f_{ctd} is the design value of the tensile strength of concrete, in accordance with EN 1992-1-1:2004.

(4) As an alternative to the rule specified in **(3)** of this subclause, integrity of the joint after diagonal cracking may be ensured by horizontal hoop reinforcement. To this end the following total area of horizontal hoops should be provided in the joint.

a) In interior joints:

$$A_{sh} f_{ywd} \geq \gamma_{Rd}(A_{s1}+A_{s2}) f_{yd}(1-0,8\,\nu_d) \tag{5.36a}$$

b) In exterior joints:

$$A_{sh} f_{ywd} \geq \gamma_{Rd} A_{s2} f_{yd}(1-0,8\,\nu_d) \tag{5.36b}$$

where γ_{Rd} is equal to 1,2 (cf **5.5.2.3(2)**) and the normalised axial force ν_d refers to the column above the joint in expression (5.36a), or to the column below the joint in expression (5.36b).

(5) The horizontal hoops calculated as in **(3)** and **(4)** of this subclause should be uniformly distributed within the depth h_{jw} between the top and bottom bars of the beam. In exterior joints they should enclose the ends of beam bars bent toward the joint.

(6) Adequate vertical reinforcement of the column passing through the joint should be provided, so that:

$$A_{sv,i} \geq (2/3) \cdot A_{sh} \cdot (h_{jc} / h_{jw}) \tag{5.37}$$

where A_{sh} is the required total area of the horizontal hoops in accordance with **(3)** and **(4)** of this subclause and $A_{sv,i}$ denotes the total area of the intermediate bars placed in the relevant column faces between corner bars of the column (including bars contributing to the longitudinal reinforcement of columns).

(7) **5.4.3.3(1)** applies.

(8) **5.4.3.3(2)** applies.

(9)P **5.4.3.3(3)**P applies.

5.5.3.4 Ductile Walls

5.5.3.4.1 Bending resistance

(1)P The bending resistance shall be evaluated and verified as for columns, under the most unfavourable axial force for the seismic design situation.

(2) In primary seismic walls the value of the normalised axial force ν_d should not exceed 0,35.

5.5.3.4.2 Diagonal compression failure of the web due to shear

(1) The value of $V_{Rd,max}$ may be calculated as follows:

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a) outside the critical region:

as in EN 1992-1-1:2004, with the length of the internal lever arm, z, equal to $0,8l_w$ and the inclination of the compression strut to the vertical, $\tan\theta$, equal to 1,0.

b) in the critical region:

40% of the value outside the critical region.

5.5.3.4.3 Diagonal tension failure of the web due to shear

(1)P The calculation of web reinforcement for the ULS verification in shear shall take into account the value of the shear ratio $\alpha_s = M_{Ed}/(V_{Ed}\, l_w)$. The maximum value of α_s in a storey should be used for the ULS verification of the storey in shear.

(2) If the ratio $\alpha_s \geq 2{,}0$, the provisions of in EN 1992-1-1:2004 **6.2.3(1)-(7)** apply, with the values of z and $\tan\theta$ taken as in **5.5.3.4.2(1)** a).

(3) If $\alpha_s < 2{,}0$ the following provisions apply:

a) the horizontal web bars should satisfy the following expression (see EN 1992-1-1:2004, **6.2.3(8)**):

$$V_{Ed} \leq V_{Rd,c} + 0{,}75\rho_h f_{yd,h} b_{wo} \alpha_s l_w \tag{5.38}$$

where

ρ_h is the reinforcement ratio of horizontal web bars ($\rho_h = A_h/(b_{wo} \cdot s_h)$);

$f_{yd,h}$ is the design value of the yield strength of the horizontal web reinforcement;

$V_{Rd,c}$ is the design value of the shear resistance for members without shear reinforcement, in accordance to EN 1992-1-1:2004,

In the critical region of the wall $V_{Rd,c}$ should be equal to 0 if the axial force N_{Ed} is tensile.

b) Vertical web bars, anchored and spliced along the height of the wall in accordance with EN 1992-1-1:2004, should be provided to satisfy the condition:

$$\rho_h f_{yd,h} b_{wo} z \leq \rho_v f_{yd,v}\, b_{wo}\, z + \min N_{Ed} \tag{5.39}$$

where

ρ_v is the reinforcement ratio of vertical web bars ($\rho_v = A_v/b_{wo} \cdot s_v$);

$f_{yd,v}$ is the design value of the yield strength of the vertical web reinforcement;

and where the axial force N_{Ed} is positive when compressive.

