(7) The horizontal reinforcement within the joint body may be reduced by $\Delta A_{sx} \leq \Delta A_{sz}$, provided that the ratio of the horizontal reinforcement remaining within the joint body satisfies expression (5.26). The tensile reinforcement of the "beam" top and bottom fibres at the faces of the pier should then be increased by ΔA_{sx}, over the reinforcement required in the relevant "beam" sections for the verification in flexure under capacity design effects. Additional bars to cover this requirement should be placed within the joint width b_j; these bars should be adequately anchored, so as to be fully effective at a distance h_b from the pier face.

5.6.3.6 Deck verification

(1)P It shall be verified that no significant yielding occurs in the deck. This verification shall be carried out:

– for bridges of limited ductile behaviour, under the most adverse design action effect in accordance with **5.5**;

– for bridges of ductile behaviour, under the capacity design effects determined in accordance with **5.3**.

(2) When the horizontal component of the seismic action in the transverse direction of the bridge is considered, yielding of the deck for flexure within a horizontal plane is considered to be significant if the reinforcement of the top slab of the deck yields up to a distance from its edge equal to 10% of the top slab width, or up to the junction of the top slab with a web, whichever is closer to the edge of the top slab.

(3) When verifying the deck on the basis of capacity design effects for the seismic action acting in the transverse direction of the bridge, the significant reduction of the torsional stiffness of the deck with increasing torsional moments should be accounted for. Unless a more accurate calculation is made, the values specified in **2.3.6.1(4)** may be assumed for bridges of limited ductile behaviour, or 70% of these values for bridges of ductile behaviour.

5.7 Resistance verification for steel and composite members

5.7.1 Steel piers

5.7.1.1 General

(1) For the verification of the pier under multi-component action effects, **5.6.1(1)** applies.

(2)P Energy dissipation is allowed to take place only in the piers and not in the deck.

(3)P For bridges designed for ductile behaviour, the provisions of EN 1998-1:2004, **6.5.2**, **6.5.4** and **6.5.5** for dissipative structures apply.

(4) The provisions of EN 1998-1:2004, **6.5.3** apply. However cross-sectional class 3 is allowed only when $q \leq 1{,}5$.

(5) The provisions of EN 1998-1:2004, **6.9** apply for all bridge piers.

5.7.1.2 Piers as moment resisting frames

(1)P In bridges designed for ductile behaviour, the design values of the axial force, N_{Ed}, and shear forces, $V_{E,d}$, in piers consisting of moment resisting frames shall be assumed to be equal to the capacity design action effects N_C and V_C, respectively, as the latter are specified in **5.3**.

(2)P The design of the sections of plastic hinges both in beams and columns of the pier shall satisfy the provisions of EN 1998-1:2004, **6.6.2**, **6.6.3** and **6.6.4**, using the values of N_{Ed} and V_{Ed} as specified in **(1)**P.

5.7.1.3 Piers as frames with concentric bracings

(1)P The provisions of EN 1998-1: 2004 apply with the following modifications for bridges designed for ductile behaviour.

– The design values for the axial shear force shall be in accordance with **5.3,** taking the force in all diagonals as corresponding to the overstrength $\gamma_o N_{pl,Rd}$ of the weakest diagonal (see **5.3** for γ_o).

– The second part of expression (6.12) in EN 1998-1:2004, **6.7.4** shall be replaced by the capacity design action $N_{Ed} = N_C$

5.7.1.4 Piers as frames with eccentric bracings

(1)P The provisions of EN 1998-1:2004, **6.8** apply.

5.7.2 Steel or composite deck

(1)P In bridges designed for ductile behaviour ($q > 1,5$) the deck shall be verified for the capacity design effects in accordance with **5.3**. In bridges designed for limited ductile behaviour ($q \leq 1,5$) the verification of the deck shall be carried out using the design action effects from the analysis in accordance with expression (5.4). The verifications may be carried out in accordance with the relevant rules of EN 1993-2:2005 or EN 1994-2:2005 for steel or composite decks, respectively.

5.8 Foundations

5.8.1 General

(1)P Bridge foundation systems shall be designed to conform to the general requirements set forth in EN 1998-5:2004, **5.1**. Bridge foundations shall not be intentionally used as sources of hysteretic energy dissipation and therefore shall, as far as practicable, be designed to remain elastic under the design seismic action.

(2)P Soil structure interaction shall be assessed where necessary on the basis of the relevant provisions of EN 1998-5: 2004, Section **6**.

