oct 2026 · First issue
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Project
Element
Date

Job no.
Made by
Checked

Inputs · slab and reinforcement
Shaded cells only

	National Annex		UK NA (BS EN)	
Table 3.1	Concrete strength class		C32/40	
3.2.2	Reinforcement characteristic yield strength (≤ 500 MPa, bars ar	f_{yk}	500	MPa
8.2(2)	Maximum aggregate size	d_g	20	mm
	Span type		Interior span	Continuous: Concise EC2 Table 15.3
	Table 15.3 conditions met (≥ 3 similar spans or 2 spans, uniform load)		Yes	Continuous spans only
5.3.2.2	Effective span (Table 15.3: the largest span)	l	7500	mm
	Rib spacing (centres)	s	750	mm
	Rib width	b_w	150	mm Tapered: smallest width
	Overall depth	h	350	mm
	Topping thickness (flange)	h_f	100	mm
4.4.1	Nominal cover (to the links or the outer bars)	c_{nom}	25	mm
	Bottom bars per rib: number	n_b	2	
	Bottom bars: diameter	φ_b	16	mm
	Top bars per rib (supports): number	n_t	2	
	Top bars: diameter	φ_t	16	mm
	Links in the rib, two legs: diameter (0 = none)	φ_w	6	mm
	Links: spacing	s_w	200	mm Used if $\phi_w > 0$
	Topping mesh: wire diameter (both ways)	φ_m	7	mm
	Topping mesh: spacing	s_m	200	mm
5.3.1(6)	Transverse ribs at a clear spacing ≤ 10 h?		Yes	No: ribs designed as beams

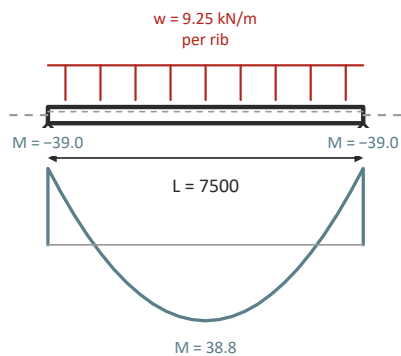
Inputs · loads
Characteristic, per m²

	Permanent load in addition to the self weight	g_k	1.50	kN/m ²
	Variable load	q_k	3.50	kN/m ²
EN 1990 A1.2(B)	Partial factor, permanent	γ_G	1.35	
EN 1990 A1.2(B)	Partial factor, variable	γ_Q	1.50	
7.4.2(2)	Brittle partitions on a span over 7 m?		No	
	Inputs valid and consistent			OK

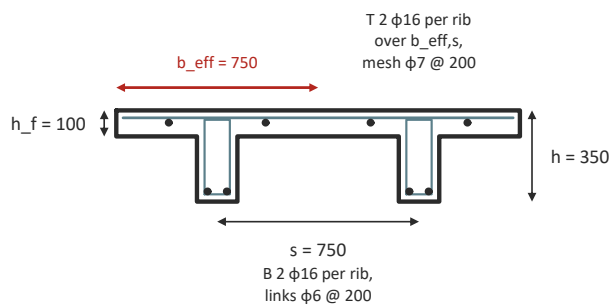
Results · per rib

Span bending $A_{s,req} / A_{s,prov}$ (flange in compression)		0.75		OK
Support bending $A_{s,req} / A_{s,prov}$ (rib in compression)		0.85		OK
Shear in the rib V_{Ed} / V_{Rd} ($V_{Rd,c}$; links where needed)		0.48		OK
Links in the rib ($V_{Ed} / V_{Rd,c}$; slab to 5.3.1(6))		1.27		! Needed
Longitudinal shear, rib to topping v_{Ed}	V_{Ed}	0.66	MPa	OK
Deflection actual $l/d \div$ limiting l/d		0.40		OK
Detailing (bar gaps, mesh, links, $A_{s,max}$)		402	mm ²	OK
Ribbed slab to 5.3.1(6) (spacing, rib depth, topping, transv)	h_f	100	mm	OK
Table 15.3 conditions (continuous spans) Q_k / G_k		0.67		OK

Your rib, to scale · elevation, load and design moments per rib (schematic: sagging below, hogging above)



Cross-section, to scale · two ribs, topping and mesh, bars and links as entered, effective flange width at mid-span



Calculation · loads and actions (per rib)

