TITLE 6 STRUCTURAL MEMBERS

CHAPTER 12

STRUCTURAL MEMBERS

Article 52. Structural plain concrete members

52.1 Scope

Structural plain concrete members are those members constructed using concrete that does not contain any reinforcement or which contains reinforcement only to minimise the effects of cracking and which is generally in the form of mesh near to faces.

This chapter does not apply to plain concrete structural elements that have their own special standards, other than in a subsidiary manner.

52.2 Concretes that may be used

The concretes defined in 39.2 may be used for plain concrete members.

52.3 Design loads

The combined design loads applicable at Ultimate Limit States are as indicated in Article 13.

52.4 Design of sections under compression

In a section of a plain concrete member acted upon only by a perpendicular compression force a design value of N_d (positive), applied at a point G, with eccentricity components (e_x, e_y) relative to a system of centroidal axes a component eccentricity of (e_x, e_y), relative to a system of cobaricentric axes, (case a; figure 52.4.a), N_d shall be considered to be applied at virtual point $G_1(e_{1x}, e_{1y})$, which will be the point which is the least favourable from the following two points:

$$G_{1x}\ (e_x + e_{xa},\ e_y) \text{ or } G_{1y}\ (e_x,\ e_y + e_{ya})$$

in which:

h_x and h_y	Maximum dimensions in these directions.
e_{xa}	$=0,05h_x \geq 2$cm
e_{ya}	$=0.05h_y \geq 2$cm

The resultant stress σ_d is calculated by assuming a uniform distribution of stresses in one part of the section, called its effective section, of area A_e (case b; figure 52.4.a), defined by a secant and whose baricentre coincides with the virtual application point G_1 of the perpendicular force and assuming the rest of the section to be inactive.

The safety condition is given by:

$$\frac{N_d}{A_e} \leq 0{,}85\, f_{cd}$$

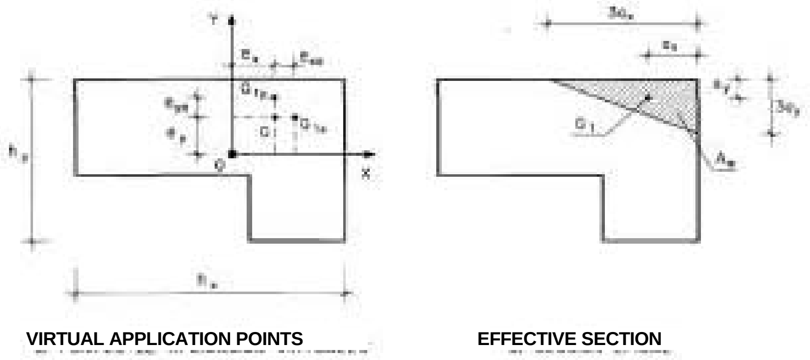


Figure 52.4.a

52.5 Design of sections under compression and shear stress

In a section of a plain concrete member, acted upon by a diagonal compression force with components with a design value of N_d and V_d (positive) applied at the point G, the virtual application point G1, and the effective area A_e, shall be determined as in 52.4. The safety conditions are as follows:

$$\frac{N_d}{A_e} \leq 0{,}85\, f_{cd} \qquad \frac{V_d}{A_e} \leq f_{ct,d}$$

52.6 Consideration of slenderness

In a plain concrete member subjected to compression, either with or without any shear stress, the first order effects produced by N_d have to be increased by second order effects, due to its slenderness (52.6.3). In order to take account of these, N_d shall be considered to be acting on a point G_2 which results from displacing G_1 (52.4) by a fictitious eccentricity defined in 52.6.4.

52.6.1 Virtual width

$b_v=2c$ shall be adopted for the virtual width b_v, of the section of a member, with c being the minimum distance between the centroid of the section (figure 52.6.1) and a straight line meeting tangent to the perimeter.

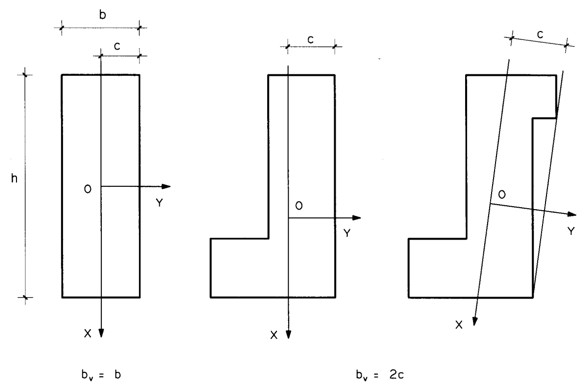


Figure 52.6.1

52.6.2 Buckling length

$l_o = \beta l$, shall be taken as the buckling length, l_o of a member, with l being the height of the member between its base and top, and $\beta = \beta_o \xi$ being the slenderness factor, with $\beta_o = 1$ in members with a horizontally braced top and $\beta_o = 2$ in members without any brace at the top.
The factor ζ takes account of the effect of the bracing by transverse walls and is:

$$\zeta = \sqrt{\frac{s}{4\,l}} \leq 1$$

in which:
s Distance between bracing walls.

In columns and other exempt elements $\zeta = 1$ shall be adopted.

52.6.3 Slenderness

The slenderness λ of a plain concrete member shall be determined using the following expression:

$$\lambda = \frac{l_o}{b_v}$$

52.6.4 Fictitious eccentricity

The buckling effect of a member with a slenderness, λ, shall be considered equivalent to the buckling caused by the addition of fictitious eccentricity, e_a in a direction of the axis, and parallel to the virtual width b_v of the section, with a value of:

$$e_a = \frac{15}{E_c}(b_v + e_1)\,\lambda^2$$

in which:
E_c Instantaneous secant modulus deformation of the concrete in N/mm^2 at 28 days (39.6).
e_1 Determinant eccentricity (figure 52.6.4), which is valid for:

- Members with horizontally braced tops: the maximum value of e_{1y} on the z_0.abscissa.

$$\frac{l}{3} \leq z_o \leq \frac{2\,l}{3}.$$

- Members whose tops are not braced: the value of e_{1y} at its base.

The member shall be calculated at the abscissa z_o with component eccentricity (e_{1x}, $e_1 + e_a$) and at each end with its corresponding eccentricity (e_{1x}, e_{1v}).

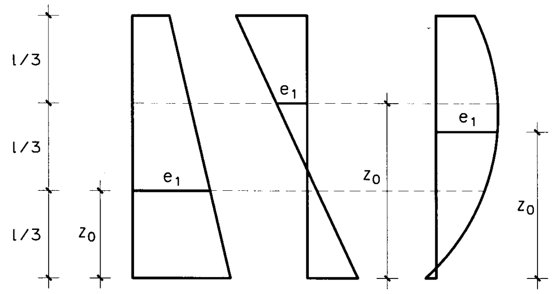


Figure 52.6.4

Article 53. Beams

Beams subjected to bending shall be designed in accordance with Article 42 or the simplified formulae in Annexe No. 7, based on the design values of the material strengths (Article 15) and the factored values for combined loads (Article 13). If bending is combined with shear force, the member shall be designed for the latter, in accordance with Article 44, and Article 45 if torsion is also present. The Longitudinal Shear Limit State shall be verified in composite members (Article 47).

The Cracking Deformation and Vibration Limit States shall also be verified when necessary, in accordance with Articles 49, 50 and 51 respectively.

Sub-paragraph 18.2.1 shall be taken into account in the case of T-sections and special shaped sections.

The configuration of reinforcements shall comply with the requirements in Article 69, in the case of passive reinforcements, and 70, in the case of active reinforcements.

Article 54. Supports

Supports shall be designed to withstand normal forces in accordance with Article 42 or the simplified formulae in Annexe No. 7, based on the design values for material strengths, (Article 15) and the factored values for combined loads (Article 13). If there is appreciable support slenderness, the Instability Limit State shall be verified Article 43). If there is shear force, the member shall be designed to withstand this in accordance with Article 44 and in accordance with 45 if torsion is also present.

The Cracking Limit State shall also be verified if necessary in accordance with Article 49.

The smallest dimension of in situ cast supports shall be at least 25 cm.

The configuration of reinforcements shall comply with the requirements in Article 69, in the case of passive reinforcements, and 70, in the case of active reinforcements.

The main reinforcement shall comprise at least four bars, in the case of rectangular sections, and six bars in the case of circular sections, with the gap between two consecutive bars being no more than 35 cm. The diameter of the thinnest compressed bar shall be at least 12mm. In addition, these bars shall be secured using hoops or stirrups, with the maximum gaps and minimum diameters for the transverse reinforcement indicated in 42.3.1.

Stirrups may be either circular or helical in form in circular supports.

Article 55. Two-way slabs or plates

55.1 Two-way slabs, including flat slabs, on continuous supports

This Article covers two-way reinforced and pre-stressed slabs, including flat slabs, continuous supports.

Unless evidence is supplied to the contrary, the total depth of the slab shall be at least *L/40* or 8 cm, with *l* being the span of the smallest bay.

The indications in Article 22 shall be followed as regards structural analysis.

The various combinations of design loads shall be examined when verifying the various Limit States in accordance with the criteria set out in Article 13.

The Ultimate Limit State of Failure due to perpendicular stresses shall be verified in accordance with Article 42, taking into consideration an equivalent bending stress that

accounts for the effect produced by the bending and torsional moments present at each point within the slab.

The Shear Limit State shall be verified in accordance with the indications in Article 44.

Whenever necessary, the Cracking, Deformation and Vibration Limit States shall also be verified in accordance with Articles 49, 50 and 51, respectively.

The configuration of reinforcements shall comply with the requirements in Article 69, in the case of passive reinforcements, and 70, in the case of active reinforcements.

A transverse reinforcement parallel to the design direction of the supports shall always be incorporated in rectangular slabs supported on two sides, and designed to resist a moment of 20% of the main moment.

55.2 Two-way slabs, including flat slabs, on isolated supports

This article is applicable to structures consisting of reinforced concrete slabs that are either solid or hollow, with ribs in two perpendicular directions, that generally do not have beams to transmit the loads to the supports, resting directly on columns with or without capitals.

Unless specially justified, the total depth of reinforced concrete slabs shall not be less than the following values:

- Solid slabs of constant thickness, *L*/32.
- Hollow slabs of constant thickness, *L/28*

With *L* being the larger dimension of the frame.

The distance between the centre lines of ribs shall not exceed 100 cm, and the thickness of the top cover shall not be less than 5 cm, which shall incorporate mesh distribution reinforcement.

The indications in Article 22 shall be followed when undertaking structural analysis.

The various combinations of weighted loads shall be examined when verifying the various limit states, in accordance with the criteria set out in Article 13.

The Ultimate Limit State of Failure due to perpendicular stresses shall be verified in accordance with Article 42, taking into consideration an equivalent bending stress that accounts for the effect produced by the bending and torsional moments present at each point within the slab.

The Limit State of Failure due to shear stresses shall be verified in accordance with the indications in Article 44. In particular, the ribs where they meet their drop panels, and edge elements, beams and transverse reinforcements shall be verified.

The Limit State of Failure due to torsion shall be verified in edge beams and transverse reinforcements in accordance with the indications in Article 45.

The Punching Limit State shall be verified in accordance with indications in del Article 46.

Whenever necessary, the Cracking, Deformation and Vibration Limit States shall also be verified in accordance with Articles 49, 50 and 51, respectively.

The configuration of reinforcements shall comply with the requirements in Article 69, in the case of passive reinforcements, and 70, in the case of active reinforcements.

Article 56. Shells

Unless supporting evidence can be provided to the contrary, shells with concrete thicknesses less than the following shall not be built:

- Folded shells: 9 cm.
- Single curvature shells: 7 cm.
- Double curvature shells: 5 cm.

Unless special evidence is provided, the following provisions shall be satisfied:

a) The reinforcements in the thin slab shall be configured in a rigorously symmetrical manner with regard to its average surface.

b) The mechanical ratio in any section of the thin slab shall satisfy the following limitation:

$$\omega \leq 0,30 + \frac{5}{f_{cd}}$$

in which f_{cd} is the design compressive strength of the concrete, expressed in N/mm^2.

c) The main reinforcements shall not be more than the following distances apart:

- Three times the thickness of the shell if a mesh is placed on its average surface.
- Five times the thickness of the shell, if a mesh is placed close to both faces.

d) The coverings of the reinforcements shall satisfy the general conditions set out in 37.2.4.

The indications in Article 23, shall be followed when conducting a structural analysis of the shells.

The various combinations of factored actions in accordance with the criteria set out in article 13 shall be examined when verifying the various Limit States.

The Ultimate Limit State due to perpendicular stresses shall be verified in accordance with Article 42, bearing in mind the axial stress and biaxial bending stress at each point in the shell.

The Limit State for Shear Stress shall be verified in accordance with the indications in Article 44.

The Punching Limit State shall be verified in accordance with the indications in Article 46.

The Cracking Limit State shall also be verified in accordance with Article 49 whenever necessary.

The reinforcements shall be configured in accordance with the requirements in Article 69, in the case of passive reinforcements, and Article 70 in the case of active reinforcements.

Article 57. Walls

Walls subjected to bending shall be designed in accordance with Article 42 or the simplified formulae in Annexe No. 7, based on the design values for the strength of their constituent materials and the design values of the combined actions (Article 13). If bending is combined with shear, the member shall be designed to withstand this stress in accordance with Article 44.