5.8.2 Design action effects

(1)P For the purpose of resistance verifications, the design action effects on the foundations shall be determined in accordance with **(2)**P to **(4)**.

(2)P Bridges of limited ductile behaviour ($q \leq 1,5$) and bridges with seismic isolation

The design action effects shall be those resulting from expression (5.4) with seismic effects obtained from the linear analysis of the structure for the seismic design situation in accordance with **5.5**, with the analysis results for the design seismic action multiplied by the q-factor used (i.e. effectively using $q = 1$).

(3)P Bridges of ductile behaviour ($q > 1,5$).

The design action effects shall be obtained by applying the capacity design procedure to the piers in accordance with **5.3**.

(4) For bridges designed on the basis of non-linear analysis, the provisions of **4.2.4.4(2)e** apply.

5.8.3 Resistance verification

(1)P The resistance verification of the foundations shall be carried out in accordance with EN 1998-5:2004, **5.4.1** (Direct foundations) and **5.4.2** (Piles and piers).

6 DETAILING

6.1 General

(1)P The rules of this Section apply only to bridges designed for ductile behaviour and aim to ensure a minimum level of curvature/rotation ductility at the plastic hinges.

(2)P For bridges of limited ductile behaviour, rules for the detailing of critical sections and specific non-ductile components are specified in **6.5**.

(3)P In general, plastic hinge formation is not allowed in the deck. Therefore there is no need for the application of special detailing rules other than those applying for the design of bridges for the non-seismic actions.

6.2 Concrete piers

6.2.1 Confinement

6.2.1.1 General requirements

(1)P Ductile behaviour of the compression concrete zone shall be ensured within the potential plastic hinge regions.

(2)P In potential hinge regions where the normalised axial force (see **5.3(3)**) exceeds the limit:

$$\eta_k = N_{Ed}/A_c f_{ck} > 0{,}08 \qquad (6.1)$$

confinement of the compression zone in accordance with **6.2.1.4** should be provided, except as specified in **(3)**.

(3)P No confinement is required in piers if, under ultimate limit state conditions, a curvature ductility μ_Φ = 13 for bridges of ductile behaviour, or μ_Φ = 7 for bridges of limited ductile behaviour, is attainable, with the maximum compressive strain in the concrete not exceeding the value of:

$$\varepsilon_{cu2} = 0{,}35\% \qquad (6.2)$$

NOTE: The condition of **(3)**P may be attainable in piers with flanged section, when sufficient flange area is available in the compressive zone.

(4) In cases of deep compression zones, the confinement should extend at least up to the depth where the value of the compressive strain exceeds $0{,}5\varepsilon_{cu2}$

(5)P The quantity of confining reinforcement is defined through the mechanical reinforcement ratio:

$$\omega_{wd} = \rho_w f_{yd}/f_{cd} \qquad (6.3)$$

where:

(a) In rectangular sections:

ρ_w is the transverse reinforcement ratio defined as:

$$\rho_w = \frac{A_{sw}}{s_L b} \tag{6.4}$$

where:

A_{sw} is the total area of hoops or ties in the one direction of confinement;

s_L is the spacing of hoops or ties in the longitudinal direction;

b is the dimension of the concrete core perpendicular to the direction of the confinement under consideration, measured to the outside of the perimeter hoop.

(b) In circular sections:

The volumetric ratio ρ_w of the spiral reinforcement relative to the concrete core is used:

$$\rho_w = \frac{4A_{sp}}{D_{sp} \cdot s_L} \tag{6.5}$$

where:

A_{sp} is the area of the spiral or hoop bar

D_{sp} is the diameter of the spiral or hoop bar

s_L is the spacing of these bars.