(4) Horizontal web bars should be fully anchored at the ends of the wall section, e.g. through 90° or 135° hooks.

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(5) Horizontal web bars in the form of elongated closed or fully anchored stirrups may also be assumed to fully contribute to the confinement of the boundary elements of the wall.

5.5.3.4.4 Sliding shear failure

(1)P At potential sliding shear planes (for example, at construction joints) within critical regions the following condition shall be satisfied:

$$V_{Ed} \leq V_{Rd,S}$$

where $V_{Rd,S}$ is the design value of the shear resistance against sliding.

(2) The value of $V_{Rd,S}$ may be as follows:

$$V_{Rd,S} = V_{dd} + V_{id} + V_{fd} \tag{5.40}$$

with:

$$V_{dd} = \min \begin{cases} 1{,}3 \cdot \Sigma A_{sj} \cdot \sqrt{f_{cd} \cdot f_{yd}} \\ 0{,}25 \cdot f_{yd} \cdot \Sigma A_{sj} \end{cases} \tag{5.41}$$

$$V_{id} = \Sigma A_{si} \cdot f_{yd} \cdot \cos\varphi \tag{5.42}$$

$$V_{fd} = \min \begin{cases} \mu_f \cdot \left[\left(\Sigma A_{sj} \cdot f_{yd} + N_{Ed}\right) \cdot \xi + M_{Ed} / z\right] \\ 0{,}5\eta \cdot f_{cd} \cdot \xi \cdot l_w \cdot b_{wo} \end{cases} \tag{5.43}$$

where

V_{dd} is the dowel resistance of the vertical bars;

V_{id} is the shear resistance of inclined bars (at an angle φ to the potential sliding plane, e.g. construction joint);

V_{fd} is the friction resistance;

μ_f is the concrete-to-concrete friction coefficient under cyclic actions, which may be assumed equal to 0,6 for smooth interfaces and to 0,7 for rough ones, as defined in EN 1992-1-1:2004, **6.2.5(2)**;

z is the length of the internal lever arm;

ξ is the normalised neutral axis depth;

ΣA_{sj} is the sum of the areas of the vertical bars of the web and of additional bars arranged in the boundary elements specifically for resistance against sliding;

ΣA_{si} is the sum of the areas of all inclined bars in both directions; large diameter bars are recommended for this purpose;

$$\eta = 0{,}6\ (1 - f_{ck}(\mathrm{MPa})/250) \tag{5.44}$$

N_{Ed} is assumed to be positive when compressive.

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(3) For squat walls the following should be satisfied :

a) at the base of the wall V_{id} should be greater than $V_{Ed}/2$;

b) at higher levels V_{id} should be greater than $V_{Ed}/4$.

(4) Inclined bars should be fully anchored on both sides of potential sliding interfaces and should cross all sections of the wall within a distance of $0{,}5 \cdot l_w$ or $0{,}5 \cdot h_w$, whichever is smaller, above the critical base section.

(5) Inclined bars lead to an increase of the bending resistance at the base of the wall, which should be taken into account whenever the acting shear V_{Ed} is computed in accordance with the capacity design rule (see **5.5.2.4.1(6)**P and **(7)** and **5.5.2.4.2(2)**). Two alternative methods may be used.

a) The increase of bending resistance ΔM_{Rd}, to be used in the calculation of V_{Ed}, may be estimated as:

$$\Delta M_{Rd} = \frac{1}{2} \cdot \Sigma A_{si} \cdot f_{yd} \cdot \sin \varphi \cdot l_i \tag{5.45}$$

where

l_i is the distance between centrelines of the two sets of inclined bars, placed at an angle of $\pm\phi$ to the potential sliding plane, measured at the base section;

and the other symbols are as in expression (5.42).

b) An acting shear V_{Ed} may be computed disregarding the effect of the inclined bars. In expression (5.42) V_{id} is the net shear resistance of the inclined bars (i.e. the actual shear resistance reduced by the increase of the acting shear). Such net shear resistance of the inclined bars against sliding may be estimated as:

$$V_{id} = \Sigma A_{si} \cdot f_{yd} \cdot \left[\cos\varphi - 0{,}5 \cdot l_i \cdot \sin\varphi / (\alpha_s \cdot l_w)\right] \tag{5.46}$$

5.5.3.4.5 Detailing for local ductility

(1) Paragraph **5.4.3.4.2(1)** applies.

