# STRUCTURAL ELEMENTS

# CHAPTER XII

## Article 52 Plain concrete structural elements

### 52.1 Scope of application

Plain concrete structural elements are considered as being both those built of concrete without any reinforcement and those which have reinforcement only to reduce the effects of cracking, generally in the form of fabrics close to their faces.

This chapter is applicable in a purely subsidiary manner to those plain concrete structural elements which have their own specific standards.

**COMMENTS**

Examples of structural elements that may be constructed using plain concrete are, among others, the following: building walls, on basement floors or on other floors, generally with steel fabrics on both faces, foundation footings for brick or concrete walls, footings and foundation piles consisting of reinforced concrete or rolled steel pillars and containing walls in situations of little height etc.

The employment of other types of reinforcement in certain concrete elements, such as steel fibres, may have effects similar to those of the previously mentioned fabrics, such as those designed to reduce cracking and improve ductility.

### 52.2 The concretes that may be employed

Those concretes defined in 39.2 may be used for plain concrete elements.

**COMMENTS**

Very thick elements should take into account the effects of heat produced during setting, which may, at times, indicate the use of low hydration heat cements.

### 52.3 Design actions

The combined design actions that apply at the Ultimate Limit States are those contained in Article 13.

### 52.4 Design of sections under compression

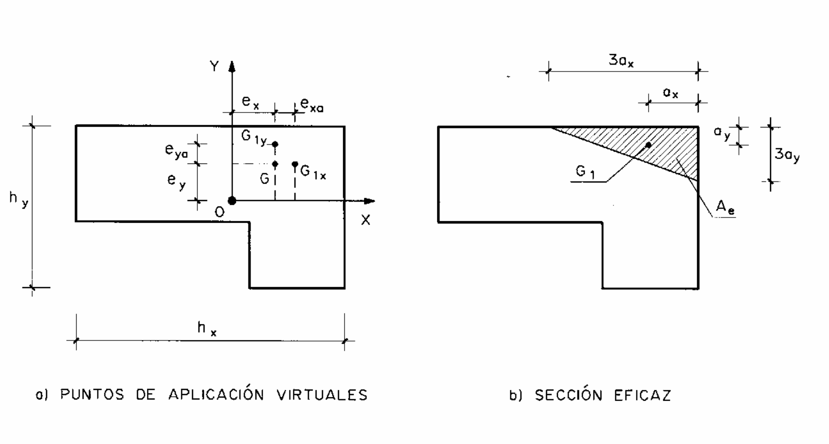
In a plain concrete element section, acted upon only by a perpendicular compressive stress with design value $N_d$ (positive), applied at point $G$, with eccentricity components ($e_x$, $e_y$) relative to a system of centroidal axes (case a; figure 52.4.a), $N_d$ should be taken as being applied at virtual point $G_1$ ($e_{1x}$, $e_{1y}$), which should be the less favourable of the following two points:

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

where:

| | |
|---|---|
| $h_x$ and $h_y$ | The maximum dimensions in these directions. |
| $e_{xa}$ | $= 0.05\ h_x \geq 2$ cm. |
| $e_{ya}$ | $= 0.05\ h_y \geq 2$ cm |

Figure 52.4.a




a) VIRTUAL APPLICATION POINTS

b) EFFECTIVE SECTION

The resulting stress $\sigma_d$ is calculated by assuming a uniform stress distribution in a part of the section, which is termed the effective section, of area $A_e$ (case b; figure 52.4.a) delimited by a secant, and with a centroid coinciding with the virtual point of application $G_1$ of the perpendicular stress, the rest of the section being considered inactive.

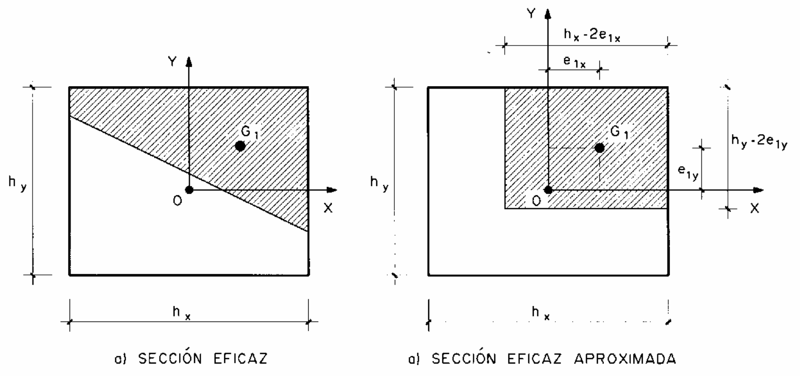
The safety condition is given by:

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

**COMMENTS**

When the effective section is geometrically difficult to determine, it may be replaced by an approximate effective section, consisting of the total section, with the centroid coinciding with point $G_1$ (figure 52.4.b). Any potential error is always on the side of safety, since the effective section has maximum area. If a suitable choice is made, any error committed will be small.

Figure 52.4.b




a) EFFECTIVE SECTION

b) APPROXIMATE EFFECTIVE SECTION

## 52.5 Design of sections under compression and shear stress

In a section of a plain concrete element which is acted upon by a diagonal compressive stress, with components of design value $N_d$ and $V_d$ (both positive) applied at point *G*, the virtual point of application $G_1$ and the effective area $A_e$ may be determined as stated in 52.4. The safety conditions are given by:

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

COMMENTS

Strictly speaking, safety conditions should be based on an intrinsic concrete strength curve, although conclusive experimental results are not available in order to establish the same.

When compressions are dominant, as is most frequently the case, the established safety conditions are sufficiently adjusted to any intrinsic curve and, when the compression is small, they are on the side of safety.

A more exact test than the one included in the article could be done in this way:

$$\frac{\sigma_{md}}{2} + \sqrt{\left(\frac{\sigma_{md}}{2}\right)^2 + (\tau_{md})^2} \leq 0.85 f_{cd}$$

$$\sqrt{\left(\frac{\sigma_{md}}{2}\right)^2 + \tau_{md}^2} - \frac{\sigma_{md}}{2} \leq f_{ct,d}$$

Where

$$\sigma_{md} = \frac{N_d}{A_e} \qquad \tau_{md} = \frac{V_d}{A_e}$$

Taking fct,d as:

$$f_{ct,d} = \frac{f_{ct,k}}{\gamma_c}$$

being $f_{ct,k}$ as defined in 31.1.