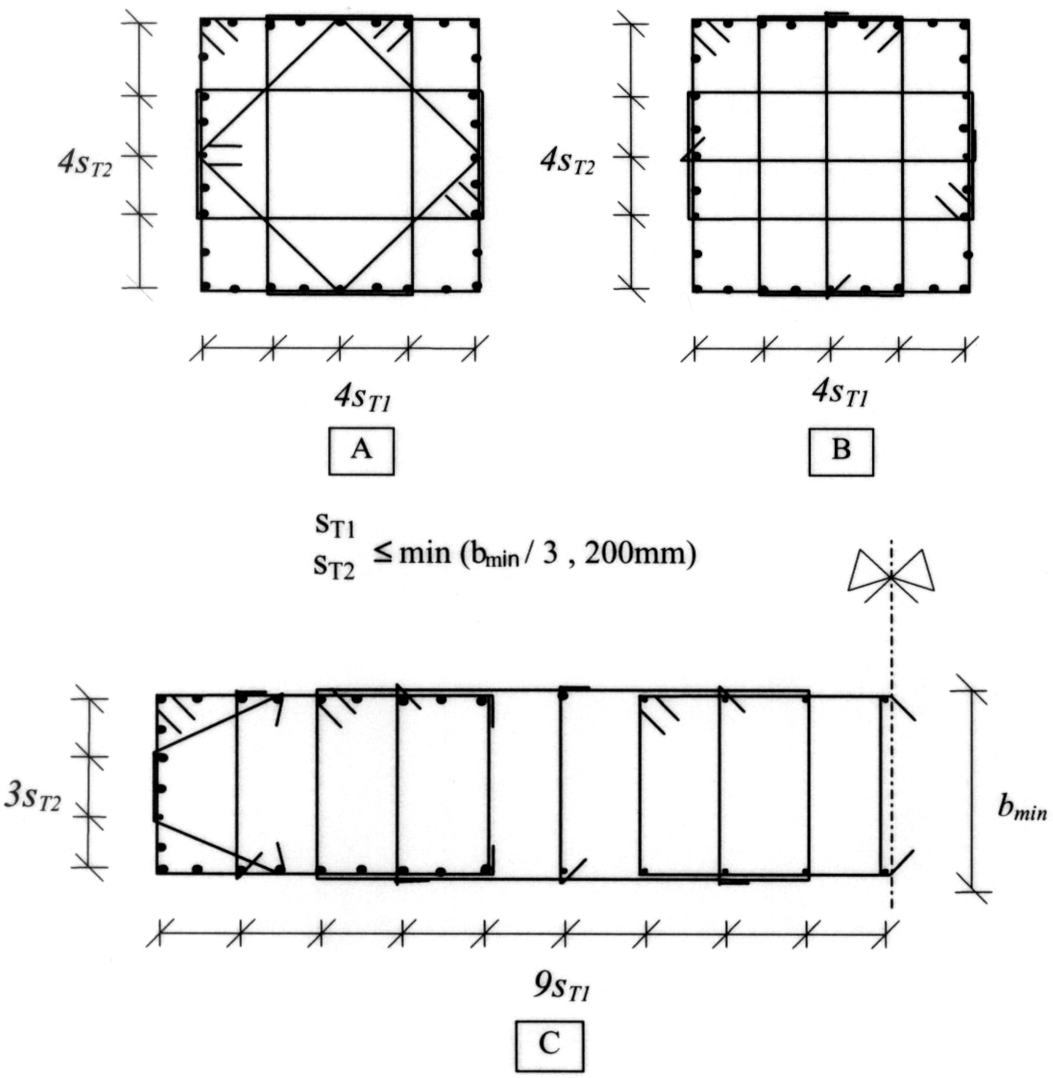
6.2.1.2 Rectangular sections

(1)P The spacing of hoops or ties in the longitudinal direction, s_L, shall satisfy both of the following conditions:

- $s_L \leq 6$ times the longitudinal bar diameter, d_{bL}
- $s_L \leq 1/5$ of the smallest dimension of the confined concrete core, to the hoop centre line.

(2)P The transverse distance s_T between hoop legs or supplementary cross-ties shall not exceed 1/3 of the smallest dimension b_{min} of the concrete core to the hoop centre line, nor 200mm (see Figure 6.1a).

(3)P Bars inclined at an angle $\alpha > 0$ to the transverse direction in which ρ_w refers to shall be assumed to contribute to the total area A_{sw} of expression (6.4) by their area multiplied by $\cos\alpha$.



Key
A : 4 closed overlapping hoops
B : 3 closed overlapping hoops plus cross-ties
C : closed overlapping hoops plus cross-ties

Figure 6.1a: Typical confinement details in concrete piers with rectangular section using overlapping rectangular hoops and cross-ties

6.2.1.3 Circular sections

(1)P The spacing of spiral or hoop bars, s_L, shall satisfy both of the following conditions:

$s_L \leq$ 6 times the longitudinal bar diameter, d_{bL}

$s_L \leq$ 1/5 of the diameter of the confined concrete core to the hoop centre line.

6.2.1.4 Required confining reinforcement

(1)P Confinement is implemented through rectangular hoops and/or cross-ties or through circular hoops or spirals.

NOTE The National Annex may prohibit the use of a certain type of confinement reinforcement. It is recommended that all types of confinement are allowed.