The Cracking Limit State shall also be verified in accordance with Article 49.

The configuration of reinforcements shall comply with the requirements in Article 69, in the case of passive reinforcements, and 70, in the case of active reinforcements.

Article 58. Foundation elements

58.1 General

The provisions in this Article shall directly apply to footings and pile caps forming the foundations for isolated or linear supports, although their general principles may be applied to combined foundation members.

This Article also covers the case of continuous foundation members for various supports (foundation slabs).

It finally includes the connecting beams, piles and plain concrete footings.

58.2 Classification of structural concrete foundations

Pile caps and foundation footings may be classified as stiff or flexible.

58.2.1 Stiff foundations

Stiff foundations include the following:

- Pile caps whose offset *v* in the main offset direction is less than 2h. (figure 58.2.1.a).
- Footings whose offset *v* in the main offset direction is less than 2h. (figure 58.2.1.b).
- Bored piles.
- Massive foundation members: counterweights, massive gravity walls, etc.

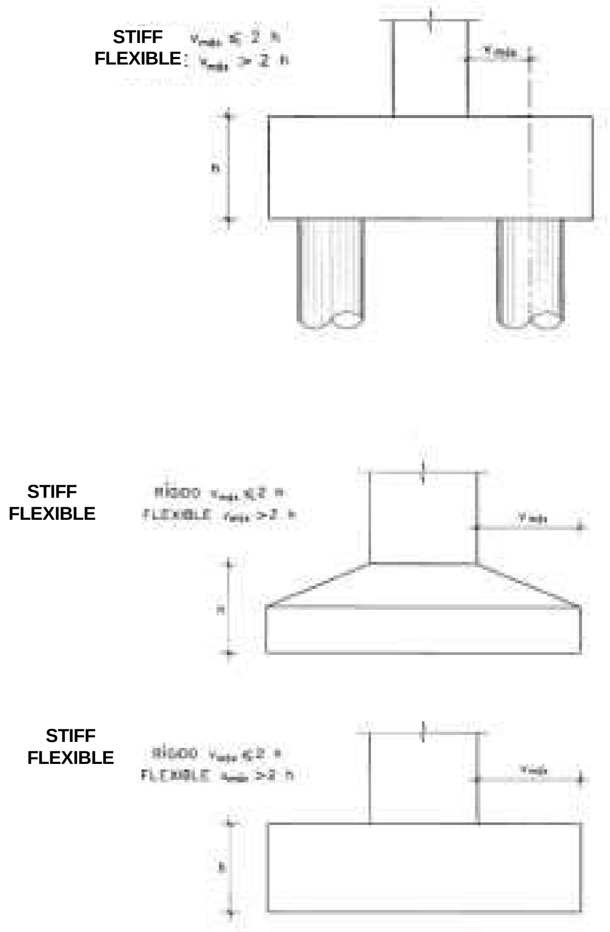


Figure 58.2.1.a

In rigid foundations, the strain distribution is not linear at a section level, the most suitable general analysis method is therefore the strut-and-tie method contained in Articles 24 and 40.

58.2.2 Flexible foundations

Flexible foundations include the following:

- Pile caps whose offset *v* in the main offset direction is more than 2h. (figure 58.2.1.a).
- Footings whose offset *v* in the main offset direction is more than 2h. (figure 58.2.1.b).
- Raft foundations.

In flexible foundations, the distribution of strains may be considered linear on a section level, with the general theory of bending being applicable

58.3 General design criteria

Foundation members shall be designed to resist acting loads and induced reactions. The stresses acting on the foundation element are therefore required to be fully transmitted to the ground or to the piles on which they are resting.

When defining the dimensions of the foundation and verifying the ground stresses or the reactions of piles, the worst combinations transmitted by the structure shall be considered, taking into account the second order effects in the case of slender supports, the dead weight of the foundation member, and that of the ground acting on it, with their characteristic values being adopted in every case.

When verifying the various Ultimate Limit States for the foundation element, the effects of the stresses in the ground and reactions of the piles obtained for the stresses transmitted through the structure for the worst design combinations, shall be considered bearing in mind the second order effects in the case of slender supports, the design value of the dead weight of the foundation element whenever necessary, and that of the ground acting upon it.

58.4 Verifying of elements and reinforcement dimensioning

58.4.1 Rigid foundations

The general bending theory does not apply in this type of member and a strut and tie model needs to be defined in accordance with the criteria indicated in Article 24; the reinforcements needs to be dimensioned and the conditions in the concrete need to be verified in accordance with the requirements set out in Article 40.

A model must be established in each case, enabling the equilibrium between the external loads transmitted by the structure, the loads due to the ground overburden on the footings, pile caps, etc., and the soil stresses or pile reactions.

58.4.1.1 Rigid footings

The model shown in figure 58.4.1.1.a, shall be used in rectangular footings subjected to straight bending-compression provided that the effect of the weight of the footing and the ground above it can be disregarded.

The main reinforcement shall be designed to resist the tensile force Td indicated in the model, which is obtained from:

$$T_d = \frac{R_{1d}}{0,85\,d}\,(\,x_1 - 0,25\,a\,) = A_s\,f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2), in which R_{1d} is the sum of the stresses in the shaded trapezium in the width of the footing, and x_1 is the distance between the centre of gravity of the trapezium and the load line of N_{1d} and with the meaning of the rest of the variables being as shown in figure 58.4.1.1.a and the stresses σ_{1d} and σ_{2d} being those obtained considering only the loads transmitted by the structure. This reinforcement shall be incorporated without any reduction of section, along the entire length of the footing, and anchored in accordance with the criteria set out in Article 69. The welded transverse bar anchorage type is particularly recommended in this case.

The strength of the nodes in the model does not generally need to be verified if the characteristic strength of the concrete in the piles is the same as the characteristic strength of the concrete in the footing. In all other cases, the verification indicated in paragraph 40.4 shall be carried out.

However, verifying the nodes implicitly involves verifying the struts.

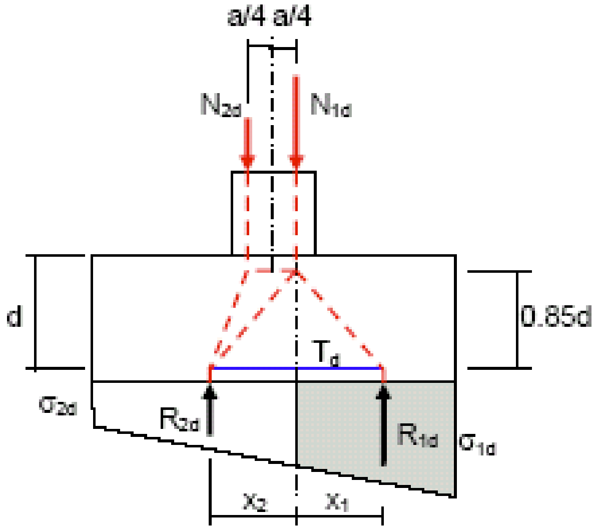


Figure 58.4.1.1.a

58.4.1.2 Rigid pile caps

The reinforcement necessary shall be determined on the basis of the tensions in the ties in the model adopted for each pile cap. For the most common cases, the various models and the expressions enabling the reinforcements to be determined are indicated in the following paragraphs.

Verifying the strength of the concrete in nodes is not generally required of in situ cast piles and if these and the columns are made from a concrete with a characteristic strength that is the same as the characteristic strength of the concrete in the pile cap. In other cases, the verification in paragraph 40.4 will have to be carried out.

However, the verifying the nodes implicitly involves verifying the struts.

58.4.1.2.1 Pile caps on two piles

4.1.2.1.1 Main reinforcement

The reinforcement shall be designed to resist the design tension, T_d in figure 58.4.1.2.1.1.a, which may be taken to be:

$$T_d = \frac{N_d\,(\,v + 0{,}25\,a\,)}{0{,}85\,d} = A_s\,f_{yd}$$

with $f_{yd} \leq$ 400 N/mm^2 (40.2) and in which N_d corresponds to the design axial load of the most loaded pile.

The lower reinforcement shall be incorporated without any reduction in its section, along the entire length of the pile cap. This reinforcement shall be anchored in a straight line or at right angles using welded transverse bars, starting from vertical planes which pass through the axis of each pile (figure 58.4.1.2.1.1.b).

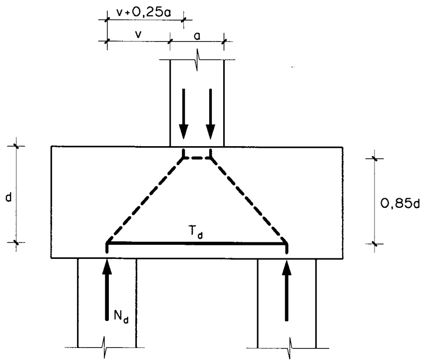


Figure 58.4.1.2.1.1.a

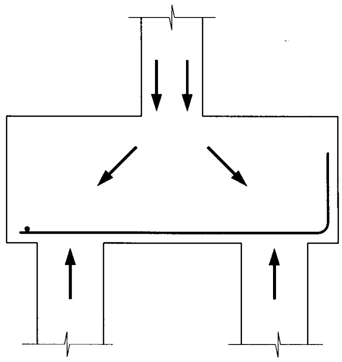


Figure 58.4.1.2.1.1.b

58.4.1.2.1.2 Secondary reinforcement

The secondary reinforcement of pile caps on two piles shall comprise:

- A longitudinal reinforcement arranged in the upper face of the pile cap and which extends without any steps along its entire length. Its mechanical strength shall not be less than 1/10 of the mechanical strength of the lower reinforcement.
- A horizontal reinforcement and a vertical reinforcement arranged in a grid configuration in the side faces. The vertical reinforcement shall comprise closed hoops which tie the upper longitudinal reinforcement to the lower longitudinal reinforcement. The horizontal reinforcement shall comprise closed hoops that tie the aforementioned vertical reinforcement (figure 58.4.1.2.1.2.a). The ratio of these reinforcements with reference to the cross-sectional concrete area perpendicular to its direction shall be at least 4%. If the width exceeds half the thickness, the reference cross-section shall be taken to be a width that is half that of the depth.

With a high concentration of reinforcement it is also recommended that the vertical hoops described in this paragraph are brought closer together in the anchorage area of the main reinforcement, in order to ensure that main reinforcement is well bound together in the anchorage zone. (figure 58.4.1.2.1.2.b).

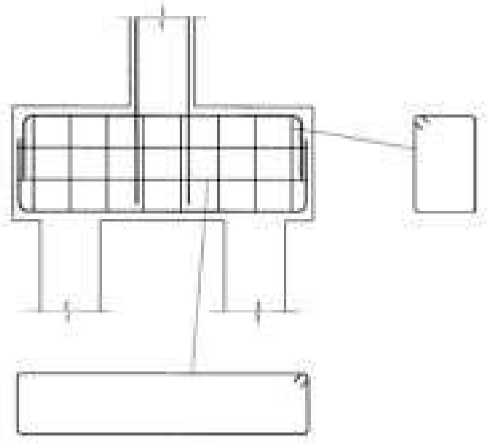


Figure 58.4.1.2.1.2.a

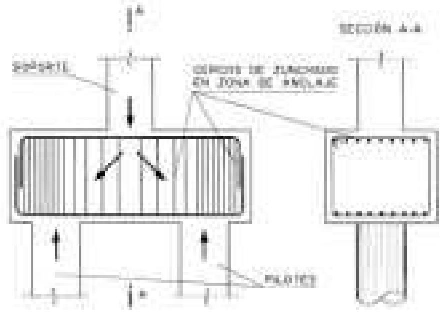


Figure 58.4.1.2.1.2.b

58.4.1.2.2 Pile caps on several piles

The reinforcement for pile caps on several piles may be classified as:

- Main reinforcement

  Located in strips over the piles (see figure 58.4.1.2.2.a). A band or strip is defined as a zone whose centre line is the line which joint the centres of the piles and whose width is the same as the diameter of the pile plus twice the distance between the top face of the pile and the centre of gravity of the reinforcement of the tie reinforcement. (see figure 58.4.1.2.2.b).

- Secondary reinforcement:

  Located between the strips (see 58.4.1.2.2.1.a)

- Secondary vertical reinforcement:

  Located in the form of hoops, tying in the main strip reinforcement (see 58.4.1.2.2.2.b)

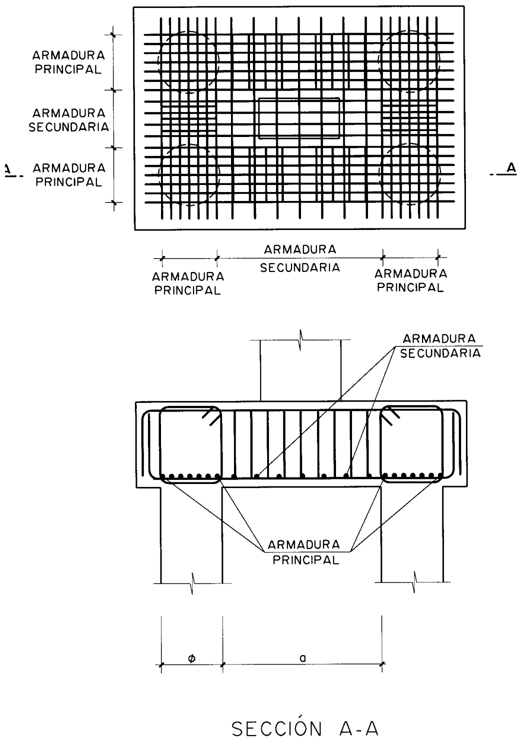


Figure 58.4.1.2.2.a

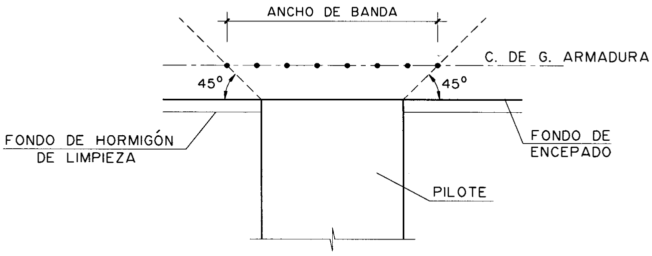


Figure 58.4.1.2.2.b

58.4.1.2.2.1 Main reinforcement and horizontal secondary reinforcement

The lower main reinforcement shall be fitted in bands or strips on piles. This reinforcement shall be arranged so that it is anchored from a vertical plane which passes through the centre line of each pile.

