Table 1: EC8 rules for detailing and dimensioning of primary beams (secondary beams: as in DCL)

	DC H	DCM	DCL
"critical region" length	$1.5h_w$	h_w	
Longitudinal bars (L):			
ρ_{min}, tension side	$0.5f_{ctm}/f_{yk}$		$0.26f_{ctm}/f_{yk}$, 0.13%(0)
ρ_{max}, critical regions(1)	$\rho'+0.0018f_{cd}/(\mu_\phi\varepsilon_{sy,d}f_{yd})$(1)		0.04
$A_{s,min,}$ top & bottom	2Φ14 (328mm^2)	-	
$A_{s,min}$, top-span	$A_{s,top\text{-}supports}/4$	-	
$A_{s,min}$, critical regions bottom	$0.5A_{s,top}$(2)		-
$A_{s,min}$, supports bottom	$A_{s,bottom\text{-}span}/4$(0)		
d_{bL}/h_c - bar crossing interior joint(3)	$\leq \dfrac{6.25(1+0.8\nu_d)}{(1+0.75\dfrac{\rho'}{\rho_{max}})}\dfrac{f_{ctm}}{f_{yd}}$	$\leq \dfrac{7.5(1+0.8\nu_d)}{(1+0.5\dfrac{\rho'}{\rho_{max}})}\dfrac{f_{ctm}}{f_{yd}}$	-
d_{bL}/h_c - bar anchored at exterior joint(3)	$\leq 6.25(1+0.8\nu_d)\dfrac{f_{ctm}}{f_{yd}}$	$\leq 7.5(1+0.8\nu_d)\dfrac{f_{ctm}}{f_{yd}}$	-
Transverse bars (w):			
(i) outside critical regions			
spacing $s_w\leq$	0.75d		
$\rho_w\geq$	$0.08\sqrt{}(f_{ck}(MPa)/f_{yk}(MPa)$(0)		
(ii) in critical regions:			
$d_{bw}\geq$	6mm		
spacing $s_w\leq$	$6d_{bL}$, $\dfrac{h_w}{4}$, $24d_{bw}$, 175mm	$8d_{bL}$, $\dfrac{h_w}{4}$, $24d_{bw}$, 225mm	-
Shear design:			
V_{Ed}, seismic(4)	$1.2\dfrac{\sum M_{Rb}}{l_{cl}}\pm V_{o,g+\psi_2 q}$ (4)	$\dfrac{\sum M_{Rb}}{l_{cl}}\pm V_{o,g+\psi_2 q}$ (4)	From the analysis for the "seismic design situation"
$V_{Rd,max}$ seismic (5)	As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)b_{wo}zf_{cd}\sin 2\delta$ (5), with $1\leq\cot\delta\leq 2.5$		
$V_{Rd,s}$, outside critical regions(5)	As in EC2: $V_{Rd,s}=b_w z\rho_w f_{ywd}\cot\delta$ (5), with $1\leq\cot\delta\leq 2.5$		
$V_{Rd,s}$, critical regions(5)	$V_{Rd,s}=b_w z\rho_w f_{ywd}$ ($\delta=45^o$)	As in EC2: $V_{Rd,s}=b_w z\rho_w f_{ywd}\cot\delta$, with $1\leq\cot\delta\leq 2.5$	
If $\zeta\equiv V_{Emin}/V_{Emax}$(6) <-0.5: inclined bars at angle $\pm\alpha$ to beam axis, with cross-section A_s/direction	If $V_{Emax}/(2+\zeta)f_{ctd}b_w d>1$: $A_s=0.5V_{Emax}/f_{yd}\sin\alpha$ & stirrups for $0.5V_{Emax}$	-	

(0) NDP (Nationally Determined Parameter) according to Eurocode 2. The Table gives the value recommended in Eurocode 2.

(1) μ_ϕ is the value of the curvature ductility factor that corresponds to the basic value, q_o, of the behaviour factor used in the design.

(2) The minimum area of bottom steel, $A_{s,min}$, is in addition to any compression steel that may be needed for the verification of the end section for the ULS in bending under the (absolutely) maximum negative (hogging) moment from the analysis for the "seismic design situation", M_{Ed}.