(2)P The minimum amount of confining reinforcement shall be determined as follows:

– for rectangular hoops and cross-ties

$$\omega_{wd,r} \geq \max\left(\omega_{w,req};\ \frac{2}{3}\omega_{w,min}\right) \tag{6.6}$$

where:

$$\omega_{w,req} = \frac{A_c}{A_{cc}}\ \lambda\eta_k + 0{,}13\frac{f_{yd}}{f_{cd}}(\rho_L\text{-}0{,}01) \tag{6.7}$$

where:

A_c is the area of the gross concrete section;

A_{cc} is the confined (core) concrete area of the section to the hoop centerline;

$\omega_{w,min}$, λ are factors specified in Table 6.1; and

ρ_L is the reinforcement ratio of the longitudinal reinforcement.

Depending on the intended seismic behaviour of the bridge, the minimum values specified in Table 6.1 apply.

Table 6.1: Minimum values of λ and $\omega_{w,min}$

Seismic Behaviour	λ	$\omega_{w,min}$
Ductile	0,37	0,18
Limited ductile	0,28	0,12

– for circular hoops or spirals

$$\omega_{wd.c} \geq \max\left(1{,}4\omega_{w,req};\ \omega_{w,min}\right) \tag{6.8}$$

(3)P When rectangular hoops and cross-ties are used, the minimum reinforcement condition shall be satisfied in both transverse directions.

(4)P Interlocking spirals/hoops are quite efficient for confining approximately rectangular sections. The distance between the centres of interlocking spirals/hoops shall not exceed $0{,}6D_{sp}$, where D_{sp} is the diameter of the spiral/hoop (see Figure 6.1b).

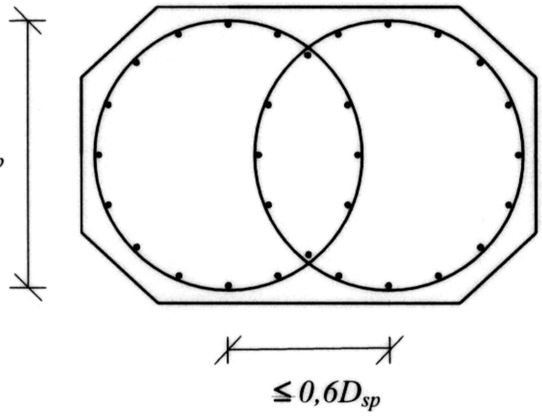


Figure 6.1b: Typical confinement detail in concrete piers using interlocking spirals/hoops

6.2.1.5 Extent of confinement - Length of potential plastic hinges

(1)P When $\eta_k = N_{Ed}/A_c f_{ck} \leq 0,3$ the design length L_h of potential plastic hinges shall be estimated as the largest of the following values:

– the depth of the pier section within the plane of bending (perpendicular to the axis of rotation of the hinge);

– the distance from the point of maximum moment to the point where the design moment is less than 80% of the value of the maximum moment.

(2)P When $0,6 \geq \eta_k > 0,3$ the design length of the potential plastic hinges as determined in **(1)**P shall be increased by 50%.

(3) The design length of plastic hinges (L_h) defined above should be used exclusively for detailing the reinforcement of the plastic hinge. It should not be used for estimating the plastic hinge rotation.

(4)P When confining reinforcement is required, the amount specified in **6.2.1.4** shall be provided over the entire length of the plastic hinge. Outside the length of the hinge the transverse reinforcement may be gradually reduced to the amount required by other criteria. The amount of transverse reinforcement provided over an additional length L_h adjacent to the theoretical end of the plastic hinge shall not be less than 50% of the amount of the confining reinforcement required in the plastic hinge.

6.2.2 Buckling of longitudinal compression reinforcement

(1)P Buckling of longitudinal reinforcement shall be avoided along potential hinge areas, even after several cycles into the post-yield region.