(2) Paragraph **5.4.3.4.2(2)** applies.

(3) Paragraph **5.4.3.4.2(3)** applies.

(4) Paragraph **5.4.3.4.2(4)** applies.

(5) Paragraph **5.4.3.4.2(5)** applies.

(6) Paragraph **5.4.3.4.2(6)** applies.

(7) Paragraph **5.4.3.4.2(8)** applies.

(8) Paragraph **5.4.3.4.2(10)** applies.

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(9) If the wall is connected to a flange with thickness $b_f \geq h_s/15$ and width $l_f \geq h_s/5$ (where h_s denotes the clear storey height), and the confined boundary element needs to extend beyond the flange into the web for an additional length of up to $3b_{wo}$, then the thickness b_w of the boundary element in the web should only follow the provisions in **5.4.1.2.3(1)** for b_{wo} (Figure 5.11).

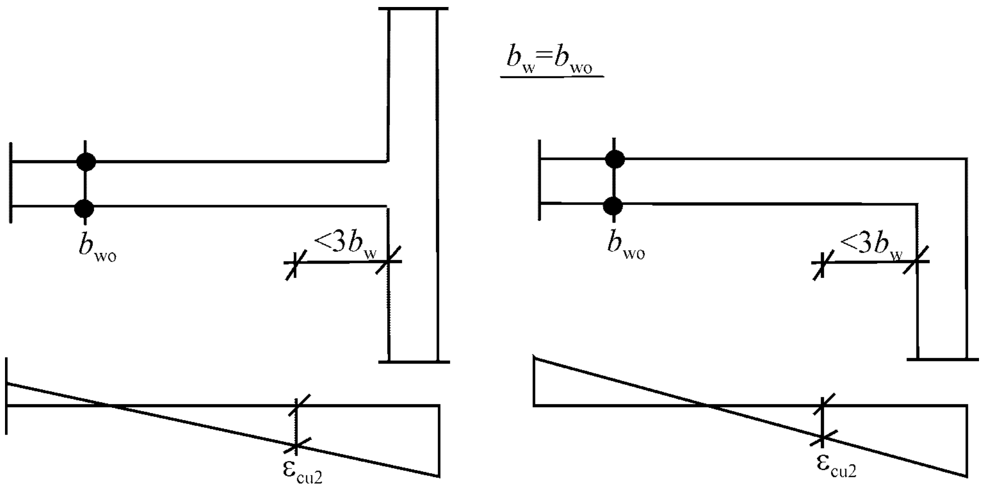

Figure 5.11: Minimum thickness of confined boundary elements in DCH walls with large flanges

(10) Within the boundary elements of walls the requirements specified in **5.5.3.2.2(12)** apply and there should be a minimum value of ω_{wd} of 0,12. Overlapping hoops should be used, so that every other longitudinal bar is engaged by a hoop or cross-tie.

(11) Above the critical region boundary elements should be provided for one more storey, with at least half the confining reinforcement required in the critical region.

(12) **5.4.3.4.2(11)** applies.

(13)P Premature web shear cracking of walls shall be prevented, by providing a minimum amount of web reinforcement: $\rho_{h,min} = \rho_{v,min} = 0,002$.

(14) The web reinforcement should be provided in the form of two grids (curtains) of bars with the same bond characteristics, one at each face of the wall. The grids should be connected through cross-ties spaced at about 500 mm.

(15) Web reinforcement should have a diameter of not less than 8 mm, but not greater than one-eighth of the width b_{wo} of the web. It should be spaced at not more than 250 mm or 25 times the bar diameter, whichever is smaller.