2.4.2.4(1)	Partial factor, concrete	γ_c	1.50	NDP sheet
2.4.2.4(1)	Partial factor, reinforcement	γ_s	1.15	NDP sheet
3.1.6(1)	Coefficient on compressive strength (flexure)	α_{cc}	0.85	NDP sheet
3.1.6(1)	Coefficient on compressive strength (shear)	α_{cc}	1.00	NDP sheet
6.2.2(1), 6.4.4(1)	Coefficient $0.18 / \gamma_c$	$C_{Rd,c}$	0.120	NDP sheet
6.2.3(2)	Upper limit of $\cot \theta$	$\cot \vartheta_{max}$	2.50	NDP sheet
5.5(4)	Coefficient for x/d limit	k_1	0.40	NDP sheet
5.5(4)	Coefficient for x/d limit	k_2	1.00	NDP sheet
7.4.2(2)	Upper limit on the stress / area factor		1.50	NDP sheet

7.4.2(2)	L/d ≤ 40K applies (1 = yes)		1.00	NDP sheet
Table 3.1	Characteristic cylinder strength	f_{ck}	32	MPa
Table 3.1	Mean tensile strength	f_{ctm}	3.02	MPa
3.1.6(1)	Design compressive strength $\alpha_{cc} f_{ck} / \gamma_c$	f_{cd}	18.13	MPa
3.2.7	Design yield strength	f_{yd}	434.8	MPa
	Self weight 25 (hf + bw (h – hf) / s)	g_{sw}	3.75	kN/m ²
EN 1990 (6.10)	Permanent design load per rib $\gamma_G (g_k + g_{sw}) s$	w_G	5.32	kN/m
EN 1990 (6.10)	Variable design load per rib $\gamma_Q q_k s$	w_Q	3.94	kN/m
	Total design load per rib	w	9.25	kN/m
Table 15.3	Span moment (cG wG + cQ wQ) l ²	M_{span}	38.8	kNm
Table 15.3	Left support moment (hogging)	M_{left}	39.0	kNm
Table 15.3	Right support moment (hogging)	M_{right}	39.0	kNm
Table 15.3	Shear at the left support	V_{left}	38.2	kN
Table 15.3	Shear at the right support	V_{right}	38.2	kN

Calculation · effective flange and bending

5.5(4)	Redistribution ratio implied (1.0, or 0.85 for Table 15.3)	δ	0.85	
5.5(4)	Limiting neutral axis depth $\min((\delta - k_1)/k_2; \epsilon_{cu}/(\epsilon_{cu} + \epsilon_{yd}))$	$x_{u/d}$	0.450	
3.1.7(3)	Limiting K (acc/γc) 0.8ξ (1 – 0.4ξ)	K'	0.167	
	Effective depth, bottom bars $h - c - \phi_w - \phi_b/2$	d_b	311	mm
	Effective depth, top bars $h - c - \max(2\phi_m; \phi_w) - \phi_t/2$	d_t	303	mm Under both mesh layers
Figure 5.2	Distance between zero moments in the span (1.0 / 0.85 / 0.7) l	l_0	5250	mm
5.3.2.1(3)	Half the clear distance between ribs (s – bw)/2	b_1	300	mm
(5.7a)	Flange outstand $\min(0.2 b_1 + 0.1 l_0; 0.2 l_0; b_1)$	$b_{eff,1}$	300	mm
(5.7)	Effective flange width, span $b_w + 2 b_{eff,1}$	b_{eff}	750	mm
(5.7)	Effective flange width at a support ($l_0 = 0.3 l$, for $A_{s,min}$)	$b_{eff,s}$	720	mm
	Moment with the stress block filling the topping $f_{cd} b_{eff} h_f (d -$	M_f	355.0	kNm
3.1.7(3)	Span: stress block depth (in the topping if $M \leq M_f$)	λx	9.3	mm N/A: rib too small
5.5(4)	Span: neutral axis depth ratio $\lambda x / (0.8 d)$	x/d	0.037	
	Span: As required (flange and rib forces) / f_{yd}	$A_{s,req}$	302	mm ² z ≤ 0.95 d
(9.1N)	Span: As,min $\max(0.26 f_{ctm}/f_{yk}; 0.0013) b_w d$	$A_{s,min}$	73	mm ²
	Span: As provided $n_b \pi \phi_b^2 / 4$	$A_{s,prov}$	402	mm ²
	Span: As,prov ≥ max($A_{s,req}$; $A_{s,min}$)	A_s	402	mm ² OK
9.2.1.2(1)	Support design moment $\max(\text{hogging}; 25 \% \text{ of the span momer})$	M_{top}	39.0	kNm
	Support: $K = M / (b_w d^2 f_{ck})$	K	0.089	
3.1.7(3)	Support: lever arm ≤ 0.95 d	z	277	mm N/A: K > K'
	Support: As required $M / (f_{yd} z)$	$A_{s,req}$	324	mm ²
(9.1N)	Support: As,min $\max(0.26 f_{ctm}/f_{yk}; 0.0013) b_{eff,s} d$ (flange in t	$A_{s,min}$	343	mm ²
	Support: As provided $n_t \pi \phi_t^2 / 4$	$A_{s,prov}$	402	mm ²
5.5(4)	Support: x/d of the bars provided $A_s f_{yd} / (0.8 b_w f_{cd} d)$	x/d	0.265	
5.5(4), 9.2.1	Support: As,prov ≥ max($A_{s,req}$; $A_{s,min}$), x/d of the bars ≤ limit w	A_s	402	mm ² OK