A secondary reinforcement in a grid configuration shall also be fitted, whose mechanical strength in each direction shall not be less than 1/4 of the mechanical strength of the strips or bands.

The main reinforcement between each set of piles in pile caps on top of three piles fitted along the vertices of an equilateral triangle, with the column located in the centroid of the triangle, may be obtained from the tension force T_d obtained from the following expression:

$$T_d = 0{,}68 \frac{N_d}{d} (\, 0{,}58\, l - 0{,}25\, a \,) = A_s\, f_{yd}$$

with f_{yd} < 400 N/mm^2 (40.2) and in which:

N_d Design axial stress of the most loaded pile (figure 58.4.1.2.2.1 .a).

d Effective depth of the pile cap (figure 58.4.1.2.2.1.a).

The tension force corresponding to each strip in pile caps on four piles with the column located at the centre of the rectangle or square may be obtained from the following expressions:

$$T_{1d} = \frac{N_d}{0,85\,d}\left(0,50\,l_1 - 0,25\,a_1 \right) = A_s\,f_{yd}$$

$$T_{2d} = \frac{N_d}{0,85\,d}\left(0,50\,l_2 - 0,25\,a_2 \right) = A_s\,f_{yd}$$

with f_{yd} < 400 N/mm^2 and in which:

N_d Design axial stress of the most loaded pile (figure 58.4.1.2.2.1 .a).
d Effective depth of the pile cap (figure 58.4.1.2.2.1.a).

The main reinforcement in continuous foundations on a linear pile cap shall be located perpendicular to the wall, calculated using the expression in sub-paragraph 58.4.1.2.1, whereas in the direction parallel to the wall, the pile cap and the wall shall be designed as a beam (which is usually a deep beam) supported on the piles (figure 58.4.1.2.2.1.c).

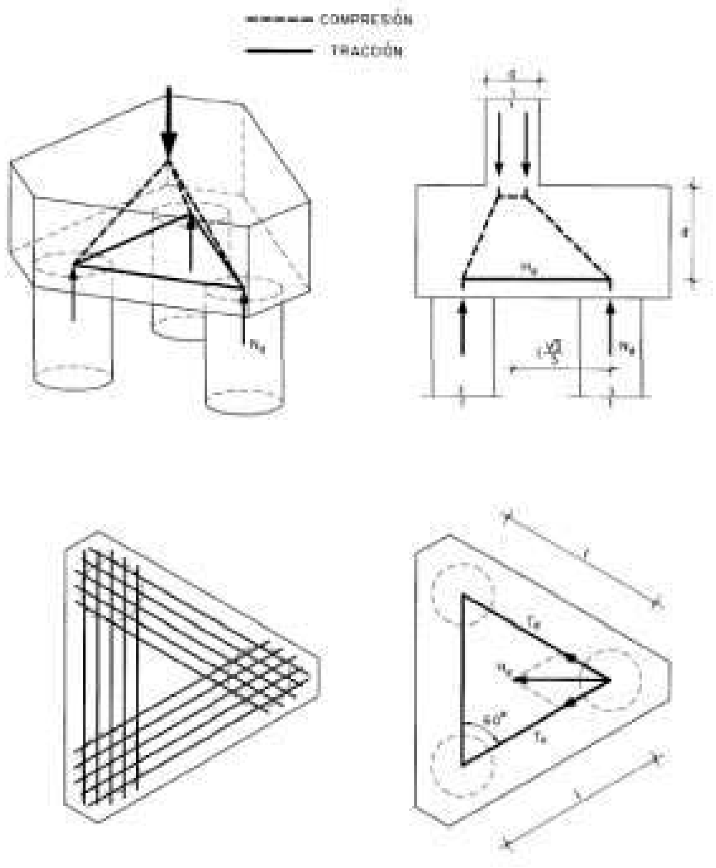
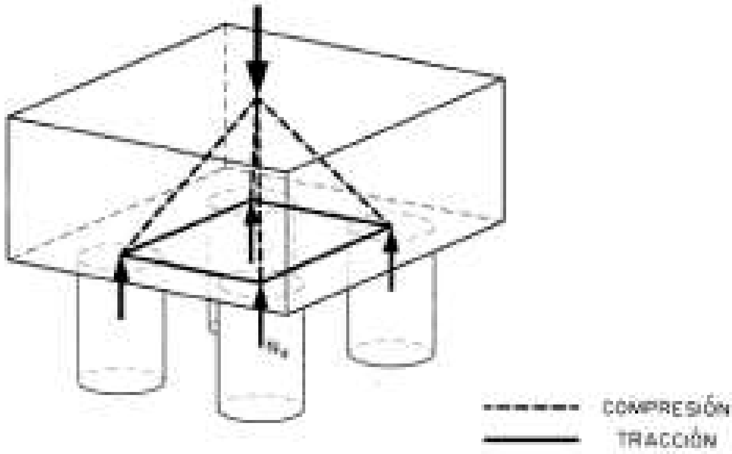


Figure 58.4.1.2.2.1.a



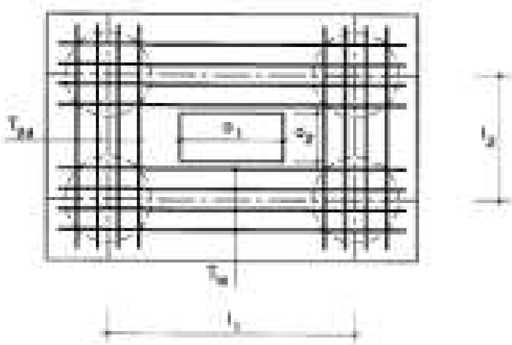


Figure 58.4.1.2.2.1.b

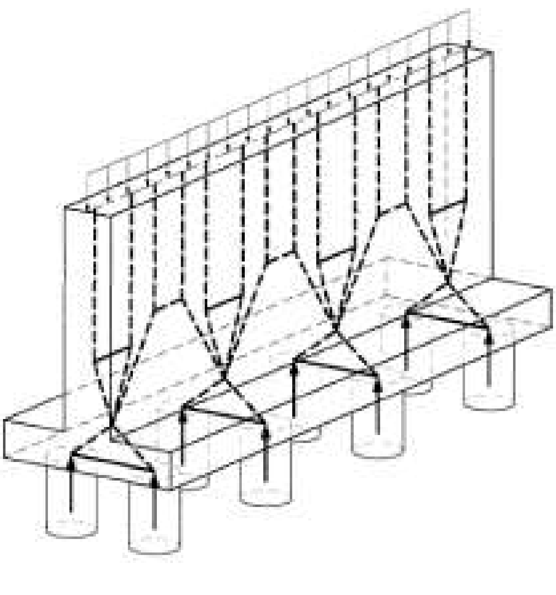


Figure 58.4.1.2.2.1.c

58.4.1.2.2.2 Vertical secondary reinforcement

Vertical secondary reinforcement shall be provided to stresses due to the dispersion of the compression field. figure 58.4.1.2.2.2, which shall have a total mechanical strength of not less than the value of N_d / 1.5n, with n≥3, being:

N_d *n* Design value of the axial force for the support.
N Number of piles

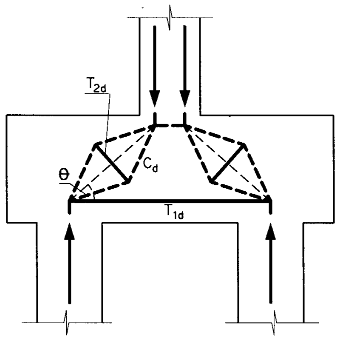


Figure 58.4.1.2.2.2

58.4.2 Flexible foundations

The general bending theory shall apply in this type of foundations.

58.4.2.1 Flexible footings and pile caps

Unless an accurate soil-foundation interaction study is undertaken, the simplified criteria described below may be used.

58.4.2.1.1 Bending analysis

The reference cross-section to be considered for bending analysis, is defined as follows: it is plane, perpendicular to the base of the footing or pile capping and takes account of the total cross-section of the footing or pile cap. It is parallel to the face of the support or the wall and is located behind this face and 0.15a away, with *a* being the dimension of the support or the wall measured orthogonally to the section considered.

The effective depth of this reference section shall be taken as being the effective depth of the section parallel to section S_1, located in the face of the support or the wall (figure 58.4.2.1.1.a).

All the foregoing assumes that the support and wall are concrete members. If this is not the case the quantity 0.15a shall be replaced by:

- 0.25a, in the case of brick or masonry
- Half the distance between the face of the support and the edge of the steel plate in the case of metal supports on top of steel load distribution plates.

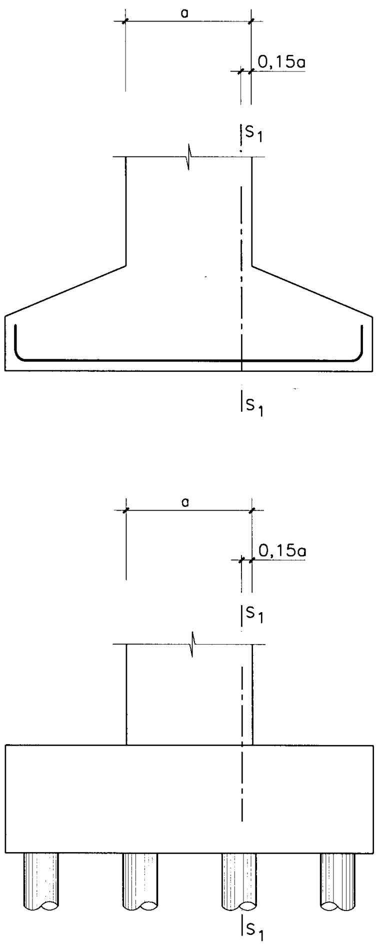


Figure 58.4.2.1.1.a

The maximum moment to be considered when analysing flexible footings and pile caps is the moment produced in the reference section, S_1 defined in the paragraph above (figure 58.4.2.1.1.b).

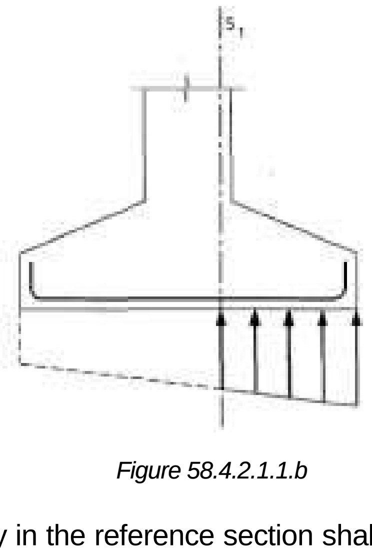


Figure 58.4.2.1.1.b

The reinforcement necessary in the reference section shall be determined using pure bending analysis in accordance with general design principles for sections subjected to the perpendicular stresses indicated in Article 42.

In flexible footings and pile caps, working in a single direction and in square foundation elements operating in two directions, the reinforcement may be uniformly distributed throughout the entire width of the foundation.

In rectangular foundation elements working in two directions, the reinforcement parallel to the larger side of the foundation, of length *a',* may be distributed uniformly across the entire width, *b'* of the foundation base. The reinforcement parallel to the smaller side *b'* shall be arranged so that a fraction of the total area A_s of *2b'/(a'+b')* is uniformly distributed in a central strip that is coaxial with the support, and has a width *b'*. The rest of the reinforcement shall be distributed uniformly in the two resulting side strips.

This width of the strip, *b'* shall not be less than *a+2h*, in which:

a The side of the support or wall parallel to the larger side of the foundation base.
h Total depth of the foundation.

If *b'* is less than *a+2h, b'* shall be replaced by *a+2h* (figure 58.4.2.1.1.c).