(3) h_c is the column depth in the direction of the bar, $\nu_d = N_{Ed}/A_c f_{cd}$ is the column axial load ratio, for the algebraically minimum value of the axial load in the "seismic design situation", with compression taken as positive.

(4) At a member end where the moment capacities around the joint satisfy: $\sum M_{Rb}>\sum M_{Rc}$, M_{Rb} is replaced in the calculation of the design shear force, V_{Ed}, by $M_{Rb}(\sum M_{Rc}/\sum M_{Rb})$

(5) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, $d-d_1$.

(6) V_{Emax}, $V_{E,min}$are the algebraically maximum and minimum values of V_{Ed} resulting from the ± sign; $V_{Emax}$is the absolutely largest of the two values, and is taken positive in the calculation of ζ; the sign of V_{Emin} is determined according to whether it is the same as that of V_{Emax} or not.

Table 2: EC8 rules for detailing and dimensioning of primary columns (secondary columns as in DCL)

	DCH	DCM	DCL
Cross-section sides, h_c, $b_c \geq$	0.25m; $h_v/10$ if $\theta=P\delta/Vh>0.1$ (1)	-	
"critical region" length (1) $\geq$	$1.5h_c$, $1.5b_c$, 0.6m, $l_c/5$	h_c, b_c, 0.45m, $l_c/5$	h_c, b_c
	Longitudinal bars (L):		
ρ_{min}	1%		$0.1N_d/A_cf_{yd}$, 0.2% (0)
ρ_{max}	4%		4% (0)
$d_{bL}\geq$	8mm		
bars per side $\geq$	3		2
Spacing between restrained bars	≤150mm	≤200mm	-
distance of unrestrained bar from nearest restrained bar	≤150mm		
	Transverse bars (w):		
Outside critical regions:			
$d_{bw}\geq$	6mm, $d_{bL}/4$		
spacing $s_w \leq$	$20d_{bL}$, h_c, b_c, 400mmm		$12d_{bL}$, $0.6h_c$, $0.6b_c$, 240mm
at lap splices, if d_{bL}>14mm: $s_w \leq$	$12d_{bL}$, $0.6h_c$, $0.6b_c$, 240mm		
Within critical regions: (2)			
$d_{bw}\geq$ (3)	6mm, $0.4(f_{yd}/f_{ywd})^{1/2}d_{bL}$	6mm, $d_{bL}/4$	
$s_w\leq$ (3),(4)	$6d_{bL}$, $b_o/3$, 125mm	$8d_{bL}$, $b_o/2$, 175mm	-
$\omega_{wd}\geq$ (5)	0.08	-	
$\alpha\omega_{wd}\geq$ (4),(5),(6),(7)	$30\mu_\phi^* \nu_d \varepsilon_{sy,d} b_c/b_o-0.035$	-	
In critical region at column base:			
$\omega_{wd}\geq$	0.12	0.08	-
$\alpha\omega_{wd}\geq$ (4),(5),(6),(8),(9)	$30\mu_\phi \nu_d \varepsilon_{sy,d} b_c/b_o-0.035$		-
Capacity design check at beam-column joints: (10)	$1.3\sum M_{Rb}\leq\sum M_{Rc}$ No moment in transverse direction of column		-
Verification for M_x-M_y-N:	Truly biaxial, or uniaxial with $(M_z/0.7, N)$, $(M_y/0.7, N)$		
Axial load ratio $\nu_d=N_{Ed}/A_cf_{cd}$	≤ 0.55	≤ 0.65	-
	Shear design:		
V_{Ed} seismic (11)	$1.3\frac{\sum M_{Rc}^{ends}}{l_{cl}}$ (11)	$1.1\frac{\sum M_{Rc}^{ends}}{l_{cl}}$ (11)	From the analysis for the "seismic design situation"
$V_{Rd,max}$ seismic (12), (13)	As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(MPa)/250)b_{wo}zf_{cd}\sin2\delta$, with $1\leq\cot\delta\leq2.5$		
$V_{Rd,s}$ seismic (12), (13), (14)	As in EC2: $V_{Rd,s}=b_wz\rho_wf_{ywd}\cot\delta+N_{Ed}(h-x)/l_{cl}$ (13) with $1\leq\cot\delta\leq2.5$		

(0) Note (0) of Table 1 applies.