## 52.6 Consideration of slenderness

In a plain concrete element that is subject to compression, with or without shear stress, the first order effects produced by $N_d$ are increased by second order effects because of its slenderness (52.6.3). In order to take these second order effects into account, $N_d$ should be considered as acting at point $G_2$, which results from displacing $G_1$ (52.4) by an imaginary eccentricity as defined in 52.6.4.

**COMMENTS**

The transversal deformation produced by the eccentric compression or first order deformation is increased by the sag or second order transversal deformation.

### 52.6.1 Virtual width

The virtual width $b_v$ of the section of an element is given by: $b_v$ = 2$c$, where $c$ is the minimum distance from the section centroid (figure 52.6.1) to a line meeting the perimeter.

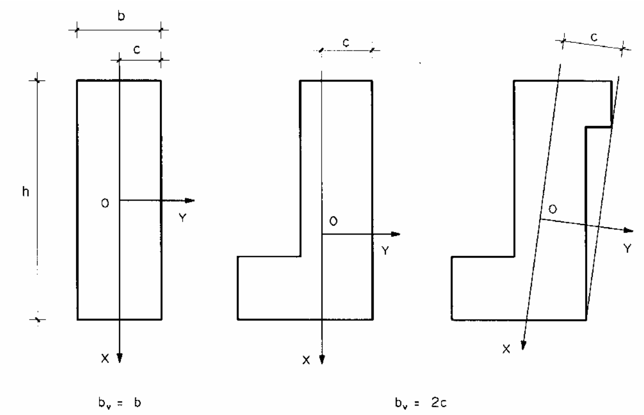



Figure 52.6.1

**COMMENTS**

In a rectangular section wall, with a width $b$, it is $b_v$=$b$.

### 52.6.2 Effective buckling length.

The effective buckling length $l_o$ of an element is expressed as: $l_o$ = $\beta l$, where $l$ is the height of the element from base to top, and $\beta=\beta_o\zeta$ the slenderness factor, with $\beta_o=1$ in elements in which the top is horizontally braced and $\beta_o$=2 in elements where the top is not braced. The factor $\zeta$ takes into account the effect of bracing by transverse walls, and has a value of:

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

where:

*s* The spacing between bracing walls.

In columns or other exempt elements $\zeta = 1$ is taken.

**COMMENTS**

In a wall that is braced by transversal walls, when the spacing of these is less than four times the height of the wall, the sag transversal deformation is reduced, which should be taken into account through the $\zeta$ factor.

### 52.6.3 Slenderness

The slenderness $\lambda$ of a plain concrete element is determined by the following expression:

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

### 52.6.4 Fictitious eccentricity

The buckling effect of an element with slenderness $\lambda$ is considered equivalent to that produced by the addition of a fictitious eccentricity $e_a$ in the direction of the axis and parallel to the virtual width $b_v$ of the section having a value of:

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

where:

$E_c$ Modulus of elasticity of concrete of the concrete in N/mm$^2$, at 28 days (39.6).
$e_1$ Determinant eccentricity (figure 52.6.4), having the value:

- Elements where the top is horizontally braced: the maximum value of $e_{1y}$ on the abscissa $z_o$.

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

- Elements where the top is not braced: the value of $e_{1y}$ at the base:

The element is calculated on the abscissa $z_o$ with eccentricity components ($e_{1x}$, $e_1 + e_a$) and at each end with its corresponding eccentricity ($e_{1x}$, $e_{1y}$).

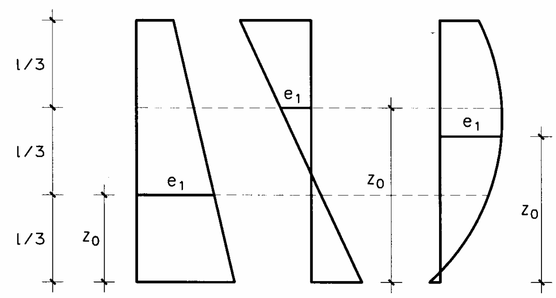



Figure 52.6.4

**COMMENTS**

The fictitious eccentricity that is calculated in this fashion includes the deformation due to creep in an average environment.

## Article 53 Floor slabs

Reinforced or prestressed concrete slabs are controlled by that established in the applicable Instruction for the Design and Execution of Reinforced or Prestressed Concrete One-way Slabs currently in force, but should comply with the provisions contained in this Instruction in all other respects.

## Article 54 Beams

Beams that are subject to bending should be designed in accordance with Article 42 or the simplified formulae of Appendix 8, based on the design values for the strengths of the materials (Article 15) and the increased values of combined actions (Article 13). In the event that bending is combined with shear force, the member should be designed for shear in accordance with Article 44, and in accordance with Article 45 if torsion is also present. For composite members the Limit State for longitudinal shear (Article 47) should be verified.

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

In the case of T-beams or specially shaped beams, sub-section 18.2.1 should be applied.

The reinforcement distribution should comply with the provisions of Article 66 for reinforcing steel and 67 for prestressing steel.

**COMMENTS**

This article is intended to serve as a reminder of the various verifications that should be carried out in the case of beams. Evidently, all the articles in this instruction are either directly or indirectly applicable to all types of members, however, here the emphasis has been placed on those that are most closely associated with the members under bending.

## Article 55 Columns

Columns should be designed for combined axial force and bending in accordance with Article 42 or the simplified formulae of Appendix 8, based on the design values for the materials' strengths (Article 15) and the increased values of combined actions (Article 11). When there is appreciable column slenderness, the Instability Limit State should be verified (Article 43). If there is shear force, the member should be designed for the this force in accordance with Article 44, and in conformity with Article 45 if torsion is also present.

When applicable, the Cracking Limit State should be verified in accordance with Article 49.

The smallest dimension of columns executed on site should have a dimension greater than, or equal to, 25 cm.

The reinforcement distribution should comply with the provisions of Article 66 for reinforcing steel and 67 for prestressing steel.

The main reinforcement should consist of at least four bars in the case of rectangular sections and six bars in that of circular sections, with a spacing no greater than 35 cm between two consecutive bars. The diameter of the thinnest compression bar should not be less than 12 mm. In addition, the bars should be secured by ~~links or~~ stirrups, which should have maximum spacing and minimum diameters for transverse reinforcement as stated in 42.3.1.