The reinforcement designed shall be anchored in accordance with the less favourable of the following two criteria:

- The reinforcement shall be anchored in accordance with the conditions in Article 69, of cross-section S_2 located on an effective depth of the reference section S_1.
- The reinforcement shall be anchored beyond section S_3 (figure 58.4.2.1.1.d) for a force:

$$T_d = R_d \frac{v + 0{,}15\,a - 0{,}25\,h}{0{,}85\,h}$$

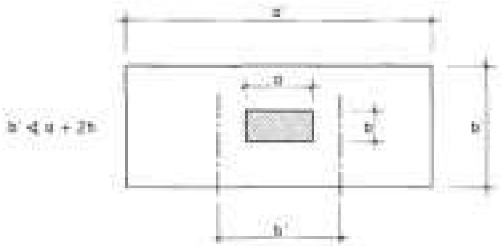


Figure 58.4.2.1.1.c

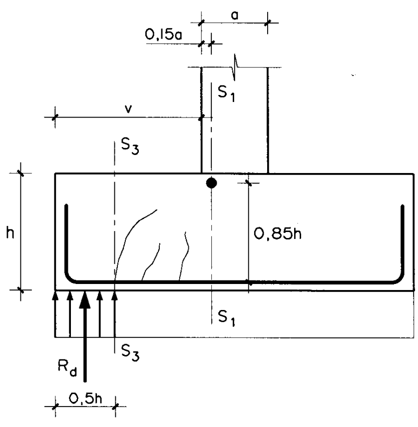


Figure 58.4.2.1.1.d

58.4.2.1.2 Design for tangential stresses

The resistance to tangential stresses in flexible footings and pile caps, near to concentrated loads or reactions (at supports and piles) shall be verified for shear as a linear element and for punching shear.

The footing or pile cap shall be verified in shear in accordance with the provisions in Article 44, in the reference section S_2.

The reference section S_2 shall be located a distance that is equal to the effective width away from the face of the support, wall or pedestal, or the midpoint between the face of the support and the edge of the steel plate, in the case of metal supports on top of steel distribution plates. This reference section shall be plane, perpendicular to the base of the footing or pile cap and take account of the overall section of the aforementioned foundation element.

The Limit State Shear Punching shall be verified in accordance with Article 46°.

58.4.2.1.3 Cracking verification

The Cracking Limit State shall be verified in accordance with Article 49 whenever necessary.

58.4.2.2 Raft foundations

]This paragraph relates to reinforced or pre-stressed concrete surface elements (slabs), used for the foundations of various supports.

The models described in Article 22 may be used to obtain the forces.

When verifying the various Limit States, the various combinations of factored actions shall be studied in accordance with the criteria set out in Article 13°.

The Ultimate Limit State for Perpendicular Stresses shall be verified in accordance with Article 42, considering an equivalent bending force which takes account of the effect produced by the bending and torsional moments present at each point in the slab.

The Failure Limit State for Shear shall be verified in accordance with the indications in Article 44.

The Limit State for Punching Shear shall be verified in accordance with the provisions in Article 46.

Similarly, whenever necessary, the Cracking Limit State shall also be verified in accordance with Article 49.

The configuration of reinforcements shall comply with the requirements in Article 69, with regard to passive reinforcements, and 70 with regard to active reinforcements.

58.5 Centring and tie beams

Centring beams are linear elements that may be used to resist construction eccentricities or moments in pile heads where pile caps are used on one or two piles, if these do not have an individual strength to resist these actions, or in offset footings.

Tie beams are linear elements connecting superficial or deep foundations, and are particularly necessary for foundations in earthquake zones.

These elements shall generally satisfy the requirements set out for beams in Article 53.

58.6 Piles

Piles are verified in a similar manner to their support, as indicated in Article 54, in which the soil at least partially prevents buckling.

A minimum eccentricity defined in accordance with tolerances shall always be considered.

When dimensioning in situ cast piles without any pile casing, a design diameter of d_{cal} equal to 0.95 times the pile's nominal diameter d_{nom} shall be used and the following conditions satisfied:

$$d_{nom} - 50\ mm \leq d_{cal} = 0.95\ d_{nom} \leq d_{nom} - 20\ mm$$

58.7 Plain concrete footings

The depth and width of a plain concrete footing resting on the ground, shall be determined so that the design virtual tensile strength values and shear strengths are not exceeded.

The reference section, S_1, considered when undertaking bending analysis is defined as follows:

It is plane, perpendicular to the base of the footing and takes account of the overall cross-section of the footing or pile cap. It is parallel to the face of the support or the wall and is located behind this face at a distance of 0.15a, with *a* being the dimension of the support or the wall measured orthogonally to the section considered. The total depth of this reference section shall be taken as being the total depth of the section parallel to section S_1, located in the face of the support or the wall (figure 58.4.2.1.1.a).

All the foregoing assumes that the support and wall are concrete elements. If this is not the case the quantity 0.15a shall be replaced by:

- 0.25a, in the case of masonry walls
- Half the distance between the face of the support and the edge of the steel plate in the case of metal supports on top of steel distribution load plates.

The reference section to be considered in shear design shall be located a distance away that is equal to the depth, starting from the face of the support, wall, pedestal, or from the midpoint between the face of the column and the edge of the steel plate, in the case of metal supports on steel load distribution plates. This reference section is plane, perpendicular to the base of the footing and takes account of the overall cross-section of this footing.

The reference section to be considered in punching shear design shall be perpendicular to the base of the footing and defined so that it has as small a perimeter as possible and is not located closer to the perimeter of the support, wall or pedestal than half the total depth of the footing.

The factored bending moment and the factored shear in the corresponding reference section shall produce tensile bending and mean tangential shear stresses, whose value shall be lower than the concrete's design virtual bending and shear strengths.

The bending analysis shall be conducted assuming a state of plane stress and strain, and on the assumption that the entire section is whole i.e. that the concrete is not cracked.

The footing shall be verified for shear and punching shear in the reference sections defined above, with its shear strength being defined by the most restrictive condition.

The concrete's design tensile and shear strength shall be taken to be the value $f_{ct,d}$ indicated in Article 52°.

The value of $2f_{ct,d}$ shall be used when verifying punching shear.

58.8 Minimum dimensions and reinforcements for footings, pile caps and raft foundations

58.8.1 Minimum depths and dimensions

The minimum depth at the edges of plain concrete footings shall not be less than 35 cm.

The total minimum depth at the edges of reinforced concrete members shall not be less than 25 cm if they are resting on the ground, or 40 cm in the case of pile caps on top of piles. In addition, in the latter situation, their thickness shall not at any point be less than the diameter of the pile.

The distance between any point in the perimeter of the pile and the external perimeter of the base of the pile cap shall not be less than 25 cm.

58.8.2 Layout of reinforcement

The longitudinal reinforcement shall satisfy the provisions in Article 42. The minimum ratio refers to the total amount of reinforcement in the bottom, upper face, and side walls in the direction concerned.

The reinforcements arranged in the upper, lower and side faces, shall not be more than 30 cm apart.

58.8.3 Minimum vertical reinforcement

Transverse reinforcement does not need to be incorporated in flexible footings or pile caps, provided that this is not required by the design and the concrete is placed without any discontinuities.

If the footing or pile cap essentially behaves as a broad beam designed as a linear element in accordance with 58.4.2.1.2.1, the transverse reinforcement shall satisfy the provisions in Article 44.

If the footing or pile cap basically behaves in two directions and is designed in shear in accordance with 58.4.2.1.2.2, the transverse reinforcement shall satisfy the provisions in Article 46.

Article 59. Structures with precast elements

59.1 Aspects relating to structures comprising precast members in general

59.1.1 General

This article covers several specific aspects applicable to structures partially or fully comprising precast concrete elements.

Given the evolutionary nature of their construction, when designing of precast structures and members, have to be considered in the actions analysis as well as in the limit state verifications:: (1) Temporary situations, (2) provisional and final supports, (3) the connections between the various members.

Temporary situations during the construction of precast structures include the stripping of members, their handling and transport to the stock place, their storage, transport as far as the site, assembly and, finally, their connection.

Any dynamic loads generated during any temporary situation shall be taken into account.

59.1.2 Structural analysis

Structural analysis shall include:

- The change in geometry, the support conditions of each member, and the characteristics of its constituent materials at each stage and the interaction of each member with other elements.
- The influence on the structural system of the behaviour between connections of the elements, and in particular, their strength and deformation.
- The uncertainties in the conditions of the transmission of forces between elements, due to geometric imperfections in members or their positioning, or in their supports.

In earthquake-free regions, the beneficial effect of the inhibited horizontal deformation caused by friction between the member and its support element, may be taken into account, provided that:

- The overall stability of the structure does not depend exclusively on this friction.
- The support system prevents the accumulation of irreversible slippage, caused by asymmetric performance under cyclic loads, as may be the case in thermal cycles in the ends of bi-supported beams.
- No impact load is possible.

The effects of horizontal movement and the integrity of connections shall be taken into account in the design value of the structure's strength.

59.1.3 Connection and support of precast elements

59.1.3.1 Materials

The materials for the connection and support of elements shall be:

- Stable and durable for the structure's service life.
- Physically and chemically compatible.
- Protected from physical and chemical attack.
- Fire resistant in order to ensure the fire resistance of the structure as a whole.

Support means shall have strength and deformation characteristics that concord with those set out in the design.

Metal connections shall withstand corrosion or be provided with corrosion protection, unless they are to be solely exposed to a non-aggressive environment. Protective films may be used if they need to be inspected.

59.1.3.2 Design of connections

The connections shall have to be able to withstand the effects due to the actions considered in the design and capable of satisfying the movements and deformations set out to ensure correct resistant performance of the structure.

Any potential damage to the concrete and the ends of the elements, such as loss of cover, cracking due to splitting etc., shall be avoided. The following aspects shall therefore be taken into consideration:

- Relative movements between elements.
- Imperfections
- Stresses and type of joint
- Ease of execution
- Ease of inspection

Verification of the strength and stiffness of connections shall be based on an analysis supported, if the event of any uncertainties, by tests.

59.1.3.3 Compression connections

In these connections if shear stress is less than 10% of the compression force it can be disregarded.

Support materials, such as mortar, concrete or polymers shall be laid between the faces of elements in contact. The relative movement of their support surfaces shall be prevented during hardening. In exceptional cases, hollow supports may be constructed (without any intercalated materials), provided that their quality and perfect surfaces are guaranteed, and that the mean stresses in the contact surfaces do not exceed 0.3 f_{cd}.

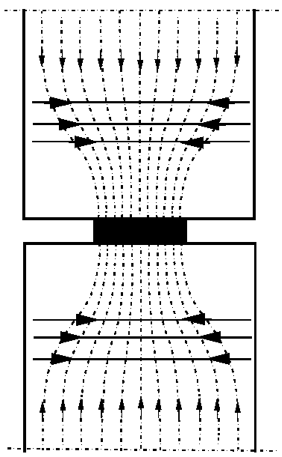


Figure 59.1.3.1. a

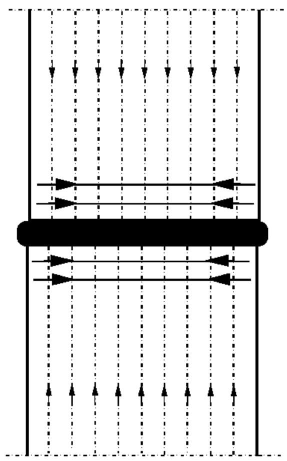


Figure 59.1.3.1 b

The effects of concentrated loads (figure 59.1.3.1.a) and the effects of the expansion of soft materials (figure 59.3.1.1.b) which generate transverse tensile stresses in the concrete, that have to be resisted using reinforcements arranged in an ad hoc manner, shall be taken into consideration in compression supports. The requirements in Article 61 shall be satisfied for the first situation, whereas in the second situation, reinforcement requirements can be calculated using the following expression:

$$A_s=0.25(t/h)N_d/f_{yd}$$

in which:

A_s Section of the reinforcement to be provided in each surface.
t Thickness of the support means comprising soft material.
h Dimension of the support means in the reinforcement direction.
N_d Axial compression force in the support.

59.1.3.4 Shear connections

The requirements in Article 47 shall be adopted to transfer the shear at the interface between two concrete elements, e.g. between a precast element and in situ concrete,

59.1.3.5 Bending and tension connections

The reinforcement shall be continuous through the connection, and anchored on the adjacent element. This continuity may be achieved by:

- Overlapping of bars
- Placing mortar in the sheathes into which the continuous reinforcements are inserted
- Welding of bars or plates
- Pre-stressing
- Other mechanical devices such as nuts and bolts.

59.1.3.6 Halving joints

The requirements set out in paragraph 64.2 shall be taken into consideration when analysing and verifying this type of element.

59.1.3.7 Anchorage of reinforcements on supports

Reinforcements shall be arranged on support elements and supported elements so that they can be anchored, taking into account geometric deviations, as indicated in figure 59.1.3.8.2.b.

59.1.3.8 Considerations for the bearing of precast members

59.1.3.8.1 General

The correct working of bearing devices means shall be ensured using appropriate reinforcement in adjacent elements, limiting the support pressures and adopting measures aimed at allowing or restricting movements.

The actions due to creep, shrinkage, temperature, out of alignment, and being out of plumb, shall be taken into consideration when designing elements in contact with bearings that do not permit any slip or rotation without a significant co-action. The effects of these actions may require the arrangement of transverse reinforcement in support elements and supported elements, or of continuity reinforcement for the tying of these elements. These actions may also affect the design of the main reinforcement of these elements.

Bearings shall be analysed and designed to ensure that they are correctly positioned, taking into account possible deviations or tolerances in their production and assembly.