(16) To counterbalance the unfavourable effects of cracking along cold joints and the associated uncertainties, a minimum amount of fully anchored vertical reinforcement should be provided across such joints. The minimum ratio of this reinforcement, ρ_{min}, necessary to re-establish the resistance of uncracked concrete against shear, is:

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$$\rho_{\min} \geq \begin{cases} \left(1{,}3 \cdot f_{\text{ctd}} - \dfrac{N_{\text{Ed}}}{A_{\text{w}}}\right) / \left(f_{\text{yd}} \cdot \left(1 + 1{,}5\sqrt{f_{\text{cd}} / f_{\text{yd}}}\right)\right) \\ 0{,}0025 \end{cases} \tag{5.47}$$

where A_{w} is the total horizontal cross-sectional area of the wall and N_{Ed} shall be positive when compressive.

5.5.3.5 Coupling elements of coupled walls

(1)P Coupling of walls by means of slabs shall not be taken into account, as it is not effective.

(2) The provisions of **5.5.3.1** may only be applied to coupling beams, if either one of the following conditions is fulfilled:

a) Cracking in both diagonal directions is unlikely. An acceptable application rule is:

$$V_{\text{Ed}} \leq f_{\text{ctd}} b_{\text{w}} d \tag{5.48}$$

b) A prevailing flexural mode of failure is ensured. An acceptable application rule is: $l/h \geq 3$.

(3) If neither of the conditions in **(2)** is met, the resistance to seismic actions should be provided by reinforcement arranged along both diagonals of the beam, in accordance with the following (see Figure 5.12):

a) It should be ensured that the following expression is satisfied:

$$V_{\text{Ed}} \leq 2 \cdot A_{\text{si}} \cdot f_{\text{yd}} \cdot \sin\alpha \tag{5.49}$$

where

V_{Ed} is the design shear force in the coupling element ($V_{\text{Ed}} = 2 \cdot M_{\text{Ed}}/l$);

A_{si} is the total area of steel bars in each diagonal direction;

α is the angle between the diagonal bars and the axis of the beam.

b) The diagonal reinforcement should be arranged in column-like elements with side lengths at least equal to $0{,}5b_{\text{w}}$; its anchorage length should be 50% greater than that required by EN 1992-1-1:2004.

c) Hoops should be provided around these column-like elements to prevent buckling of the longitudinal bars. The provisions of **5.5.3.2.2(12)** apply for the hoops..

d) Longitudinal and transverse reinforcement should be provided on both lateral faces of the beam, meeting the minimum requirements specified in EN 1992-1-1:2004 for deep beams. The longitudinal reinforcement should not be anchored in the coupled walls and should only extend into them by 150 mm.

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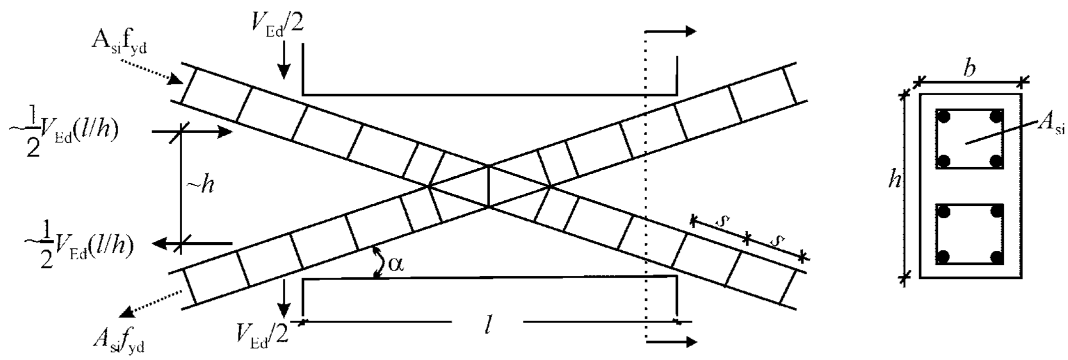

Figure 5.12: Coupling beams with diagonal reinforcement

5.6 Provisions for anchorages and splices

5.6.1 General

(1)P EN 1992-1-1:2004, Section **8** for the detailing of reinforcement applies, with the additional rules of the following sub-clauses.

(2)P For hoops used as transverse reinforcement in beams, columns or walls, closed stirrups with 135° hooks and extensions of length $10d_{bw}$ shall be used.

(3)P In DCH structures the anchorage length of beam or column bars anchored within beam-column joints shall be measured from a point on the bar at a distance $5d_{bL}$ inside the face of the joint, to take into account the yield penetration due to cyclic post-elastic deformations (for a beam example, see Figure 5.13a).