In circular columns, the stirrups may be either circular or distributed in a helicoidal form.

**COMMENTS**

The purpose of this article is to provide a reminder of the various verifications that should be carried out in the case of columns. Evidently, all the articles in this instruction are either directly or indirectly applicable to all types of members, however, here the emphasis has been placed on those that are most closely associated with the members under combined axial force and bending .

## Article 56 Slabs or flat slabs

### 56.1 Slabs or flat slabs on continuous supports

This article is applicable to reinforced and prestressed concrete slabs or flat slabs on continuous supports.

Unless otherwise justified, the total depth of the slab or flat slab should not be less than $l/40$ or 8 cm, where $l$ is the span corresponding to the smallest bay.

In structural analysis, the indications of Article 22 should be followed.

In order to verify the various Limit States, the different combinations of factored actions should be checked, in accordance with the criteria given in Article 13.

The Ultimate Limit State of Failure due to bending and axial force should 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 should be verified in accordance with the indications of Article 44.

In addition, where necessary, the Cracking, Deformation and Vibration Limit States should be verified in accordance with Articles 49, 50 and 51, respectively.

The reinforcement distribution should comply with the provisions of Article 66 for reinforcing steel and 67 for prestressing steel.

For rectangular flat slabs supported on two sides, transverse reinforcement should, in all cases, be distributed in parallel to the direction of the supports and designed for a moment equal to 20% of the principal moment.

COMMENTS

The purpose of this article is to provide a reminder of the various verifications that should be carried out in the case of slabs or flat slabs on continuous supports. Evidently, all the articles in this instruction are either directly or indirectly applicable to all types of members, however, here the emphasis has been placed on those that are most closely associated with the previously mentioned types.

## 56.2 Slabs or flat slabs on single 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 capital.

Except where specially justified, in the case of reinforced concrete slabs, the total depth of the slab should not be less than the following values:

- Solid slabs having a constant thickness, $L/32$.
- Hollow slabs having a constant thickness, $L/28$.

where $L$ is the panel's largest dimension.

The spacing between rib axes should not exceed 100 cm and the thickness of the compression layer should not be less than 5 cm and should incorporate fabric distribution reinforcement.

In structural analysis, the indications of Article 22 should be followed.

In order to verify the various Limit States, the different combinations of factored actions should be checked, in accordance with the criteria given in Article 13.

The Ultimate Limit State of Failure due to bending should be verified in accordance with Article 42, taking into consideration an equivalent bending moment ~~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 should be verified in accordance with the indications of Article 44. In particular, verification is required in the ribs where they meet the drop, and in edge elements~~,~~ and beams.

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

The Punching Limit State should be verified in accordance with the indications contained in Article 44.

In addition, where necessary, the Cracking, Deformation and Vibration Limit States should be verified in accordance with Articles 49, 50 and 51, respectively.

The layout of reinforcement should agree with the provisions of Articles 66 for reinforcing steel and 67 for prestressing steel.

**COMMENTS**

The purpose of this article is to provide a reminder of the various verifications that should be carried out in the case of slabs or flat slabs on single supports. Evidently, all the articles in this instruction are either directly or indirectly applicable to all types of members, however, here the emphasis has been placed on those that are most closely associated with the previously mentioned types.

The following constructive arrangements may be followed in slabs or flat slabs on single supports.

a) Solid slabs with constant depth.

The spacing between the main reinforcement should not exceed 25 cm, nor twice the slab thickness, and its diameter must not exceed one tenth of the slab thickness.
The upper and lower reinforcement corresponding to the direction with the least internal forces in each panel should have a cross section that is at least 25% of the similar reinforcements in the main direction.
In addition to the reinforcement that results from the slab design, the slab edges should also incorporate that corresponding to any specific eventual stresses that have to be taken into consideration.
The reinforcement should be arranged in each direction in the following manner:

- In middle strips: uniformly.
- In column strips:
    - Those corresponding to positive bending moments, uniformly.
    - Those corresponding to negative bending moments, taking into account 22.4.6.

b) Hollow slabs.

The distribution of the reinforcement between the ribs and the panel drops should be carried in accordance with that established for solid slabs, with the limits established for the maximum reinforcement diameter and amount in the direction of least internal forces also being applicable.
In spite of what is set out in Article 44, stirrups with a spacing not exceeding $0.5d$ should be arranged in the hollow slab edge ribs, that are capable of resisting the shear stresses and the forces that are produced.

c) In both solid and hollow slabs, the lower reinforcement of the support strips in each direction should be either continuous or overlapping. A minimum of two bars should pass through the inside of the interior column and these should be anchored in the exterior columns.

d) In both solid and hollow slabs that are not braced against movement, the reinforcement length should be determined by calculation, but should not be less than that indicated in figure 56.2.