59.1.3.8.2 Bearings for elements connected to one another (non-isolated)

The equivalent length of a simple support, such as the one in figure 59.1.3.8.2.a, may be calculated as follows:

$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}$$

in which:

a_1 Net length of the support means which is not less than the minimum value in table 59.1.3.8.2.1, which generates a support pressure of σ_{Ed}.

$$\sigma_{Ed} = \frac{N_d}{b_1 \cdot a_1} \leq f_{Rd}$$

in which:

N_d Design value of the force to be resisted in the support.

b_1 Net width of the support *(figure 59.1.3.8.2.a)*

f_{Rd} Design strength of the bearing. In the absence of more accurate specifications, the value of 0.4 f_{cd} may be adopted for the strength of the support in the case of dry supports (without any levelling material), or the strength of the mortar or intermediate levelling element, shall never be less than 85% of the smaller of the design strengths of the concrete in the elements in contact. The provisions in 59.2.3.3. shall be followed in the case of linear supports of surface elements, such as hollow cored slabs.

a_2 Distance considered to be non-effective between the external edge of the support element and the edge of the element in accordance with figure 59.1.3.8.2.a and table 59.1.3.8.2.2.

a_3 Distance equivalent to a_2 in the supported element, in accordance with figure 59.1.3.8.2 and table 59.1.3.8.2.3.

Aa_2 Tolerance on deviations in the distance between bearing elements in accordance with table 59.1.3.8.2.4.

Δa_3 Tolerance in deviations on the length of the supported element, $\Delta a_3 = l_n / 2500$.

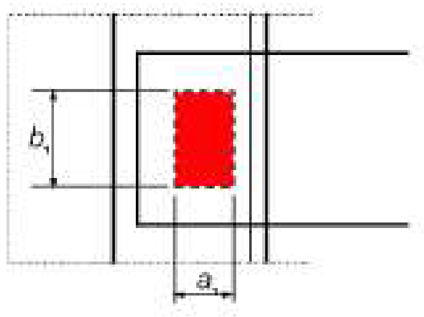


Figure 59.1.3.8.2.a

Table 59.1.3.8.2.1. Minimum values of a_1 in mm.

	Relative stress in the support σ_{Ed} / f_{cd}		
Type of support	≤ 0.15	0.15 – 0.4	> 0.4
Aligned supports (slabs, roofs)	25	30	40
Ribbed slabs, beams and purlins	55	70	80
Concentrated supports (beams)	90	110	140

Table 59.1.3.8.2.2. Values for distance a_2, in mm, which is assumed to be non-effective from the external face of the support element.

Relative stress in the support σ_{Ed} / f_{cd}				
Material and type of support		≤ 0.15	0.15 – 0.4	> 0.4
Steel	linear concentrated	0 5	0 10	10 15
Reinforced concrete $f_{ck} \geq 30$ N/mm^2	linear concentrated	5 10	10 15	15 25
Plain or reinforced concrete $f_{ck} \geq 30$ N/mm^2	linear concentrated	10 20	15 25	25 35

Table 59.1.3.8.2.3. Values of distance of a_3, in mm, which is assumed to be non-effective from the external face of the supported element

Arrangement of reinforcement	Support	
	Linear	Concentrated
Continuous bars on support	0	0
Straight bars, horizontally bent, near to the end of the element	5	15, but not less than the cover
Tendons or straight bars exposed at the end of the element	5	15
Vertical bending of the bars	15	Cover + internal bending radius

Table 59.1.3.8.2.4. Tolerance Δa_2 in the geometry of the free span between support faces. L = span en mm

Support material	Δa_2
Steel and pre-cast concrete	$10 \leq L/1200 \leq 30$mm
In situ concrete	$15 \leq L/1200 + 5 \leq 40$mm

The net length of the support means a_1 is dependent upon the distances to this from the ends of the support element and of the supported element respectively, which shall satisfy the following conditions:

$d_i \geq c_i + \Delta a,$ with bars anchored using horizontal bending.
$d_i \geq c_i + \Delta a, + r_i$ with bars anchored using vertical bending.

in which:

c_i Nominal cover of the reinforcement
Δa Tolerance for imperfections
r_i Bending radius

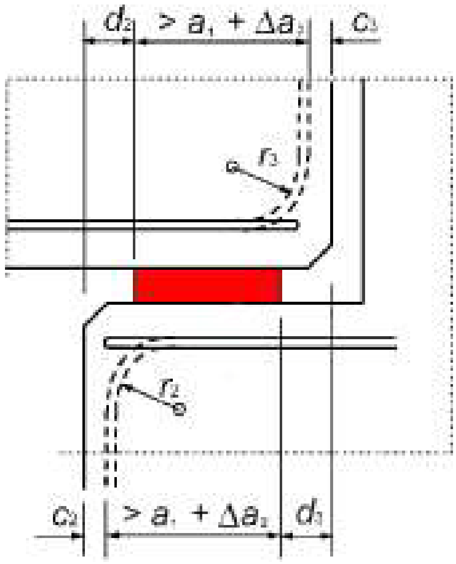


Figure 59.1.3.8.2. b Example of reinforcement detail on a support

59.1.3.8.3 Supports for isolated elements

The equivalent length shall be 20 mm more than that for a non-isolated element.

59.1.4 Pocket foundations

59.1.4.1 General

Concrete pockets shall be capable of transferring axial and shear forces, and bending moments from the column to the foundation.

59.1.4.2 Pockets with keyed surfaces

Pocket foundations which have indented surfaces may be considered to act monolithically with the column.

If the indentations are capable of resisting the transfer of shear stresses between the column and the foundation, the punching shear verification shall be carried out as if the filler and the foundation were monolithic in accordance with Article 46, and as shown in figure 59.1.4.2.

59.1.4.3 Pocket foundations with smooth surfaces

In this case, it is assumed that the axial force and the stress moments are transmitted from the column to the foundation, via the system of forces F_1, F_2 and F_3 and the corresponding friction forces through the concrete filling as shown in figure 59.1.4.3.

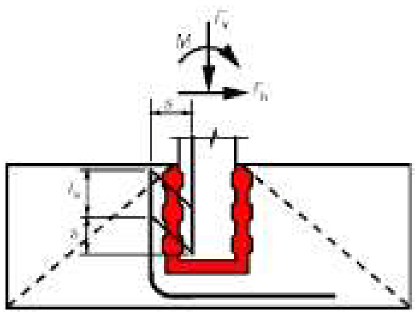


Figure 59.1.4.2

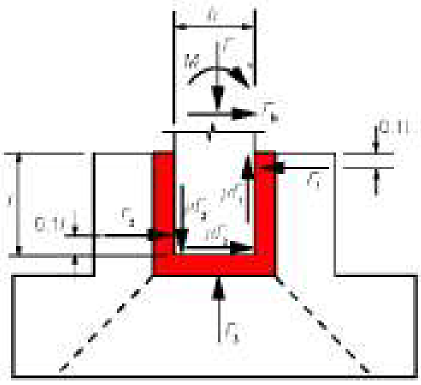


Figure 59.1.4.3

The embedding of the column inside the column pocket in these joints shall be greater than 1.2 times the column thickness ($l \geq 1.2\ h$).

The friction coefficient should not be taken greater than $\mu = 0.3$.

59.1.5 Tying systems

In plane elements, such as walls and slabs loaded in their planes and acting as envelopes, the interaction between their various constituent elements may be obtained by tying the elements together using perimeter transverse reinforcements and/or internal tie beams.

59.2 One-way slabs comprising secondary beams and hollow-core slabs

This Article refers to one-way slabs comprising joists and pre-cast hollow-core slabs with infill elements, in situ concrete and reinforcement incorporated in situ, and basically subjected to bending.

The various combinations of factored actions shall be studied in accordance with the criteria set out in Article 13, when verifying the various Limit States. The Failure Limit State with perpendicular stresses shall be verified in accordance with Article 42. If bending is combined with shear stress, the Ultimate Limit State for Shear Stress shall be verified in accordance with the information in Article 44. If a torsional moment is present, the Ultimate Failure Limit State in torsion of linear elements shall be verified in accordance with Article 45.

If concentrated loads are present in hollow-core slabs without an in situ cast top slab, the Limit State in Punching Shear shall be verified in accordance with Article 46. The Limit State in Longitudinal Shear shall be verified in accordance with Article 47 in slabs comprising reinforced or pre-stressed joists and in hollow-core slabs with in situ cast upper slab.

The Cracking, Deformation and Vibration Limit States shall be verified, as necessary, in accordance with Articles 49, 50 and 51, respectively.

The maximum distance between any secondary beam supports shall be determined, taking account of the fact that during in situ concreting the characteristic execution load on the beams or slabs is the total dead weight of the slab plus an execution imposed load of not less than 1 kN/m^2. Stresses may be determined from linear calculation, assuming constant stiffness in the beam or slab and adopting the distance between the end supports of the secondary beams and the centre lines of the secondary beam supports as the design span L_a of each length (figure 59.2).

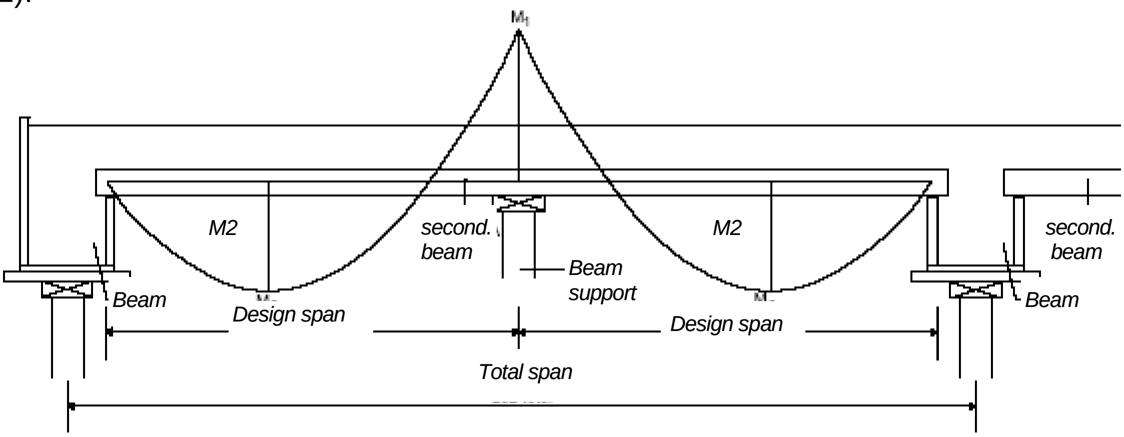


Figure 59.2

In pre-stressed joists and pre-stressed hollow-core slabs, it shall also be ensured that under the action of design execution loads and the effect of pre-stressing following transfer, calculated with all the losses being deducted up until the date of execution of the slab, (and adopting the safety coefficients for the serviceability Limit States corresponding to a temporary situation, in accordance with 12.2), the following stress limits shall not be exceeded:

a) On secondary supports, the maximum compression stress in the lower fibre of the secondary beam or hollow-core slab shall not exceed 60% of the concrete's compression strength, and the bending strength defined in 39.1. shall not be exceeded in its upper fibre.

b) The maximum compression stress in the upper fibre of the secondary beam or hollow-core slab in bays shall not exceed 60% of the concrete's compression strength and the decompression state (zero tension stress) shall not be exceeded in its lower fibre.

The arrangement of reinforcements shall comply with the requirements in Article 69, in the case of passive reinforcements and with the requirements of Article 70, in the case of active reinforcements.

Annexe No. 12 contains arrangements of reinforcements, constructional aspects and specific design aspects for this type of slab.

59.2.1 Geometric conditions

The transverse section of the slab shall satisfy the following requirements (figure 59.2.1):

a) An in situ cast upper slab shall be placed, whose minimum thickness, h_o, shall be 40 mm on top of secondary beams, ceramic or concrete infill members, or pre-stressed hollow-core slabs, and 50 mm on top of infill members of other types, or on top of any type of infill in the case of zones with a design seismic acceleration greater than 0.16 g.
The in situ cast upper concrete slab may be eliminated in pre-stressed hollow-core slabs, apart from where large lateral or large concentrated loads obtain, provided that compliance with the Ultimate and Serviceability Limit States are suitably evidenced. In this case, in order to ensure the combined working of the slabs and the transverse transmission of loads (especially where point or linear loads are present), a tie shall be fitted in the zone where slabs are connected to the main beams or walls.

b) The profile of the infill member shall be such that at any distance c away from its centre line of symmetry, the thickness of the in situ concrete upper slab shall not be less than c/8 in the case of composite infill members and c/6, in the case of hollow infill members.

c) In slabs comprising joists without any transverse reinforcements connected to the in situ poured concrete, the profile of the infill member shall leave a gap of at least 30 mm on either side of the upper face of the secondary beam.

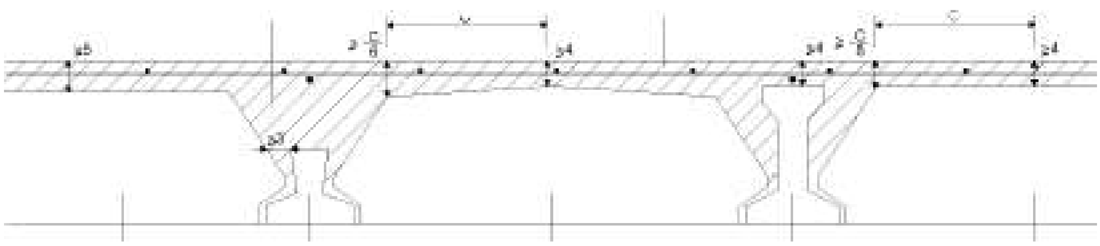


Figure 59.2.1 Geometric conditions of slabs

d) The minimum thickness of the webs and of the upper and lower flanges in pre-stressed hollow-core slabs shall be greater than any of the following values:

- $\sqrt{2h}$, with *h* being the total depth of the pre-cast member in mm.
- 20 mm.
- Result obtained by adding 10 mm to the maximum aggregate size.

e) The shape of the joint between pre-stressed hollow-core slabs shall be suitable for ensuring the introduction of filler concrete so that an enclosed space is created that can transmit the shear force between adjoining slabs and to facilitate the introduction of any reinforcements therein, and ensure proper bonding. The width of the joint at its top shall not be less than 30 mm and if their interior contains any longitudinal tie bars, the width of the joint at the bar shall be at least the greater of the following two values:

- ϕ + 20 mm
- $\phi + 2D$

with D and ϕ expressed in mm.