(1) h_v is the distance of the inflection point to the column end further away, for bending within a plane parallel to the side of interest; l_c is the column clear length.

(2) For DCM: If a value of q not greater than 2 is used for the design, the transverse reinforcement in critical regions of columns with axial load ratio ν_d not greater than 0.2 may just follow the rules applying to DCL columns.

(3) For DCH: In the two lower storeys of the building, the requirements on d_{bw}, s_w apply over a distance from the end section not less than 1.5 times the critical region length.

(4) Index c denotes the full concrete section and index o the confined core to the centreline of the hoops; $b_o$is the smaller side of this core.

(5) ω_{wd} is the ratio of the volume of confining hoops to that of the confined core to the centreline of the hoops, times f_{yd}/f_{cd}.

(6) α is the "confinement effectiveness" factor, computed as $\alpha = \alpha_s\alpha_n$; where: $\alpha_s = (1-s/2b_o)(1-s/2h_o)$ for hoops and $\alpha_s = (1-s/2b_o)$ for spirals; $\alpha_n = 1$ for circular hoops and $\alpha_n=1-\{b_o/((n_h-1)h_o)+h_o/((n_b-1)b_o)\}/3$ for rectangular hoops with n_b legs parallel to the side of the core with length b_o and n_h legs parallel to the one with length h_o.

(7) For DCH: at column ends protected from plastic hinging through the capacity design check at beam-column joints, $\mu_\phi^*$is the value of the curvature ductility factor that corresponds to 2/3 of the basic value,

q_o, of the behaviour factor used in the design; at the ends of columns where plastic hinging is not prevented because of the exemptions listed in Note (10) below, μ_ϕ^* is taken equal to μ_ϕ defined in Note (1) of Table 1 (see also Note (9) below); $\varepsilon_{sy,d}= f_{yd}/E_s$.

(8) Note (1) of Table 1 applies.

(9) For DCH: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented, because of the waivers listed in Note (10) below.

(10) The capacity design check does not need to be fulfilled at beam-column joints: (a) of the top floor, (b) of the ground storey in two-storey buildings with axial load ratio ν_d not greater than 0.3 in all columns, (c) if shear walls resist at least 50% of the base shear parallel to the plane of the frame (wall buildings or wall-equivalent dual buildings), and (d) in one-out-of-four columns of plane frames with columns of similar size.

(11) At a member end where the moment capacities around the joint satisfy: $\sum M_{Rb}<\sum M_{Rc}$, M_{Rc} is replaced by $M_{Rc}(\sum M_{Rb}/\sum M_{Rc})$.

(12) z is the internal lever arm, taken equal to 0.9d or to the distance between the tension and the compression reinforcement, $d-d_1$.

(13) The axial load, N_{Ed}, and its normalized value, ν_d, are taken with their most unfavourable value in the seismic design situation for the shear verification (considering both the demand, V_{Ed}, and the capacity, V_{Rd}).

(14) x is the compression zone depth at the end section in the ULS of bending with axial load.