Figure 56.2

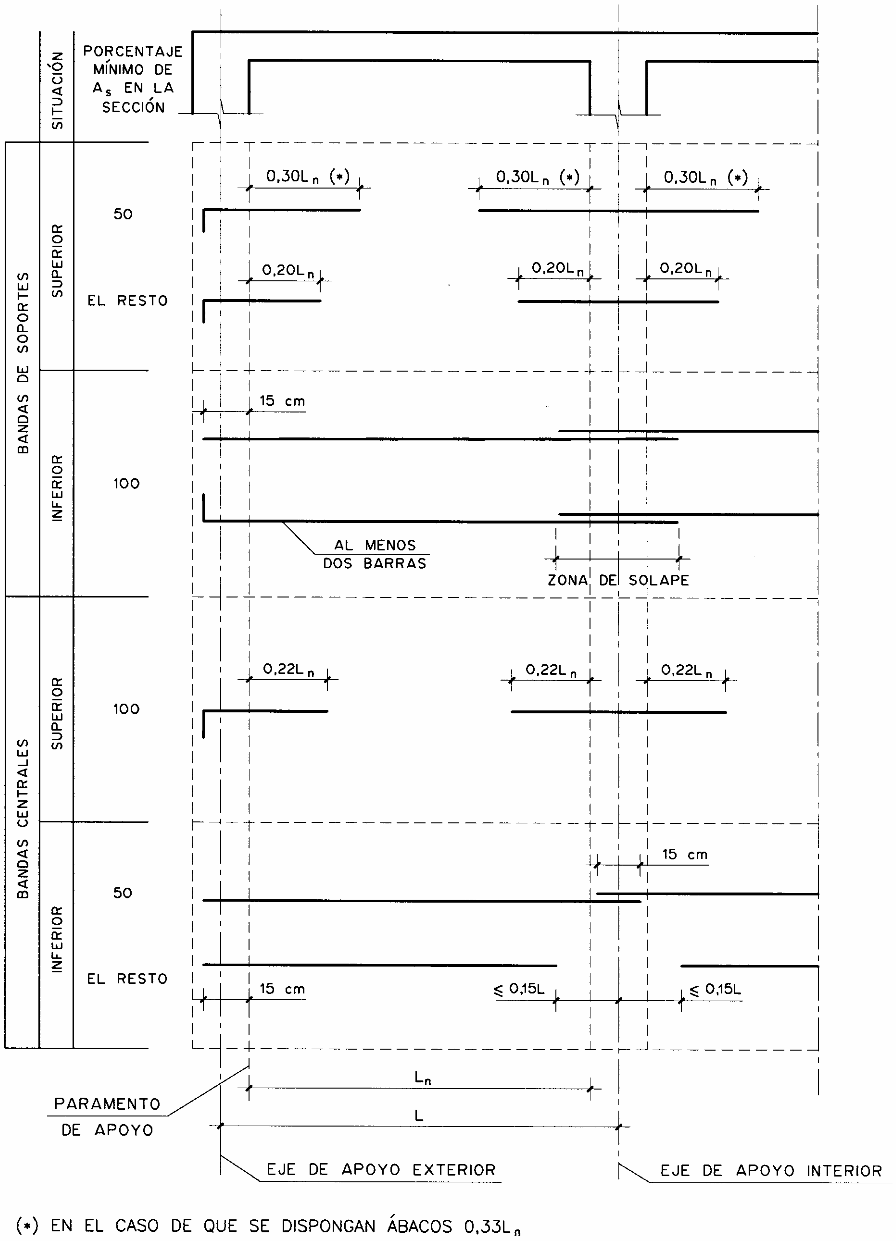
COLUMN STRIPS UPPER (Superior) SITUATION

MIDDLE STRIPS LOWER (Inferior)

MINIMUM PERCENTAGE OF As IN THE SECTION

OTHER

BEARING SURFACE




(*) EN EL CASO DE QUE SE DISPONGAN ÁBACOS $0{,}33L_n$

OUTSIDE BEARING AXIS INSIDE BEARING AXIX

(*) IN CASE ABACUS 0,36Ln WERE USED

## Article 57 Walls

Walls that are subject to bending should be designed in accordance with Article 42 or the simplified formulae of Appendix 8, based on the design values for the materials' strengths (Article 15) and the increased values of combined actions (Article 13). If bending is combined with shear force, the member should be designed for the latter stress in accordance with Article 44.

When applicable, the Cracking Limit State should also be verified in accordance with Article 49.

The reinforcement distribution should comply with the provisions of Article 66 for reinforcing steel and 67 for prestressing steel.

## Article 58 Shells

Unless otherwise justified, shells should not be constructed with less than the following thickness of concrete:

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

Unless specially justified, the following conditions should be met:

a) Reinforcement in the shell should be placed in strict symmetry with regards to its average surface.

b) The mechanical reinforcement ratio in any section of the shell should comply with the following limitation:

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

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

c) The spacing of main reinforcement should not exceed:

- Three times the shell thickness, if fabric is placed on the average surface.
- Five times the shell thickness, if fabric is placed close to both faces.

d) Reinforcement covering should comply with the general conditions as required in 37.2.4.

The indications of Article 23 should be followed in shell structural analysis.

In order to verify the various Limit States, the different combinations of factored actions should be checked, in accordance with the criteria given in Article 13.

The Ultimate Limit State due to bending and axial forces should be verified in accordance with Article 42, taking into account the axial force and biaxial bending moment at each point within the shell.

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

The Punching Limit State should be verified in accordance with the indications of Article 44.

In addition, whenever applicable, the Cracking Limit State should be verified in accordance with Article 49.

The reinforcement distribution should comply with the provisions contained in Article 66 for reinforcing steel and 67 for prestressing steel.

**COMMENTS**

The purpose of this article is to provide a reminder of the various verifications that should be carried out in the case of shells. Evidently, all the articles in this instruction are either directly or indirectly applicable to all types of members, however, here the emphasis has been placed on those that are most closely associated with the previously mentioned types.

In general, shell thickness is decided not by strength requirements, but rather by other factors: deformation characteristics, safety against buckling, reinforcement covering and guarantees of correct execution etc.

With such small thickness levels, any execution error would have significant relative importance, which makes extreme precautions essential.

In all cases, a prior study should be made of the pouring plan.

Non-compliance with condition a) of the section could lead to local effects, the influence of which would have to be studied in each case.

The remaining recommendations in this section are the results of existing experience and they should be respected, except in cases where there are highly justified reasons for not doing so.

The termination of formwork, concrete execution, on-site pouring and the formwork removal and centring operations should be carried out under the strictest conditions of good practice, avoiding all accidental movement of the shell formwork during construction.

## Article 59 Foundation elements

### 59.1 General

The provisions of this article directly apply to footings and pile caps that serve as foundations for individual or linear supports, although their general philosophy may also apply to combined foundation elements.

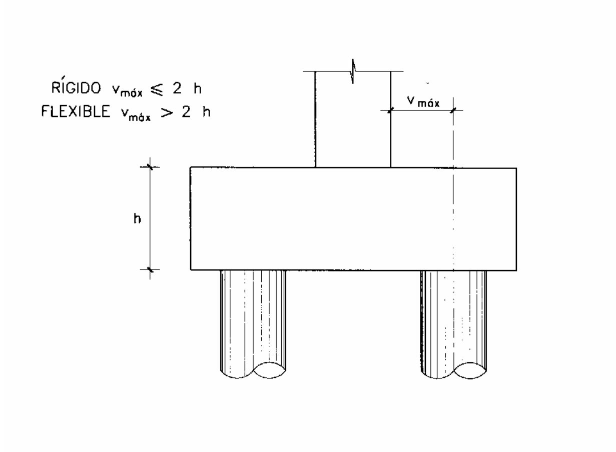
This article also applies to the case of continuous foundation elements for several supports (raft foundations).