If the longitudinal joint has to resist a vertical shear force, its surface shall be provided with at least one groove of suitable size, with regard to the strength of the filler concrete. The height of the groove shall always be at least 35 mm, its depth (or maximum width) shall be at least 10 mm and the distance between the top part of the groove and the upper surface of the pre-stressed hollow-core slab shall be at least 30 mm.

59.2.2 Distribution reinforcement

Distribution reinforcement shall be arranged in the upper in situ concrete slab; the distances between longitudinal and transverse elements shall not exceed 350 mm, it shall have a minimum diameter of 4 mm in both directions, be perpendicular and parallel to the ribs, and its ratio shall be at least the minimum set out in table 42.3.5.

Distribution reinforcement shall have a minimum diameter of 5 mm if it is taken into consideration for the purposes of checking Ultimate Limit States.

To ensure continuous working of slabs and the transverse transmission of loads in pre-stressed hollow-core slabs without any in situ upper concrete slab (especially where point or linear loads obtain), a tie shall be incorporated in zone where the slabs are connected to the main beams and walls.

59.2.3 Connections and bearings

59.2.3.1 General

Verification shall be carried out on every type of support to ensure that the tensile strength of the reinforcement arranged in the support is greater than the forces produced in the hypothesis that a crack is initiated in the face of the support at a slope of 45°.

59.2.3.2 Bearings in slabs with secondary beams

The ribs in a slab may be connected to the tying system of a wall or to a beam that has a depth that is considerably larger than that of the slab and which is called a direct support, to a plane beam, to the head of a hybrid beam, or to a intersected beam that has the same depth as the slab, and which is called an indirect bearing. Annexe 12 shows diagrams for common supports and the values of the embedded lengths of elements and overlap lengths of protruding reinforcements, to ensure the correct working of the connection.

59.2.3.3 Supports in pre-stressed hollow-core slabs

The supports may be either direct or indirect in this type of slab.

a) Direct supports in pre-stressed hollow-core slabs on beams or walls shall rest on a layer of fresh mortar at least 15 mm thick, or elastomeric material strips, or on individual supports located underneath each rib in the slab (figure 59.2.3.3.a). Pre-stressed hollow-core slabs are not allowed to be directly supported on brick; reinforced concrete transverse reinforcements shall be provided for the support.

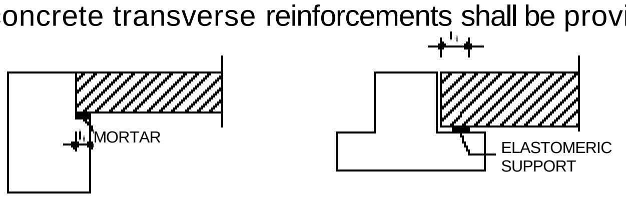


Figure 59.2.3.3.a Direct supports in hollow-core slabs

The design value of the support pressure, assuming an embedment equal to the nominal embedment less 20 mm, shall never exceed 0.4 f_{cd} in the smaller of the two concrete members in contact, where the support comprises mortar or the smaller value of 0.85 f_{cd} and the design strength of the elastomeric material, if this material is used.

b) Indirect supports may or may not comprise shores for the pre-stressed hollow-core slab.

Annexe 12 includes the minimum and nominal embedment of hollow-core slabs according to the type of support (direct or indirect), and of its conditions so that the correct functioning of the joint can be ensured.

59.2.4 Arrangement of reinforcements in slabs

The basic reinforcement in reinforced secondary slabs shall be arranged along their entire length, in accordance with sub-paragraph 42.3.2. Additional lower reinforcement may be incorporated only along part of their length. This additional reinforcement shall be arranged symmetrically about the secondary beams mid point.

Active reinforcement located in the bottom zone of a pre-stressed secondary beam shall comprise at least two bars arranged in the same horizontal plane and symmetrically about the mid vertical plane. The distance between reinforcements in pre-stressed hollow-core slabs shall be less than 400 mm and twice the member’s depth.

Upper reinforcement placed in situ in supports in slabs with secondary beams shall be placed like reinforcement for negative moments, with at least one bar on each secondary beam. If more than two bars need to be fitted per rib, these shall be distributed along the support line in order to ensure than the concrete fills the rib properly and they shall be suitably anchored on either side of the rib.

An upper reinforcement shall be fitted in the outer supports of end bays that can resist a bending moment that is at least a quarter of the maximum moment of the bay. This reinforcement shall extend from the outer face of the support along a length not less than a tenth of the span plus the width of the support. The reinforcement shall extend as a bent bar with the necessary anchorage length.

Upper reinforcement shall be arranged in pre-stressed hollow-core slabs without any in situ upper slab whenever necessary, in suitably prepared voids that are subsequently filled in and with the concrete being removed from its top for a length that is at least equal to that of the bars (figure 59.2.4.a).

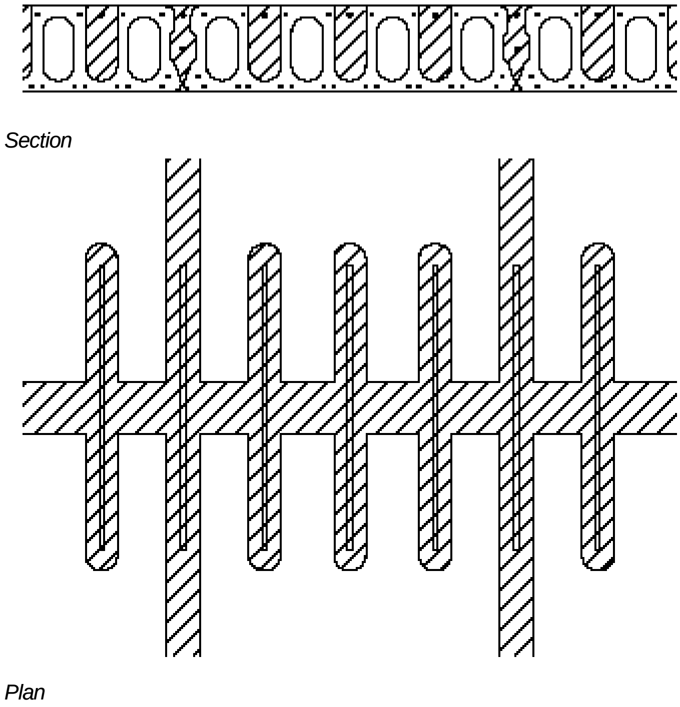


Figure 59.2.4.a Upper reinforcement in pre-stressed hollow core slabs

59.3 Other types of slabs comprising precast elements

Particular care shall be taken in the structural analysis of slabs comprising precast elements other than those contained in paragraph 59.2, such as members with a π section or channels or precast pre-slabs, taking account of the structural layout, loads, supports and the characteristics of the materials during successive constructional phases, during handling, transport, and assembly, and other aspects covered in paragraph 59.1 of this Code.

Article 60 Structural elements for bridges

60.1 Decks

60.1.1 General considerations

This Article applies to the most common bridge decks made from structural concrete, such as decks comprising precast beams, slabs, ribbed decks and box section decks.

The actions and their characteristic, representative, and design values to be considered when designing decks and the combinations to be established when verifying the various Serviceability and Ultimate Limit States, shall be as set out by the special regulations in force or, failing this, the information in this Code.

When determining the effects of these loads, the structure shall be modelled and the necessary analysis carried out in accordance with the provisions in Chapter 5.

The geometric characteristics and the materials which have to be considered when verifying Ultimate Limit States shall be as indicated in Chapter 4 and Chapter 8.

The strength and stability of the structure shall be guaranteed at all intermediate construction phases and in its final service state. To ensure this guarantee, the relevant Ultimate Limit State and Serviceability Limit States shall be undertaken at each of the verification phases adopted. Verifications shall be carried out on pre-stressed elements during the pre-stressing force transfer phase, when the structure is in put into service and at much late stages.

The degree to which forces and stresses resulting from rheological phenomena are distributed over time shall be assessed in evolutive structures. this type of phenomenon shall be analysed in accordance with Article 25 if it is significant in these cases.

The verifications for Ultimate Limit Failure State due to perpendicular stresses in decks shall be carried out in accordance with Article 42, or with the simplified formulae in Annexe No. 7, whenever applicable. When verifying and dimensioning the various elements for the Ultimate Limit Failure State due to Shear forces, the information in Article 44 shall be followed. In linear elements in which torsion may be significant, the Ultimate Limit Failure State due to Torsion shall be verified as indicated in Article 45.

The Serviceability Limit States in cracking, deformation and vibration shall be carried out whenever necessary, in accordance with Articles 49, 50 and 51.

The regions where pre-stressing forces are applied shall be dimensioned in accordance with the information in Article 61.

60.1.2 Decks comprising precast beams

The various construction phases of these decks shall be taken into consideration when verifying or dimensioning their elements and suitable account shall be taken of acting loads and structural configuration, its support system and resistant sections during each constructional phase.

The information in Article 18 shall be taken into consideration in the case of double T, channel or similar beams, so that the effective depths of their flanges to be considered in each situation can be determined.

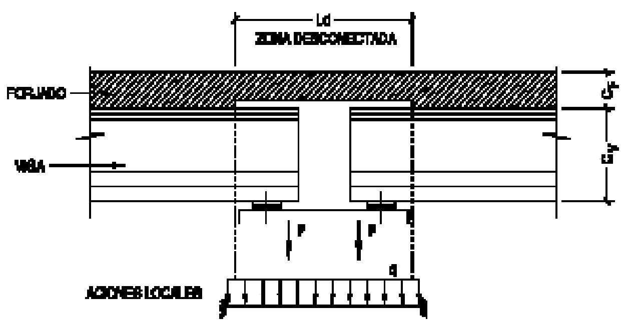
Precast beams and slab shall be connected in accordance with the requirements in Article 47.

Punching shear verifications shall be undertaken in the slab with regard to the effect of concentrated heavy vehicle loads in accordance with Article 46.

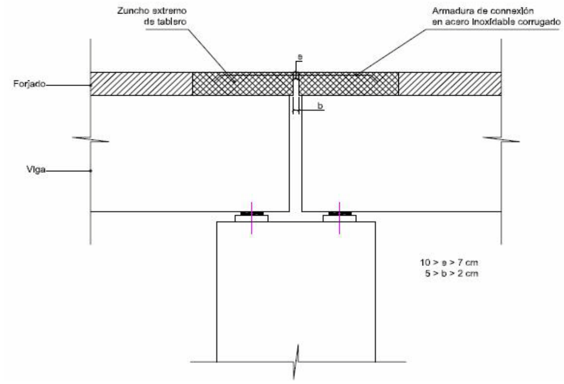
The discontinuity of isostatic decks shall be verified with particular care with regard to deck deformations in the support area, in accordance with Article 50, in order to prevent the platform from breaking due to the relative rotation of the two decks in their support zone. Instantaneous and time-dependent deformations, which can be generated during the life of the beams and, in particular, between their manufacture and their incorporation in the structure, shall be taken into consideration.

When for reasons of driving comfort, the number of transverse joints in a road surface needs to be minimised, this may comprise a continuous slab between decks or a joint or hinge may be incorporated between the compression slabs of the decks, using tie rods. In the first case, the slab shall be rendered continuous on the ends of precast beams and these shall be separated from the latter along a specified length L_d (figure 60.1.2). When dimensioning this zone, not only shall local loads be taken into consideration, but also the forces generated by the deformations imposed in the element due to the relative rotation of the ends of the two decks.

If a continuity hinge with penetrating continuous reinforcement is fitted, for reasons of durability this shall be made from deformed stainless steel.



a) Continuity slab between decks



b) Articulation between connecting slabs using tie rods

Figure 60.1.2

If half way supports are included in precast elements, these type D regions shall be verified and dimensioned using the struts and ties models indicated in Article 59. The dimensioning of the areas where pre-stressing forces are applied shall be undertaken in accordance with the information in Article 62. Channel shaped sections or similar containing envelopes shall be determined in accordance with paragraph 60.5.

60.1.3 Slab decks

Decks in which the ratio between the width of their enclosed part and their span is less than 0.25, may be considered to be linear elements for the purposes of analysing forces and verifying Limit States. If this is not the case, decks shall be deemed to be two-way slabs.

The joint between the enclosed parts of the slab any cantilevers, shall be dimensioned in accordance with the requirements in sub-paragraph 44.2.3.5.

Verifications of punching shear for the effects of the wheels of heavy vehicles shall be undertaken on cantilevers and zones above hollow elements, in accordance with Article 46.

The dimensioning of the zones where pre-stressing forces are applied in pre-stressed slabs shall be undertaken in accordance with the information in Article 62.