5.6.2 Anchorage of reinforcement

5.6.2.1 Columns

(1)P When calculating the anchorage or lap length of column bars which contribute to the flexural strength of elements in critical regions, the ratio of the required area of reinforcement over the actual area of reinforcement $A_{s,req}/A_{s,prov}$ shall be assumed to be 1.

(2)P If, under the seismic design situation, the axial force in a column is tensile, the anchorage lengths shall be increased to 50% longer than those specified in EN 1992-1-1:2004.

5.6.2.2 Beams

(1)P The part of beam longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops.

(2)P To prevent bond failure the diameter of beam longitudinal bars passing through beam-column joints, d_{bL}, shall be limited in accordance with the following expressions:

a) for interior beam-column joints:

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$$\frac{d_{\mathrm{bL}}}{h_{\mathrm{c}}} \leq \frac{7{,}5 \cdot f_{\mathrm{ctm}}}{\gamma_{\mathrm{Rd}} \cdot f_{\mathrm{yd}}} \cdot \frac{1+0{,}8 \cdot \nu_{\mathrm{d}}}{1+0.75 k_{\mathrm{D}} \cdot \rho' / \rho_{\max}} \tag{5.50a}$$

b) for exterior beam-column joints:

$$\frac{d_{\mathrm{bL}}}{h_{\mathrm{c}}} \leq \frac{7{,}5 \cdot f_{\mathrm{ctm}}}{\gamma_{\mathrm{Rd}} \cdot f_{\mathrm{yd}}} \cdot \left(1+0{,}8 \cdot \nu_{\mathrm{d}}\right) \tag{5.50b}$$

where

h_c is the width of the column parallel to the bars;

f_{ctm} is the mean value of the tensile strength of concrete;

f_{yd} is the design value of the yield strength of steel;

ν_d is the normalised design axial force in the column, taken with its minimum value for the seismic design situation ($\nu_d = N_{Ed}/f_{cd} \cdot A_c$);

k_D is the factor reflecting the ductility class equal to 1 for DCH and to 2/3 for DCM;

ρ' is the compression steel ratio of the beam bars passing through the joint;

ρ_{max} is the maximum allowed tension steel ratio (see **5.4.3.1.2(4)** and **5.5.3.1.3(4)**);

γ_{Rd} is the model uncertainty factor on the design value of resistances, taken as being equal to 1,2 or 1,0 respectively for DCH or DCM (due to overstrength owing to strain-hardening of the longitudinal steel in the beam).

The limitations above (expressions (5.50)) do not apply to diagonal bars crossing joints.

(3) If the requirement specified in **(2)**P of this clause cannot be satisfied in exterior beam-column joints because the depth, h_c, of the column parallel to the bars is too shallow, the following additional measures may be taken, to ensure anchorage of the longitudinal reinforcement of beams.

a) The beam or slab may be extended horizontally in the form of exterior stubs (see Figure 5.13a).

b) Headed bars or anchorage plates welded to the end of the bars may be used (see Figure 5.13b).

c) Bends with a minimum length of $10d_{bL}$ and transverse reinforcement placed tightly inside the bend of the bars may be added (see Figure 5.13c).

(4)P Top or bottom bars passing through interior joints, shall terminate in the members framing into the joint at a distance not less than l_{cr} (length of the member critical region, see **5.4.3.1.2(1)**P and **5.5.3.1.3(1)**P) from the face of the joint.

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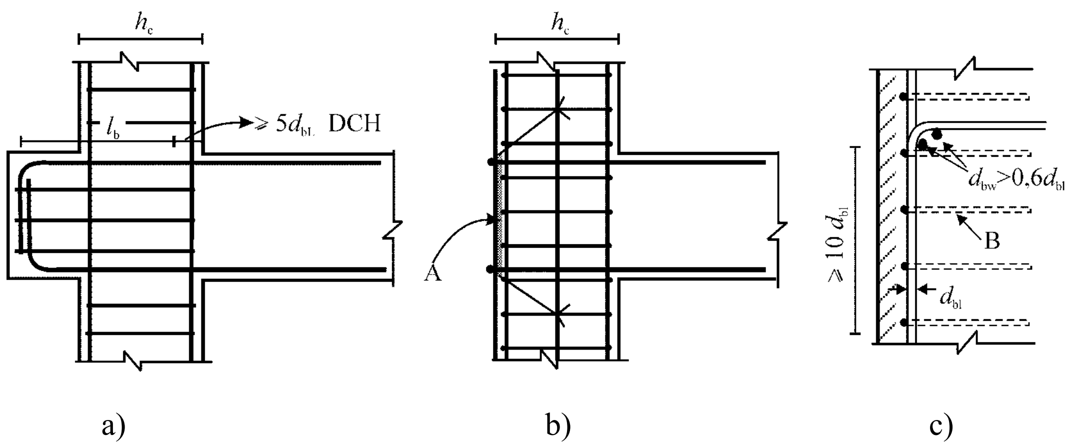