Finally, tie beams, piles and plain concrete footings are also included.

### 59.2 The classification of structural concrete foundations

Pile caps and footings may be classified as being either rigid or flexible.

Figure 59.2.a

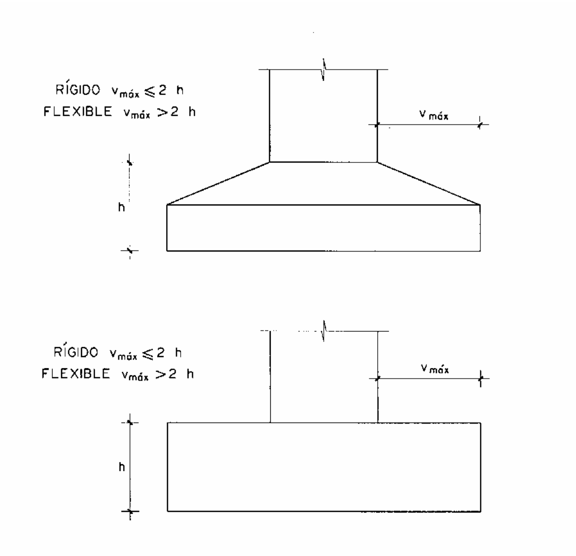



STIFF
FLEXIBLE

Figure 59.2.b

STIFF
FLEXIBLE

STIFF
FLEXIBLE




**COMMENTS**

The concept of rigidity referred to in the article relates to the structure and does not presuppose any specific behaviour on the ground stress conditions.

## 59.2.1 Rigid foundations

The rigid foundations group includes the following:

- Pile caps where offset *v* in the main direction of greatest offset is less than 2*h* (figure 59.2.a).

- Footings where offset *v* in the main direction of greatest offset is less than 2*h* (figure 59.2.b).

- Bored piles.

- Massive foundation elements: counterweights and massive gravity walls etc.

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

**COMMENTS**

For the calculations of the stresses in this type of element, in the ground or the reactions in the piles, it may be considered that, in general, the foundation element behaves as a rigid solid that is subjected to the forces transmitted by the support and to the stresses resulting from the ground or the reactions in the piles (figures 29.2.1.a and b).

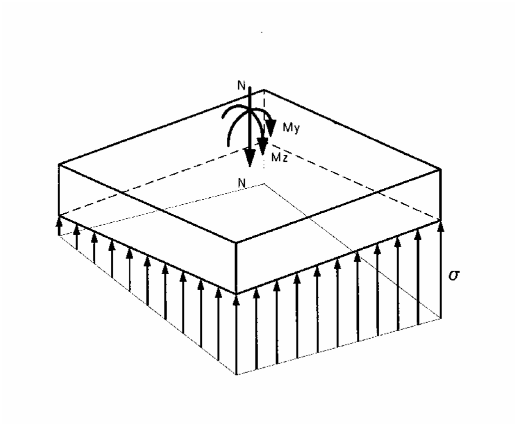



Figure 59.2.1.a

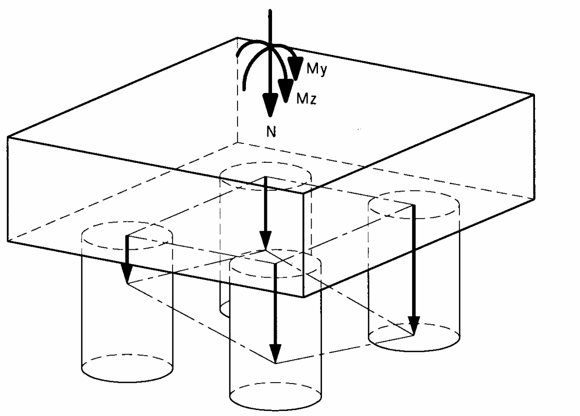



Figure 59.2.1.b

### 59.2.2 Flexible foundations

The flexible group of foundations includes the following:

- Pile caps where offset *v* in the main direction of greatest offset is greater than *2h* (figure 59.2.a).

- Footings where offset *v* in the main direction of greatest offset is greater than 2*h* (figure 59.2.b).

- The 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.

COMMENTS

In this type of foundation, the foundation forces, together with the ground's response, depend on the relative flexibility of the foundations and the ground; its evaluation should take into consideration a suitable ground-foundation interaction model.

### 59.2.3 Other foundation elements

This includes tie beams, covered in section 59.5, piles, which are dealt with in section 59.6, and plain concrete footings, described in section 59.7

## 59.3 General design criteria

Foundation elements should be dimensioned so that they are able to resist the acting loads and the induced reactions. This requires that the forces acting on the foundation element be transmitted in full to the ground or the piles on which it rests.

In order to define the dimensions of the foundation and to verify the ground stresses or pile reactions, the worst-case combinations transmitted by the structure should be considered by taking into account second order effects in the case of slender supports, the self-weight of the foundation element, and that of the ground bearing on it, using the characteristic values of all these.

For verification of the foundation element at the various Ultimate Limit States, consideration should be given to the effects of ground stresses or pile reactions, calculated for the stresses transmitted by the structure under the increased worst-case combinations, taking into account second order effects in the case of slender supports, and the increased action of the self-weight of the foundation element, where necessary, and that of the ground bearing on it.

**COMMENTS**

The values of the acceptable ground stresses or the acceptable pile loading are established through the theory and practice of Soil Mechanics and the current specific standard.

## 59.4 Verification of elements and reinforcement dimensioning

### 59.4.1 Rigid foundations

In this type of element, the general theory of bending does not apply, and it is therefore necessary to define a strut-and-tie model, in accordance with the criteria indicated in Article 24, and to obtain the reinforcement and verify the conditions in the concrete in conformity with the requirements established in Article 40.

A model should be established for each case in order to balance the external actions transmitted by the structure, the actions due to the ground overburden on the footings, pile caps etc., and the soil stresses or pile reactions.