Calculation · shear in the rib

6.2.2(1)	Left support: pl of the bars in tension there	ρ_l	0.0088	
(6.2)	Left support: VRd,c	$V_{Rd,c}$	30.1	kN
6.2.2(1)	Right support: pl of the bars in tension there	ρ_l	0.0088	
(6.2)	Right support: VRd,c	$V_{Rd,c}$	30.1	kN
(6.2)	Shear utilisation without links $\max(V_{Ed} / VRd,c)$		1.27	
	Larger support shear and the smaller depth	V_{Ed}	38.2	kN
6.2.3(1)	Inner lever arm 0.9 min(d)	z	273	mm
(6.6N)	Strength reduction factor $v_1 = 0.6 (1 - f_{ck}/250)$	v_1	0.523	
3.1.6(1)	Design compressive strength for shear	$f_{cd,v}$	21.33	MPa
(6.9)	Crushing ratio $2 V_{Ed} / (b_w z v_1 f_{cd})$ (= $V_{Ed} / VRd,max$ at 45°)		0.167	
6.2.3(2)	Strut inclination $1 \leq \cot \theta \leq \cot \theta_{max}$	$\cot \theta$	2.500	
(6.9)	Maximum shear resistance $b_w z v_1 f_{cd} / (\cot \theta + \tan \theta)$	$V_{Rd,max}$	157.4	kN
(6.8)	Links required $V_{Ed} / (z f_{ywd} \cot \theta)$	$A_{sw/s}$	0.129	mm ² /mm

(9.5N)	Minimum links where links are used $0.08 \sqrt{f_{ck}} / f_{yk} \cdot b_w$	$A_{sw/s_{min}}$	0.136	mm ² /mm	
	Links provided $2 \pi \phi_w^2 / 4 / s$	$A_{sw/s_{prov}}$	0.283	mm ² /mm	
	Links needed (minimum where $V_{Ed} \leq V_{Rd,c}$)		0.136	mm ² /mm	
(9.6N)	Maximum longitudinal link spacing $0.75 d$	$s_{l,max}$	227	mm	
9.2.2(8)	Transverse spacing of the two legs $b_w - 2c - \phi_w$	s_t	94	mm	
(9.8N)	Maximum transverse spacing of legs $\min(0.75 d; 600)$	$s_{t,max}$	227	mm	
6.2.1(4), 6.2.3	Shear: no links ($V_{Ed} \leq V_{Rd,c}$, slab to 5.3.1(6)), or links $\geq$ needed,	V_{Ed}	38.2	kN	OK

Calculation · longitudinal shear between the rib and the topping (6.2.4)