60.1.4 Ribbed decks

Article 18 shall be taken into consideration so that the effective depths of the flanges to be considered in each situation can be determined.

The joints between ribs and upper slab in horizontal and vertical joining sections shall be in accordance with the requirements in sub-paragraph 44.2.3.5

Verifications of punching shear due to concentrated heavy goods vehicle loads shall be carried out on the upper slab in accordance with Article 46.

The region where the pre-stressing force is applied shall be dimensioned in accordance with the information in Article 62. If membranes are included in support sections, these shall be dimensioned in accordance with paragraph 60.5.

60.1.5 Box girder decks

Article 18 shall be taken into consideration when determining the effective widths of the flanges to be considered in each situation.

The horizontal and vertical joining sections of joints between the various slabs forming the box girder shall be in accordance with the requirements in sub-paragraph 44.2.3.5.

Verifications of punching shear due to concentrated heavy goods vehicle loads shall be carried out on the upper slab and cantilevers in accordance with Article 46.

The end regions where the pre-stressing force is applied shall be dimensioned in accordance with the information in Article 62. Support membranes shall be verified and dimensioned in accordance with paragraph 60.5.

60.2 Piles

This Article covers compound piles for each support line for one or more shafts with a hollow or solid transverse section, either with or without an upper head block for supporting the deck and whose foundation may comprise individual footings or pile caps, for each shaft or only for all the shafts in the support line.

The thickness of shafts with a box type transverse section, comprising a series of plane partitions, shall not be less than 1/30 of the transverse dimension of each partition. The transverse bending caused by potential differential thrusts between the inside and outside due to the ground water etc. shall be taken into consideration when designing the partitions.

The information contained in Article 61 shall be followed when dimensioning regions D, corresponding to the support zone.

The buckling length and, on the basis of this, the mechanical slenderness of each shaft, shall be determined when dimensioning and verifying the shafts and taking account of their actual connections with the deck.

Piles whose shafts have a mechanical slenderness of λ less than 100, may be deemed to be isolated elements and designed for the Ultimate Instability Limit State in accordance with paragraph 43.5.

Horizontal loads acting on the head of each pile, caused by deformations and loads from the deck may be analysed assuming that the entire structure acts in a linear way; second order effects may be disregarded.

In piles with a large slenderness ratio (λ > 100), once the loads transmitted by the deck have been distributed between the piles using linear methods, non-linear geometric and mechanical analysis shall be carried out to determine the forces in accordance with Article 21. Generally, the pile will merely need to be analysed as an isolated element taking account of its actual connections to the deck. However, in very particular cases, it may be appropriate to analyse the entire structure.

The requirements contained in Article 58 shall be followed when verifying and dimensioning foundations.

60.3 Abutments

This Article refers to closed, open abutments and chair shape bearing load elements. Abutments shall withstand the actions transmitted by the deck and support the soil providing access to the structure. The contact with the soil is an important determinant for the durability of this type of element; the requirements in Chapter 7, (Article 37) shall therefore be taken into careful consideration.

The elements of a abutment shall be considered during its various construction phases when verifying and dimensioning.

Unless special measures are adopted to guarantee the compound effect of passive thrust or potential settling of backfills on the outside of the abutment, the dimensioning of its various elements of an abutment does not generally need to be included.

The requirements contained in Article 58 shall be followed when verifying and dimensioning foundations.

The dintels or load bearing elements of an open abutment may generally be considered to be plane structures. The requirements contained in Article 58 shall be followed when verifying and dimensioning foundations.

For the purposes of designing and dimensioning abutments of the straight bottom type may be deemed to be a direct foundation for the loads transmitted by the deck through the supports. The requirements set out in Article 58 shall be followed for their verifying and dimensioning.

60.4 Anchorage zones

Anchorage zones for pre-stressed elements shall be analysed in accordance with Article 62.

60.5 Diaphragms in decks

The function of the diaphragms covered by this article, is to transfer loads from the deck to the piles and stirrups.

The geometric characteristics of diaphragms shall be such that they ensure the flow of forces from the deck to the supports located in this cross-section.

Deck diaphragms located in cross-sections that coincide with the support on piles or abutments shall be designed to transmit both horizontal and vertical axis shear, and the effect of torsion in the piles or abutments (if the deck is supported on this section using one or more than one support means) if applicable.

The design of the diaphragms shall take account of the possible eccentricity of reactions and the consequent bending of the diaphragms when in any situation, the central plane of the diaphragms does not coincide with the support axis.

Diaphragms shall be designed for both definitive and temporary situations obtaining during construction or support replacement operations.

Membranes generally constitute generalized D regions where the strut and tie method shall be used. In addition to the reinforcements obtained from the general strut and tie method, the concentrated load reinforcement will need to be fitted in the area located on top of the supports.

A reinforcement mesh of 0.30 m size and a minimum geometric ratio of 0.15% on each face and direction shall be fitted on each face of the diaphragm for crack control.

Diaphragms in which the decks' webs are directly supported on the support means on piles, shall be at least 0.50 m thick.

Diaphragms for the indirect support of deck webs on support means shall be at least twice the depth of the flanges resting on them.

Monolithic pile-deck joints comprising diaphragms shall have a thickness that is at least the same as the thickness of the faces of the piles located on their extension.

Article 61. Concentrated loads on solid block members

61.1 General

A concentrated load, applied to a solid member constitutes a D region.

The general analysis method for a D region is as indicated in Article 24. The struts, ties and nodes verifications and the characteristics of the materials to be considered shall be as indicated in Article 40.

The equivalent lattice model, in the case of the concentrated load in figure 61.1.a, shall be as indicated in figure 61.1.b.

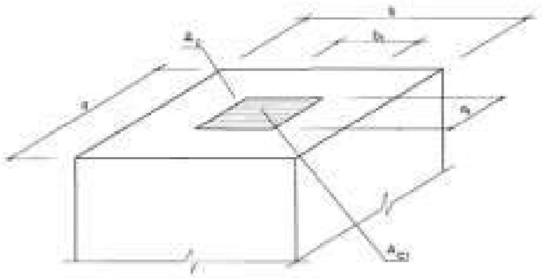


Figure 6.1.1.a

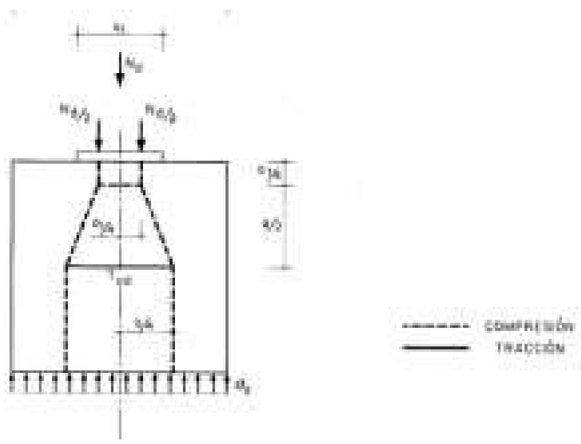


Figure 61.1.b

61.2 Verifying of nodes and struts

The maximum compressive force that may obtain in the Ultimate Limit State on a restricted surface, figure 61.1.a, of area A_{c1}, concentrically and homothetically situated on another area, A_c, assumed to be plane, may be analysed using the following formula:

$$N_d \leq A_{c1} f_{3cd}$$

$$f_{3cd} = \sqrt{\frac{A_c}{A_{c1}}}\, f_{cd} \leq 3{,}3\, f_{cd}$$

Provided the element on which the load is acting does not have any internal voids and its thickness, h is $h \geq 2A_c/u$, with u being the perimeter of A_c.

If the two surfaces, A_c and A_{c1} do not have the same centre of gravity, the perimeter of A_c shall be replaced by an internal homothetic perimeter of A_{c1} and defining an area A_c' which has its centre of gravity at the point of application of the force N, and applying to the areas A_{c1} and A_c' the formulae indicated above.

61.3 Transverse reinforcements

The tie rods T_d indicated in figure 61.1.b shall be dimensioned for the design tension indicated in the following expressions:

$T_{ad} = 0{,}25\, N_d \left(\frac{a - a_1}{a} \right) = A_s\, f_{yd}$ in a direction parallel to *a,* and

$T_{bd} = 0{,}25\, N_d \left(\frac{b - b_1}{b} \right) = A_s\, f_{yd}$ in a direction parallel to *b,* and $f_{yd} \leq 400$ *N/mm² (paragraph 40.2)*

61.4 Criteria for arrangement of reinforcements

The corresponding reinforcements shall be arranged between 0.1a and a and 0.1 b and *b,* distances away respectively. These distances shall be measured perpendicular to the surface A_c.

Stirrups which improve the confinement of concrete shall be preferably used.

Article 62. Anchorage zones

The anchorage of active reinforcements makes up a D region in which the distribution of deformations is non-linear on a section level. The general method in Articles 24 and 40 or the results of experimental studies shall therefore be applied for their analysis.

In those cases where the stresses due to the anchorages and those produced by support reactions and shear stresses may combine at the ends of members, such as beams, it will be necessary to take this combination into consideration, along with the fact that in pre-tensioned reinforcement, the pre-stressing only produces its full effect from the transmission length.

Article 63. Deep beams

63.1 General

Deep beams are straight beams, typically with constant cross-section, and ratio between its span, *l,* and its total depth, *h,* is less than 2 in simply supported beams and 2.5 on continuous beams.

The span of a bay in deep beams shall be considered to be:

- The distance between centre lines of supports, if this distance does not exceed the free distance between the faces of the supports by more than 15%.
- 1.15 times the free span if in other case.

The Bernouilli-Navier hypothesis does not apply in this type of element; the method indicated in Articles 24 and 40 shall be used for their analysis.

63.2 Minimum width

The minimum width is restricted by the maximum value of the compression in the nodes and struts according to the criteria indicated in Article 40. Potential buckling outside their plane of compression fields shall be analysed where necessary, according to Article 43.

63.3 Simply supported deep beams

63.3.1 Dimensioning of the reinforcement

When the load is uniformly distributed and applied to their upper face, the model indicated in figure 63.3.1.a shall be used and the main reinforcement shall be analysed using as the position of the mechanical lever arm *z=0.6l,* for a tensile force that is equal to:

$$T_d = 0.2\ p_d = 1 = 0.4\ Road - A_s\ f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2).

The support node shall be verified using the model in figure 63.3.1.a

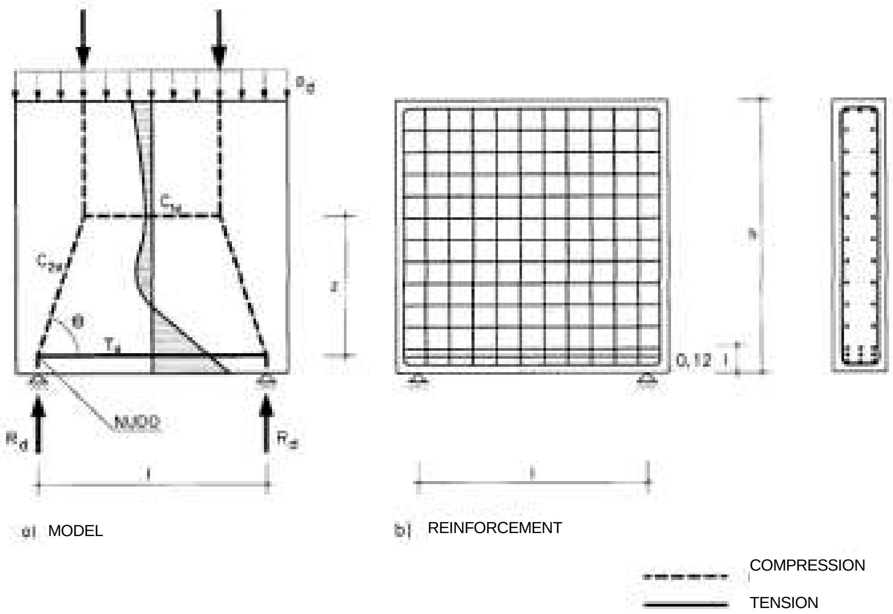


Figure 63.3.1.a

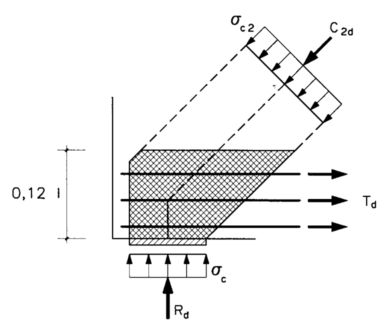
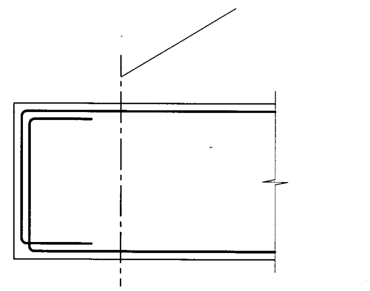


Figure 63.3.1 .b



DETAIL OF ANCHORAGE OF THE BARS
IN THE MAIN REINFORCEMENT

Figure 63.3.1.c

A minimum reinforcement of 0.1% of the ratio in each direction and each face in the element, shall be provided in addition to the main reinforcement, corresponding to T_d.

Particular care shall be paid to the anchorage of the main reinforcement (see figure 63.3.1.c), the anchorage length of which shall lie between the support axis and the end of the member.