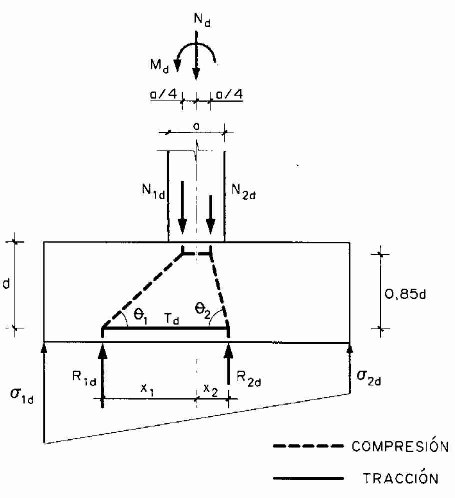
**COMMENTS**

It is impossible to indicate all the possible situations that may occur in rigid foundations, and therefore, the following section merely provides some strut and tie models for the more common situations. Nevertheless, a specific model that takes into account the particular conditions of the foundations under study, should be established for a rigid concrete foundation project.

#### 59.4.1.1 Rigid footings

The model shown in figure 59.4.1.1.a may be employed for rectangular footings that are subjected to combined axial and single bending, provided that the effect of the weight of the footing and the soil situated over it can be ignored.

Figure 59.4.1.1.a




COMPRESSION
TENSION

The main reinforcement should be designed to resist the tension force $T_d$ indicated in the model, which is:

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

with $f_{yd} \ngtr 400$ N/mm$^2$ (40.2); with the variables having the meaning given in figure 59.4.1.1.a, and the stresses $\sigma_{1d}$ and $\sigma_{2d}$ are those calculated by only taking the loads transmitted by the structure into account. This reinforcement should be distributed, without any reduction in section, along the whole length of the footing and should be anchored in accordance with the criteria established in Article 66. Anchorage by means of welded transverse bars is highly recommended in this situation.

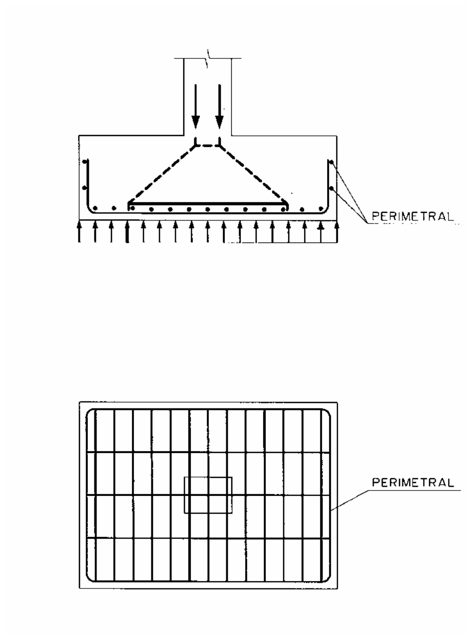
Verification of the strength at the nodes in the model is not generally required if the characteristic strength of the concrete in the columns is equal to the characteristic strength of the concrete in the footing. In any other situation, verification of section 40.4 should be carried out.

However, verification of the nodes implicitly requires the verification of the struts.

**COMMENTS**

The determination of the reinforcement may also be performed from the moment when the ground stresses are produced, together with the self-weight of the footing or of the ground that bears on it when applicable, in the section $S_1$, as defined in 59.4.2.1.1.1, in both directions independently.

In footings that are subjected to appreciable load carrying stresses and bending effects in two directions, it is recommended that, in addition to the main reinforcement, perimeter reinforcement is provided that binds the compression struts (figure (59.4.1.1.b).




PERIMETER REINFORCEMENT

PERIMETER REINFORCEMENT

Figure 59.4.1.1.b

## 59.4.1.2 Rigid pile caps

The required reinforcement should be determined on the basis of the tension forces in the ties of the model adopted for each pile cap. The following sections indicate several models, together with the expressions for determining the reinforcement for the most commonly encountered situations.

Verification of the concrete strength at nodes is not generally required when the piles are concreted *in situ* and if the columns are made of a concrete having a characteristic strength

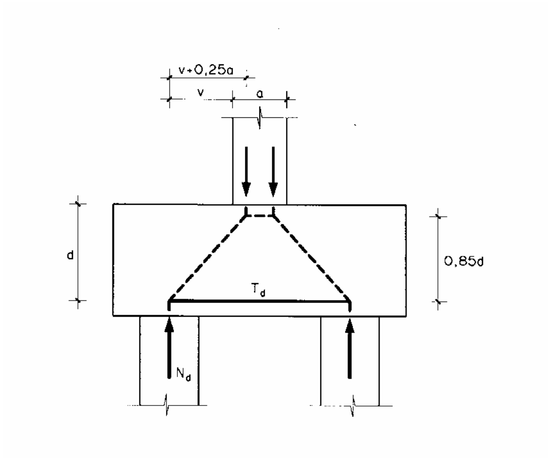



Figure 59.4.1.2.1.1.a

equal to that of the concrete in the pile cap. In any other situation, verification of section 40.4 should be carried out.

However, verification of the nodes implicitly requires the verification of the struts.

### 59.4.1.2.1 Pile caps on two piles

#### 59.4.1.2.1.1 Main reinforcement

The reinforcement should be designed to resist the design tension $T_d$ of figure 59.4.1.2.1.1.a, which may be taken as being:

with $f_{yd} \ngtr 400$ N/mm$^2$ (40.2) and $N_d$ corresponding to the design axial load on the most loaded pile.

The lower reinforcement should be distributed, without any reduction in section, along the whole length of the pile cap. This reinforcement should be anchored in a straight line or at right-angles, or by means of welded transverse bars, beyond the vertical planes that pass through the axis of each pile (figure 59.4.1.2.1.1.b).

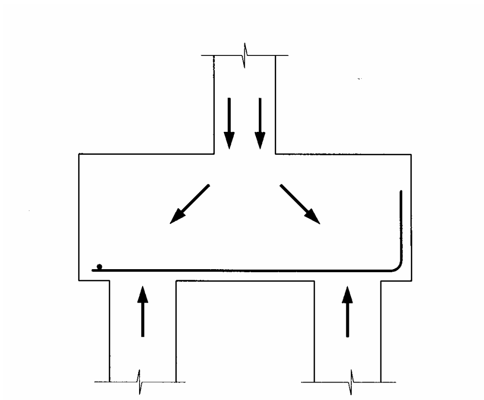


Figure 59.4.1.2.1.1.b

**COMMENTS**

It is possible to introduce a reduction in the anchorage length of the tie reinforcement, since this is compressed in the vertical direction, anchoring a force only 80% of the strength of the reinforcement.