Key

A anchor plate;

B hoops around column bars

Figure 5.13: Additional measures for anchorage in exterior beam-column joints

5.6.3 Splicing of bars

(1)P There shall be no lap-splicing by welding within the critical regions of structural elements.

(2)P There may be splicing by mechanical couplers in columns and walls, if these devices are covered by appropriate testing under conditions compatible with the selected ductility class.

(3)P The transverse reinforcement to be provided within the lap length shall be calculated in accordance with EN 1992-1-1:2004. In addition, the following requirements shall also be met.

a) If the anchored and the continuing bar are arranged in a plane parallel to the transverse reinforcement, the sum of the areas of all spliced bars, ΣA_{sL}, shall be used in the calculation of the transverse reinforcement.

b) If the anchored and the continuing bar are arranged within a plane normal to the transverse reinforcement, the area of transverse reinforcement shall be calculated on the basis of the area of the larger lapped longitudinal bar, A_{sL};

c) The spacing, s, of the transverse reinforcement in the lap zone (in millimetres) shall not exceed

$$s = \min\{h/4; 100\} \tag{5.51}$$

where h is the minimum cross-sectional dimension (in millimetres).

(4) The required area of transverse reinforcement A_{st} within the lap zone of the longitudinal reinforcement of columns spliced at the same location (as defined in EN

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1992-1-1:2004), or of the longitudinal reinforcement of boundary elements in walls, may be calculated from the following expression:

$$A_{st} = s\,(d_{bl}/50)(f_{yld}/f_{ywd}) \tag{5.52}$$

where

A_{st} is the area of one leg of the transverse reinforcement;

d_{bL} is the diameter of the spliced bar;

s is the spacing of the transverse reinforcement;

f_{yld} is the design value of the yield strength of the longitudinal reinforcement;

f_{ywd} is the design value of the yield strength of the transverse reinforcement.

5.7 Design and detailing of secondary seismic elements

(1)P Clause **5.7** applies to elements designated as secondary seismic elements, which are subjected to significant deformations in the seismic design situation (e.g. slab ribs are not subject to the requirements of **5.7**). Such elements shall be designed and detailed to maintain their capacity to support the gravity loads present in the seismic design situation, when subjected to the maximum deformations under the seismic design situation.

(2)P Maximum deformations due to the seismic design situation shall be calculated in accordance with **4.3.4** and shall account for P-Δ effects in accordance with **4.4.2.2(2)** and **(3)**. They shall be calculated from an analysis of the structure in the seismic design situation, in which the contribution of secondary seismic elements to lateral stiffness is neglected and primary seismic elements are modelled with their cracked flexural and shear stiffness.

(3) Secondary seismic elements are deemed to satisfy the requirements of **(1)**P of this subclause if bending moments and shear forces calculated for them on the basis of: a) the deformations of **(2)**P of this subclause; and b) their cracked flexural and shear stiffness, do not exceed their design flexural and shear resistance M_{Rd} and V_{Rd}, respectively, as these are determined on the basis of EN 1992-1-1:2004.

5.8 Concrete foundation elements

5.8.1 Scope

(1)P The following paragraphs apply for the design of concrete foundation elements, such as footings, tie-beams, foundation beams, foundation slabs, foundation walls, pile caps and piles, as well as for connections between such elements, or between them and vertical concrete elements. The design of these elements shall follow the rules of EN 1998-5:2004, **5.4**.

(2)P If design action effects for the design of foundation elements of dissipative structures are derived on the basis of capacity design considerations in accordance with **4.4.2.6(2)**P, no energy dissipation is expected in these elements in the seismic design situation. The design of these elements may follow the rules of **5.3.2(1)**P.

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