3.1.7(3)	Stress block depth at 0.75 of the span moment	$\lambda x_{0.75}$	7.0	mm	
	Outstand force at the span moment $f_{cd} b_{eff,1} \min(\lambda x; h_f)$	F_{out}	50.6	kN	
	Outstand force at 0.75 of the span moment	$F_{out,0.75}$	37.8	kN	
6.2.4(3)	Length Δx = half the distance from zero to the largest moment	Δx	1313	mm	
6.2.4(3)	Larger force change over either half $\max(F_{0.75}; F - F_{0.75})$	ΔF_d	37.8	kN	
(6.20)	Compression flange $v_{Ed} = \Delta F_d / (h_f \Delta x)$	$v_{Ed,c}$	0.29	MPa	
6.2.4(3)	Left support: distance to the contraflexure point (0: no hogging)	x_0	1196	mm	
(6.20)	Left support: tension flange $v_{Ed} = \frac{1}{2} \Delta F_d / (h_f x_0/2)$	$v_{Ed,t}$	0.66	MPa	
6.2.4(3)	Right support: distance to the contraflexure point (0: no hogging)	x_0	1196	mm	
(6.20)	Right support: tension flange $v_{Ed} = \frac{1}{2} \Delta F_d / (h_f x_0/2)$	$v_{Ed,t}$	0.66	MPa	
(6.20)	Longitudinal shear stress (largest)	v_{Ed}	0.66	MPa	
(6.22)	Crushing limit $v_{fcd} \sin \theta_f \cos \theta_f$ ($\theta_f = 45^\circ$)	$v_{Rd,max}$	4.74	MPa	
(6.21)	Transverse steel required $v_{Ed} h_f / (f_{yd} \cot \theta_f)$	A_{sf/s_f}	152	mm ² /m	
	Mesh provided $\pi \phi_m^2 / 4 / s_m$	A_{sf/s_f}	192	mm ² /m	
6.2.4	Topping: no crushing and the mesh $\geq$ required	v_{Ed}	0.66	MPa	OK

Tension flange: the larger change of the tension force M/z over the two halves of the hogging length (z of the rib section at each end), taken entirely in the flange, half in each outstand (conservative for any arrangement of the top bars and the mesh); anchor the bars in the flange beyond the strut needed to bring their force back to the rib (6.2.4(7)).

Calculation · deflection (7.4.2)

Concise EC2 10.5.2	Tension bars for deflection $A_{s,req} / (b_{eff} h_f + b_w (d - h_f))$	ρ	0.00283		N/A: bending fails
(7.16)	Reference ratio $\rho_0 = \sqrt{f_{ck}} \cdot 10^{-3}$	ρ_0	0.00566		
Table 7.4N	K for the structural system	K	1.5		
(7.16)	Basic span to depth ratio	l/d	69.0		
7.4.2(2)	Flanged section factor (0.8 for $b_{eff}/b_w \geq 3$, interpolated below)	F_1	0.800		
7.4.2(2)	Long span factor $7/l$ ($l > 7$ m, brittle partitions)	F_2	1.000		
(7.17)	Steel stress factor ($A_{s,prov} / A_{s,req}$)(500 / f_{yk}) ($UK \leq 1.5$)	F_3	1.332		
7.4.2	Limiting span to depth ratio ($UK NA: \leq 40 K$)	l/d_{lim}	60.0		
	Actual span to depth ratio	l/d	24.1		
7.4.2	Deflection $l/d \leq$ limit (with the bending checks passing)	l/d	24.1		OK

Calculation · detailing

8.2(2)	Minimum clear gap, bottom bars $\max(\phi_b; d_g + 5; 20)$	a_{min}	25	mm	
	Clear gap between the bottom bars $(b_w - 2c - 2\phi_w - n_b \phi_b)/(n)$	a	56	mm	
8.2(2)	Minimum vertical gap $\max(\phi_b; \phi_t; d_g + 5; 20)$	$a_{min,v}$	25	mm	
	Vertical gap between the top and bottom bars	a_v	248	mm	
9.2.1.1(3)	Maximum reinforcement $0.04 A_c$ ($A_c = s h_f + b_w (h - h_f)$)	$A_{s,max}$	4500	mm ²	
9.3.1.1(3)	Maximum mesh spacing in the topping $\min(3 h_f; 400)$	s_{max}	300	mm	
8.2(2)	Minimum clear gap of the mesh $\max(\phi_m; d_g + 5; 20)$	$a_{min,m}$	25	mm	
8.2(2)	Minimum clear gap, top bars $\max(\phi_t; d_g + 5; 20)$	$a_{min,t}$	25	mm	
	Clear gap between the top bars $b_{eff,s} / n_t - \phi_t$ (spread evenly)	a_t	344	mm	
4.4.1	Cover from the topping soffit to the top bars in the outstands	c_t	45	mm	
8.2(2), 9.2	Bars, links and mesh fit, top bars in the topping; $A_s \leq A_{s,max}$		402	mm ²	OK
5.3.1(6)	Rib spacing ≤ 1500 , rib depth below the topping $\leq 4 b_w$, $h_f \geq m_a$	h_f	100	mm	OK