## 59.4.1.2.1.2 Secondary reinforcement

In those pile caps covering two piles, the secondary reinforcement should consist of

- Longitudinal reinforcement distributed in the upper face of the pile cap and extended along its whole length. Its load-bearing capacity should not be less than 1/10 of the corresponding to the lower reinforcement.
- Horizontal and vertical reinforcement arranged in a grid on the side faces. The vertical reinforcement should consist of closed stirrups tying the upper and lower longitudinal reinforcement. The horizontal reinforcement should consist of closed stirrups that tie the aforementioned vertical reinforcement (figure 59.4.1.2.1.2.a). Both these horizontal and ~~the~~ vertical reinforcements should have an area of at least 4‰ of the area of the concrete cross-section perpendicular to its direction. If the width is greater than half the depth, the reference section should be taken as having a width equal to half the depth.

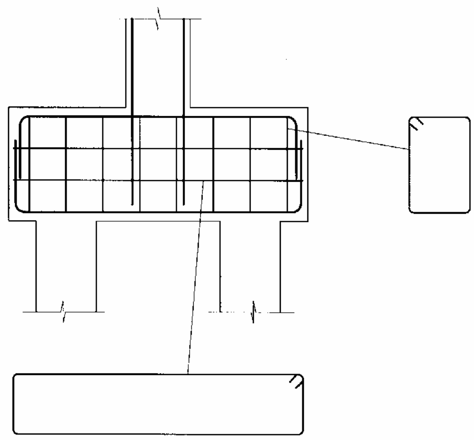


Figure 59.4.1.2.1.2.a

With a high concentration of reinforcement, it is recommended that the vertical stirrups described in this sub-section be placed closer together in the anchorage zone for the main reinforcement, in order to guarantee that the main reinforcement is well bound together within the anchorage zone (figure 59.4.1.2.1.2.b).

**COMMENTS**

The reinforcement described in this article is intended to take into account any possible eccentricities that might be produced in the pile caps due to the accidental movement of the piles with respect to their theoretical position, or due to a transversal bending moment in the column. These effects may be partially or totally assumed by centring beams, which are designed in accordance with 59.5, allowing the secondary reinforcement as described in the article to be reduced.

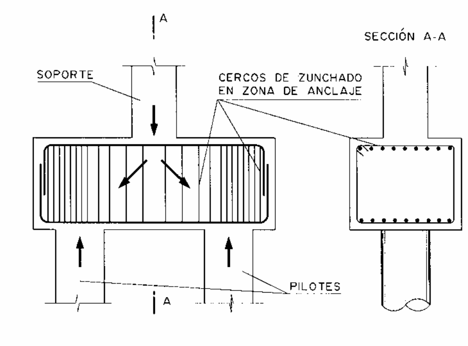



Figure 59.4.1.2.2.2.b.

### 59.4.1.2.2 Pile caps covering several piles

The reinforcement used in pile caps covering several piles may be classified as follows:

- Main reinforcement
  This is located in strips over the piles (see figure 59.4.1.2.2.a). A strip is defined as a zone that has as its main axis the line joining the pile centres, with a width equal to the diameter of the pile plus twice the distance between the upper face of the pile and the centre of gravity of the tie reinforcement (see figure 59.4.1.2.2.b).

- Secondary reinforcement:
  This is located between the strips (see 59.4.1.2.2.1.a).
- Vertical secondary reinforcement:
  This should be located as stirrups tying in the main strip reinforcement (see 59.4.1.2.2.2.a).

PRINCIPAL REINFORCEMENT

SECONDARY REINFORCEMENT

SECTION A-A

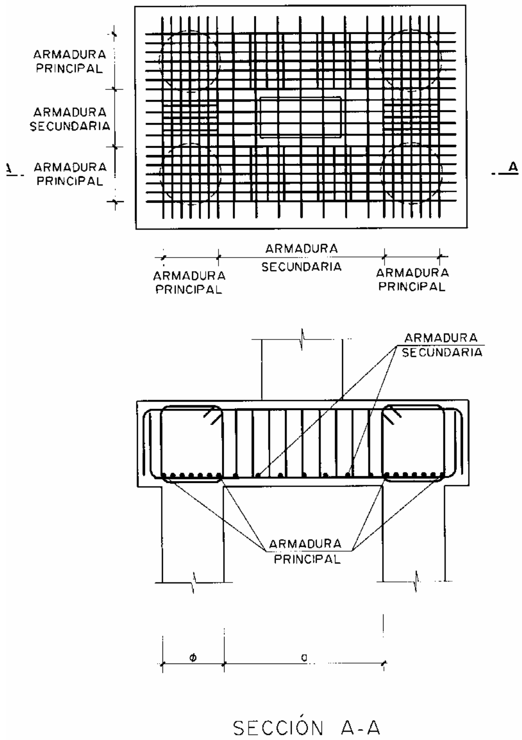
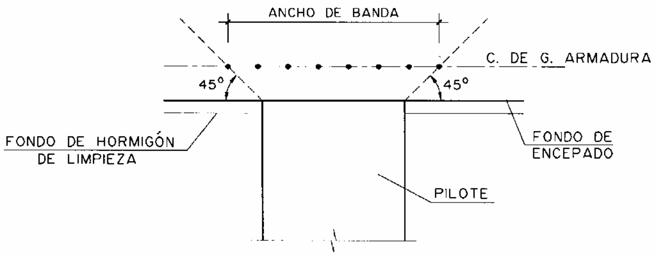