If necessary, additional reinforcement shall be provided on supports in accordance with Article 61.

63.3.2 Verifying of nodes and struts

When verifying nodes and struts, it is sufficient to check that the stress in the concrete in the support node is:

$$\frac{R_d}{ab} \leq f_{2cd}$$

in which:

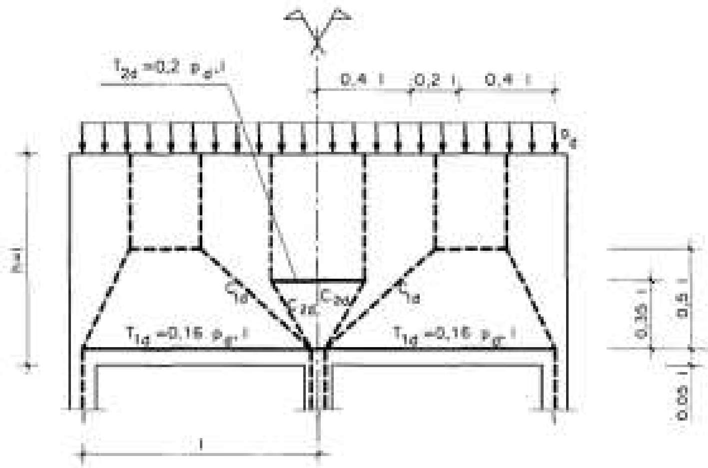
a, *b* *D*imensions of the support

$f_{2c}d$ Compressive strength of the concrete.

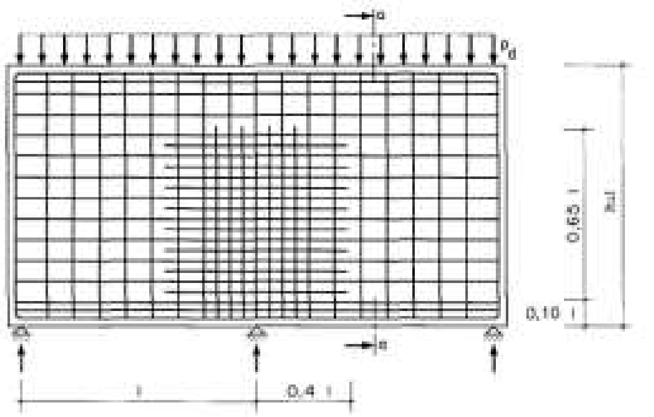
$$f_{2cd} = 0,70\ f_{cd}$$

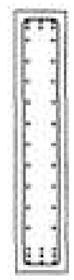
63.4 Continuous deep beams

In the case of a uniformly distributed load applied on the upper surface, the model is the one described in figures 62.4.a and b.



MODEL





SECTION

REINFORCEMENT

Figure 63.4.a

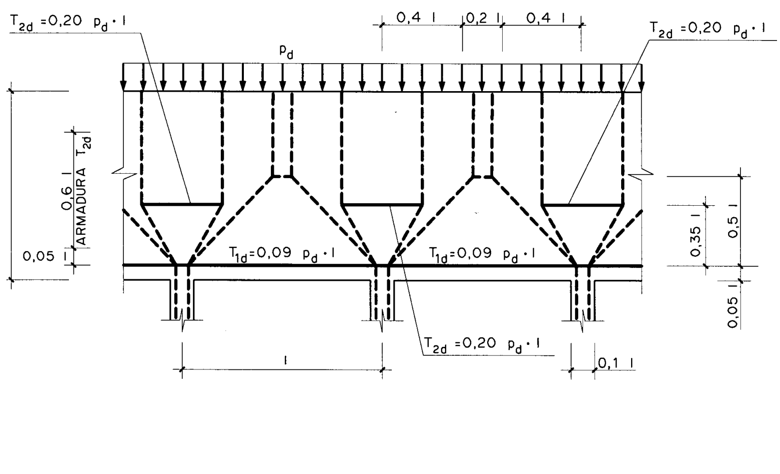


Figure 63.4. b

63.4.1 Dimensioning of the reinforcement

According to the models above, the reinforcement in the intermediate support zone in continuous beams of equal bays shall be designed for a tensile force of:

$$T_{2d} = 0{,}20\ p_d\, l = A_s\, f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2).

The lower reinforcement in end bays shall be designed for a force of:

$$T_{1d} = 0{,}16\ p_d\, l = A_s\, f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2).

The lower reinforcement in intermediate bays shall be designed for a force of:

$$T_{1d} = 0{,}09\ p_d\, l = A_s\, f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2).

A minimum reinforcement of 0.1% of the ratio in each direction and in each face of the element, shall be provided in addition to the main reinforcements indicated in the paragraph above.

In respect of the end supports, special attention should be paid to the anchorage of the reinforcement (see figure 62.3.1.c), which should have an anchorage length that lies between the support axis and the end of the member.

If necessary, additional reinforcements shall be provided in the support according to Article 61.

63.4.2 Verifying of nodes and struts

When verifying nodes and struts it is sufficient to check that the localized compression in supports is:

$$\frac{R_{ed}}{a_e\, b_e} \leq f_{2cd}$$

$$\frac{R_{id}}{a_i\, b_i} \leq f_{2cd}$$

in which:

R_{ed} Design reaction at an external end support.
R_{id} Design reaction at an internal support.
a_e, b_e Dimensions of the end support
a_i, b_i Dimensions of the internal support
f_{2cd} Compression strength of the concrete.

$$f_{2cd} = 0.70\, f_{cd}$$

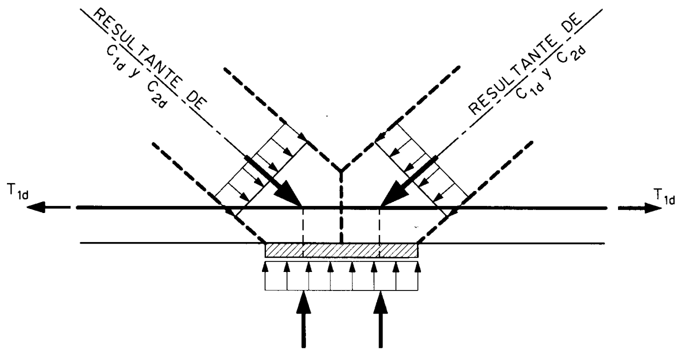


Figure 63.4.2

Article 64. Corbels and half supports

64.1 Corbels

64.1.1 Definition

Corbels are defined as a short cantilever beams in which the distance *a*, between the line of action of the main vertical load at load and the section adjacent to the support is less than or equal to the effective depth *d,* in that section (figure 64.1.1).

The effective depth d_1 measured in the external edge of the area where the load is applies shall be greater than or equal to 0.5*d*.

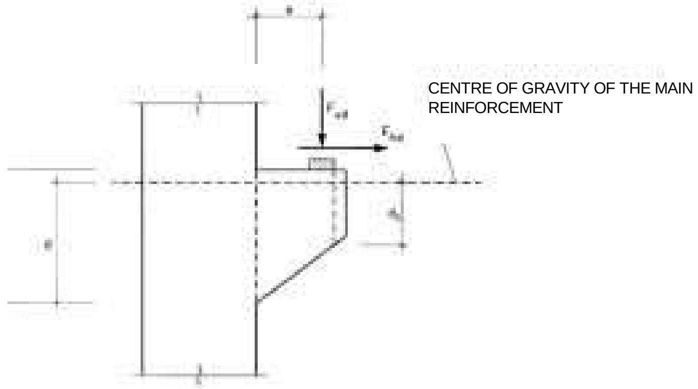


Figure 64.1.1

64.1.2 Verifying of the element and reinforcement dimensioning

Since it is a D region, the general analysis method indicated in article 24 shall be used.

Verifications on struts, ties and nodes and the characteristics and materials to be considered shall be as indicated in Article 40.

64.1.2.1 Verifying nodes and struts and design of the reinforcement

The equivalent lattice model indicated in figure 64.1.2 may be used.

The slope angle θ the diagonal compressions (struts) may, in accordance with the geometric and execution conditions, be assumed to be equal to the following values:

- $\cot g\,\theta = 1.4$ if the corbel is monolithically concreted with the column. Various values of $\cot g\,\theta$ may be adopted but they shall never be more than 2.0 subject to evidence in the form of theoretical or suitable experimental studies.
- $\cot g\,\theta = 1.0$ if the corbel is concreted on top of the hardened concrete column.
- $\cot g\,\theta = 0.6$ as for the previous case but if the hardened concrete has a low degree of surface roughness.

The effective depth *d* of the corbel (figures 64.1.1 and 64.1.2) shall satisfy the following condition:

$$d \geq \frac{a}{0,85} \cot g\,\theta$$

64.1.2.1.1 Dimensioning of the reinforcement

The main reinforcement A_s shall be dimensioned for a design tension of:

$$T_{1d} = F_{vd}\, tg\theta + F_{hd} = A_s\, f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2).

Horizontal uniformly distributed hoops (A_{se}) shall be incorporated to absorb a total tension of:

$$T_{2d} = 0.20\, F_{vd} = A_{se}\, f_{yd}$$

with $f_{yd} \leq 400$ N/mm^2 (40.2).

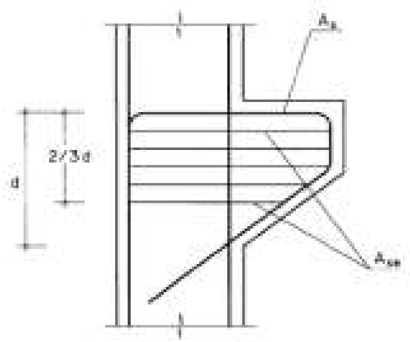


Figure 64.1.2.1.1

64.1.2.1.2 Verifying of nodes and struts

Provided the geometric conditions of 64.1.2.1 are satisfied, it is sufficient to check the localised compression at the support (node 1, figure 64.1.2).

$$\frac{F_{vd}}{b\,c} \leq f_{1cd}$$

In which:

b, c Dimensions in plan of the support.

f_{1cd} Compressive strength of the concrete.

$$f_{1cd} = 0{,}70\ f_{cd}$$

64.1.2.1.3 Anchorage of reinforcements

Both the main reinforcement and secondary reinforcements shall be suitably anchored at the ends of the corbel.

64.1.3 Suspended loads

If a corbel is subjected to a suspended load by means of a beam, (figure 64.1.3.a) various strut-and-tie systems shall be studied in accordance with Articles 24 and 40.
Horizontal reinforcement shall always be arranged near to the upper face of the corbel.

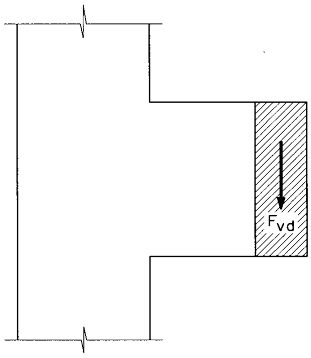


Figure 64.1.3.a

64.2 Half supports

Supports of this type are generally conflictive points where cracking and concrete degradation problems are concentrated; their use shall be avoided whenever possible.
If this type of solution is used, the necessary replacement supports and hence this load situation will have to be taken into consideration.
Because they have geometric discontinuity associated with a sudden change in section, and due a concentrated load acts upon them, beam half supports constitute a type D region; the strut-and-tie method will therefore need to be used. The complexity of the system increases in pre-stressed members because of the presence of the forces in the pre-stressed anchorages.

Article 65. Elements subjected to bursting forces

In those elements where a change in the direction of the forces occurs because of the geometry of the element, transverse tensile stresses may appear that must be resisted by reinforcement, in order to prevent failure of the cover (see figure 64). (see figure 65).
The binding reinforcement may be designed in general terms on the basis of the indications described in Articles 24 and 40.

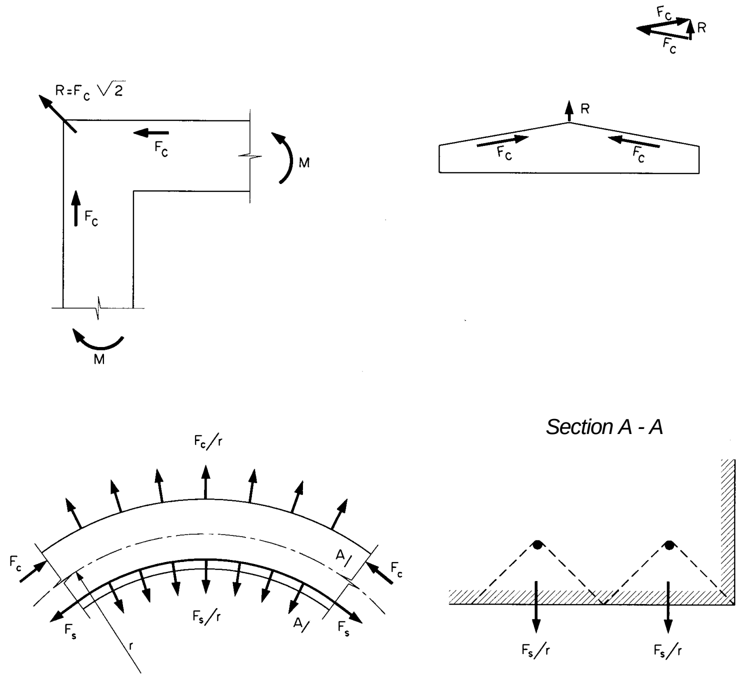


Figure 65