Table 3: EC8 rules for the detailing and dimensioning of ductile walls

	DCH	DCM	DCL
Web thickness, $b_{wo}\geq$	max(150mm, $h_{storey}/20$)		-
critical region length, $h_{cr}\geq$	$\geq \max(l_w, H_w/6)$ (1) $\leq \min(2l_w, h_{storey})$ if wall ≤6 storeys $\leq \min(2l_w, 2h_{storey})$ if wall > 6 storeys		-
Boundary elements:			
a) in critical region:			
- length l_c from edge ≥	$0.15l_w$, $1.5b_w$, length over which $\varepsilon_c > 0.0035$		-
- thickness b_w over l_c ≥	200mm; $h_{st}/15$ if $l_c \leq \max(2b_w, l_w/5)$, $h_{st}/10$ if $l_c > \max(2b_w, l_w/5)$		-
- vertical reinforcement:			
ρ_{min} over $A_c=l_cb_w$	0.5%		0.2% (0)
ρ_{max} over A_c	4% (0)		
- confining hoops (w) (2):			
$d_{bw}\geq$	8mm	in the part of the section where $\rho_L>2\%$: as over the rest of the wall (case c, below)	
spacing $s_w\leq$(3)	$\min(25d_{bh}$, 250mm)		
$\omega_{wd}\geq$(2)	0.12	0.08	-
$\alpha\omega_{wd}\geq$(3),(4)	$30\mu_\phi(\nu_d+\omega_v)\varepsilon_{sy,d}b_w/b_o-0.035$		-
b) storey above critical region	As in critical region, but $\alpha\omega_{wd}$ & ω_{wd}: 50% of those required in critical region	as over the rest of the wall (case c, below)	
c) over the rest of the wall height:	In parts of the section where $\rho_L>2\%$: - distance of unrestrained bar in compression zone from nearest restrained bar ≤150mm; - hoops with $d_{bw}\geq \max(6\text{mm}, d_{bL}/4)$ & spacing $s_w\leq \min(12d_{bL}, 0.6b_{wo}$, 240mm)(0) up to a distance of $4b_w$ above or below floor beams or slabs, or $s_w\leq \min(20d_{bL}, b_{wo}$, 400mm)(0) beyond that distance		
Web:			
- vertical bars (v):			
$\rho_{v,min}$	wherever $\varepsilon_c>0.2\%$: 0.5%; elsewhere 0.2%		0.2%(0)
$\rho_{v,max}$	4%		
$d_{bv}\geq$	8mm	-	
$d_{bv}\leq$	$b_{wo}/8$	-	
spacing $s_v\leq$	$\min(25d_{bv}$, 250mm)	$\min(3b_{wo}$, 400mm)	
- horizontal bars:			
ρ_{hmin}	0.2%	max(0.1%, $0.25\rho_v$)(0)	
$d_{bh}\geq$	8mm	-	
$d_{bh}\leq$	$b_{wo}/8$	-	
spacing $s_h\leq$	$\min(25d_{bh}$, 250mm)	400mm	
axial load ratio $\nu_d=N_{Ed}/A_cf_{cd}$	≤0.35	≤0.4	-
Design moments M_{Ed}:	If $H_w/l_w\geq 2$, design moments from linear envelope of maximum moments M_{Ed} from analysis for the "seismic design situation", shifted up by the "tension shift" a_l		From analysis for "seismic design situation"
Shear design:			
Design shear force V'_{Ed} = shear force V'_{Ed} from the analysis for "seismic design situation", times factor ε:	if $H_w/l_w\leq 2$(5): $\varepsilon=1.2M_{Rdo}/M_{Edo}\leq q$ if $H_w/l_w>2$(5), (6): $\varepsilon=\sqrt{\left(1.2\frac{M_{Rdo}}{M_{Edo}}\right)^2+0.1\left(q\frac{S_e(T_C)}{S_e(T_1)}\right)^2}\leq q$	ε=1.5	ε=1.0
Design shear force in walls of dual systems with $H_w/l_w>2$, for z between $H_w/3$ and H_w: (7)	$V_{Ed}(z)=\left(\frac{0.75z}{H_w}-\frac{1}{4}\right)\varepsilon V_{Ed}(0)+\left(1.5-\frac{1.5z}{H_w}\right)\varepsilon V_{Ed}\left(\frac{H_w}{3}\right)$		From analysis for "seismic design situation"
$V_{Rd,max}$ outside critical region	As in EC2: $V_{Rd,max}=0.3(1-f_{ck}(\text{MPa})/250)b_{wo}(0.8l_w)f_{cd}\sin2\delta$, with $1\leq\cot\delta\leq 2.5$		