(2) To meet the requirement in **(1)**P, all main longitudinal bars should be restrained against outward buckling by transverse reinforcement (hoops or cross-ties) perpendicular to the longitudinal bars at a (longitudinal) spacing s_L not exceeding δd_{bL},

where d_{bL} is the diameter of the longitudinal bars. Coefficient δ depends on the ratio f_t/f_y of the tensile strength f_{tk} to the yield strength f_{yk} of the transverse reinforcement, in terms of characteristic values, in accordance with the following relation:

$$5 \leq \delta = 2{,}5\ (f_{tk}/f_{yk}) + 2{,}25 \ \leq\ 6 \qquad (6.9)$$

(3) Along straight section boundaries, restraining of longitudinal bars should be acheived in either one of the following ways:

a) through a perimeter tie engaged by intermediate cross-ties at alternate locations of longitudinal bars, at transverse (horizontal) spacing s_t not exceeding 200 mm. The cross-ties shall have 135°-hooks at one end and 135°-hooks or 90°-hook at the other. Cross-ties with 135°-hooks at both ends may consist of two lapped spliced pieces. If η_k > 0,30, 90°-hooks are not allowed for the cross-ties. If the cross-ties have dissimilar hooks at the two ends, these hooks should be alternated in adjacent cross-ties, both horizontally and vertically. In sections of large dimensions the perimeter tie may be spliced using appropriate lapping length combined with hooks;

b) through overlapping closed ties arranged so that every corner bar and at least every alternate internal longitudinal bar is engaged by a tie leg. The transverse (horizontal) spacing s_T of the tie legs should not exceed 200 mm.

(4)P The minimum amount of transverse ties shall be determined as follows:

$$\min\left(\frac{A_t}{s_T}\right) = \frac{\Sigma A_s f_{ys}}{1{,}6 f_{yt}} (mm^2/m) \qquad (6.10)$$

where:

A_t is the area of one tie leg, in mm²;

s_T is the transverse distance between tie legs, in m;

ΣA_s is the sum of the areas of the longitudinal bars restrained by the tie, in mm²;

f_{yt} is the yield strength of the tie; and

f_{ys} is the yield strength of the longitudinal reinforcement.

6.2.3 Other rules

(1)P Due to the potential loss of concrete cover in the plastic hinge region, the confining reinforcement shall be anchored by 135°-hooks (unless a 90°-hook is used in accordance with **6.2.2(3)**a) surrounding a longitudinal bar plus adequate extension (min. 10 diameters) into the core concrete.

(2)P Similar anchoring or a full strength weld is required for the lapping of spirals or hoops within potential plastic hinge regions. In this case laps of successive spirals or hoops, when located along the perimeter of the member, should be staggered in accordance with EN 1992-1-1:2004, **8.7.2**.

(3)P No splicing by lapping or welding of longitudinal reinforcement is allowed within the plastic hinge region. For mechanical couplers see EN 1998-1:2004, **5.6.3(2)**.

6.2.4 Hollow piers

(1) The rules of **(2)** to **(4)** are not required in cases of low seismicity.

NOTE: For cases of low seismicity the Notes in **2.3.7(1)** apply.

(2) Unless appropriate justification is provided, the ratio b/h of the clear width b to the thickness h of the walls, in the plastic hinge region (length L_h in accordance with **6.2.1.5**) of hollow piers with a single or multiple box cross-section, should not exceed 8.

(3) For hollow cylindrical piers the limitation **(2)** applies to the ratio D_i/h, where D_i is the inside diameter.

(4) In piers with simple or multiple box section and when the value of the ratio η_k defined in expression (6.1) does not exceed 0,20, there is no need for verification of the confining reinforcement in accordance with **6.2.1**, provided that the requirements of **6.2.2** are met.

6.3 Steel piers

(1)P For bridges designed for ductile behaviour, the detailing rules of EN 1998-1:2004, **6.5**, **6.6**, **6.7** and **6.8**, as modified by **5.7** of the present Part, shall be applied.

6.4 Foundations

6.4.1 Spread foundation

(1)P Spread foundations such as footings, rafts, box-type caissons, piers etc., shall not enter the plastic range under the design seismic action, and hence do not require special detailing reinforcement.

6.4.2 Pile foundations

(1)P When it is not feasible to avoid localised hinging in the piles, using the capacity design procedure (see **5.3**), pile integrity and ductile behaviour shall be ensured. For this case following rules apply.

(2) The following locations along the pile should be detailed as potential plastic hinges.

(a) At the pile heads adjacent to the pile cap, when the rotation of the pile cap about a horizontal axis transverse to the seismic action is restrained by the large stiffness of the pile group in this degree-of-freedom.

(b) At the depth where the maximum bending moment develops in the pile. This depth should be estimated by an analysis that takes into account the effective pile flexural stiffness (see **2.3.6.1**), the lateral soil stiffness and the rotational stiffness of the pile group at the pile cap.

(c) At the interfaces of soil layers with markedly different shear deformability, due to kinematic pile-soil interaction (see EN 1998-5:2004, **5.4.2(1)**P).