Within the limits of 5.3.1(6), transverse ribs included, the ribbed slab may be analysed as a solid slab. $b_{eff,s}$ uses $l_0 = 0.3 l$ with the largest span, the upper bound of 0.15 ($l_1 + l_2$), so $A_{s,min}$ at the supports is not understated. Spread the top bars over the effective flange width at the support (9.2.1.2(2)); curtail and anchor them for the design moments (9.2.1.3). At an end support at least a quarter of the span bars continue and are anchored (9.2.1.4). The topping spanning between the ribs and point loads on it are not checked.

Nationally Determined Parameters

Values used by the Calculation sheet for the selected National Annex. Sources: UK NA to BS EN 1992-1-1:2004, Table NA.1; PN-EN 1992-1-1:2008, Załącznik krajowy NA. Edit only if your project specifies different values.

Parameter	Clause	UK NA	PL NA	Notes
Partial factor, concrete γ_c	2.4.2.4(1)	1.5	1.4	Persistent & transient. UK NA: recommended; PL: NA.2, Table NA.2
Partial factor, reinforcement γ_s	2.4.2.4(1)	1.15	1.15	UK NA: recommended; PL: NA.2, Table NA.2
α_{cc} (compression in flexure / axial)	3.1.6(1)	0.85	1	UK NA: 0.85; PL: recommended (NA.9)
α_{cc} (shear and punching)	3.1.6(1)	1	1	UK NA: 1.0 for other phenomena; PL: recommended (NA.9)
α_{ct} (design tensile strength)	3.1.6(2)	1	1	Recommended value in both annexes
CRd,c numerator (CRd,c = value / γ_c)	6.2.2(1), 6.4.4(1)	0.18	0.18	Recommended value in both annexes
k1 (axial stress term, punching)	6.4.4(1)	0.1	0.1	Recommended value in both annexes
$v_{Rd,max}$ factor ($v_{Rd,max} = \text{factor} \cdot v \cdot f_{cd}$)	6.4.5(3)	0.5	0.4	UK NA: 0.5 v fcd; PL: recommended value (NA.9), 0.4 v fcd since EN 1992-1-1 AC:2010 (PN-EN 1992-1-1:2008)
kmax ($v_{Ed} \leq k_{max} \cdot v_{Rd,c}$ at u1)	6.4.5	2	1.5	UK NA 6.4.5(3): 2 vRd,c. PL: no limit in PN-EN 1992-1-1:2008; 1.5 (A1:2014 recommendation) applied conse
ϕ_{large} (large bar threshold, mm)	8.8(1)	40	32	UK NA: 40 mm; PL: 32 mm recommended (NA.9)
cot θ upper limit (variable strut)	6.2.3(2)	2.5	2	UK NA: 2.5 (1.25 with axial tension); PL: NA.4, $1 \leq \cot \theta \leq 2.0$
k1 (x/d limit, $\delta \geq k_1 + k_2 x_u/d$)	5.5(4)	0.4	0.44	UK NA for $f_{yk} \leq 500$ MPa; PL: recommended (NA.9)
k2 (x/d limit, $\epsilon_{cu2} = 0.0035$)	5.5(4)	1	1.25	UK NA: $0.6 + 0.0014/\epsilon_{cu2}$; PL: $1.25(0.6 + 0.0014/\epsilon_{cu2})$
Upper limit on the (310/ σ_s) factor (0 = none)	7.4.2(2)	1.5	0	UK NA Table NA.5 Note 5: (310/ σ_s) or (500/ f_{yk})($A_{s,prov}/A_{s,req}$) ≤ 1.5 ; PL: none
L/d not to exceed 40 K (1 = applies)	7.4.2(2)	1	0	UK NA Table NA.5 Note 6; not in EN text
ϕ_{min} (longitudinal bars in columns, mm)	9.5.2(1)	12	6	UK NA: 12 mm; PL: NA.6, 6 mm
k3 (crack spacing)	7.3.4(3)	3.4	3.4	Recommended value in both annexes
k4 (crack spacing)	7.3.4(3)	0.425	0.425	Recommended value in both annexes