$V_{Rd,max}$ in critical region	40% of EC2 value	As in EC2
$V_{Rd,s}$ outside critical region	As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\delta$ with $1\leq\cot\delta\leq2.5$	
$V_{Rd,s}$ in critical region; web reinforcement ratios. ρ_h, ρ_v		
(i) if $\alpha_s=M_{Ed}/V_{Ed}l_w\geq2$: $\rho_v=\rho_{v,min}$, ρ_h from $V_{Rd,s}$:	As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\delta$ with $1\leq\cot\delta\leq2.5$	
(ii) if $\alpha_s<2$: ρ_h from $V_{Rd,s}$: (8)	$V_{Rd,s}=V_{Rd,c}+b_{wo}\alpha_s(0.75l_w)\rho_h f_{yhd}$	As in EC2: $V_{Rd,s}=b_{wo}(0.8l_w)\rho_h f_{ywd}\cot\delta$ with $1\leq\cot\delta\leq2.5$
ρ_v from: (9)	$\rho_v f_{yvd}\geq\rho_h f_{yhd}-N_{Ed}/(0.8l_w b_{wo})$	
Resistance to sliding shear: via bars with total area A_{si} at angle $\pm\phi$ to the horizontal (10)	$V_{Rd,s}=A_{si}f_{yd}\cos\phi+A_{sv}\min(0.25f_{yd},\ 1.3\sqrt{(f_{yd}f_{cd})})+0.3(1-f_{ck}(MPa)/250)b_{wo}xf_{cd}$	
$\rho_{v,min}$ at construction joints (9),(11)	$0.0025,\ \dfrac{1.3f_{ctd}-\dfrac{N_{Ed}}{A_c}}{f_{yd}+1.5\sqrt{f_{cd}f_{yd}}}$	-

(0) Note (0) of Tables 1 and 2 applies.

(1) l_w is the long side of the rectangular wall section or rectangular part thereof; $H_w$is the total height of the wall; h_{storey} is the storey height.

(2) For DC M: If for the maximum value of axial force in the wall from the analysis for the "seismic design situation" the wall axial load ratio $\nu_d=N_{Ed}/A_c f_{cd}$ satisfies $\nu_d\leq0.15$, the DCL rules may be applied for the confining reinforcement of boundary elements; these DCL rules apply also if this value of the wall axial load ratio is $\nu_d\leq0.2$ but the value of q used in the design of the building is not greater than 85% of the q-value allowed when the DC M confining reinforcement is used in boundary elements.

(3) Notes (4), (5), (6) of Table 2 apply for the confined core of boundary elements.

(4) μ_ϕ is the value of the curvature ductility factor that corresponds to the product of the basic value q_o of the behaviour factor times the value of the ratio M_{Edo}/M_{Rdo} at the base of the wall (see Note (5)); $\varepsilon_{sy,d}=f_{yd}/E_s$, ω_{vd} is the mechanical ratio of the vertical web reinforcement.

(5) $M_{Edo}$is the moment at the wall base from the analysis for the "seismic design situation"; M_{Rdo} is the design value of the flexural capacity at the wall base for the axial force N_{Ed} from the analysis for the same "seismic design situation".

(6) $S_e(T_1)$ is the value of the elastic spectral acceleration at the period of the fundamental mode in the horizontal direction (closest to that) of the wall shear force multiplied by ε; $S_e(T_c)$ is the spectral acceleration at the corner period T_C of the elastic spectrum.

(7) A dual structural system is one in which walls resist between 35 and 65% of the seismic base shear in the direction of the wall shear force considered; z is distance from the base of the wall.

(8) For b_w and d in m, f_{ck} in MPa, ρ_L denoting the tensile reinforcement ratio, N_{Ed} in kN, $V_{Rd,c}$ (in kN) is given by:

$$V_{R,c}=\left\{\max\left[180(100\rho_1)^{1/3},\ 35\sqrt{1+\sqrt{\frac{0.2}{d}}}\ f_c^{1/6}\right]\left(1+\sqrt{\frac{0.2}{d}}\right)f_c^{1/3}+0.15\frac{N}{A_c}\right\}b_w d$$

N_{Ed} is positive for compression and its minimum value from the analysis for the "seismic design situation" is used; if the minimum value is negative (tension), $V_{Rd,c}=0$.

(9) The minimum value of the axial force from the analysis for the "seismic design situation" is used as N_{Ed} (positive for compression).

(10) A_{sv} is the total area of web vertical bars and of any additional vertical bars placed in boundary elements against shear sliding; x is the depth of the compression zone.

(11) $f_{ctd=fct\kappa,0.05}/\gamma_c$ is the design value of the (5%-fractile) tensile strength of concrete.