Figure 59.4.1.2.2.a

STRIP WIDTH

REINFORCEMENT CENTROID

LOWER SURFACE OF CLEANING CONCRETE




PILE (Pilote) BOTTOM OF PILE CAP

Figure 59.4.1.2.2.b

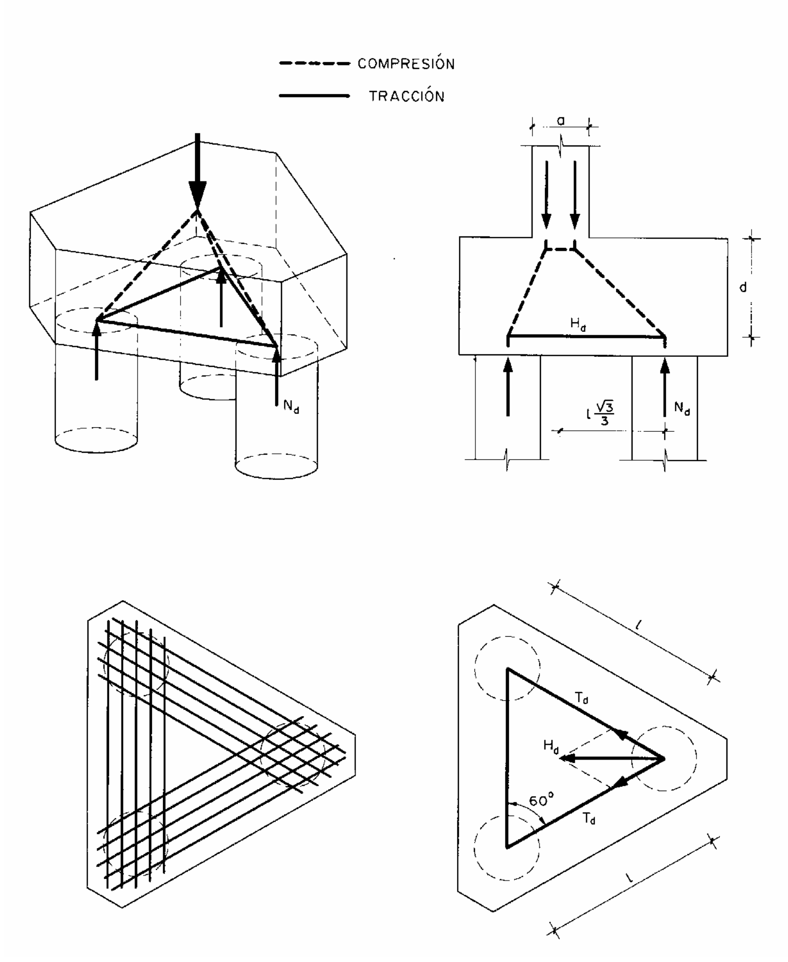
### 59.4.1.2.2.1 Main and horizontal secondary reinforcement

The lower main reinforcement should be placed in strips over the piles. This reinforcement should be arranged so that it is anchored beyond a vertical plane passing through the axis of each pile.

In addition, secondary reinforcement should also be placed in a grid, with a bearing capacity in each direction of no less than 1/4 of the bearing capacity of the strips.

Figure 59.4.1.2.2.1.a

COMPRESSION
TENSION




In the case of pile caps covering three piles placed at the vertices of an equilateral triangle, with the column located at the centroid of the triangle, the main reinforcement between each pair of piles may be calculated based on the tension force $T_d$ given by the expression:

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

with $f_{yd} \ngtr$ 400 N/mm$^2$ (40.2) and where:

$N_d$ Design value of the axial force on the pile with the greatest load (figure 59.4.1.2.2.1.a).
$d$ Effective depth of the pile cap (figure 59.4.1.2.2.1.a).

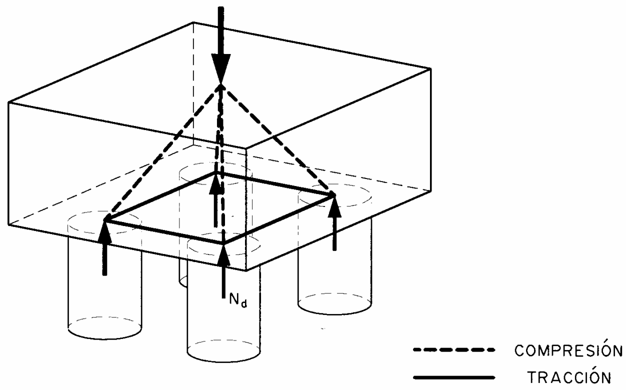
In the situation where the pile caps cover four piles with the column placed in the centre of the rectangle or square, the tension corresponding to each strip may be calculated from the following expressions:

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

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

with $f_{yd} \ngtr$ 400 N/mm$^2$ and where:

$N_d$ Design value of the axial force on the pile with the greatest load (figure 59.4.1.2.2.1.b).
$d$ Effective depth of the pile cap (figure 59.4.1.2.2.1.b).



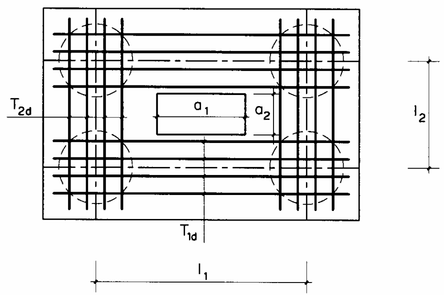
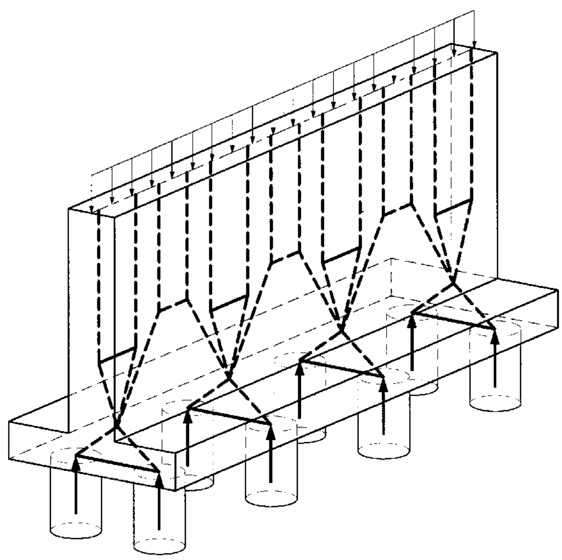




Figure 59.4.1.2.2.1.b

*Figure 59.4.1.2.2.1.c*



In the case of continuous foundations on a linear pile cap, the main reinforcement should be placed perpendicular to the wall and calculated from the expression given in section 59.4.1.2.1, while in the direction parallel to the wall, the pile cap and wall should be calculated as a beam (which is usually a deep beam) supported on the piles (figure 59.4.1.2.2.1.c).

### 59.4.1.2.2.2 Vertical secondary reinforcement

Where there is appreciable load-bearing, it is recommended that vertical secondary reinforcement be distributed as a result of the dispersion of the compression field.

The vertical secondary reinforcement (figure 59.4.1.2.2.2) should have a total capacity of not less than a value of $N_d$ / 1.5$n$, with $n \geq 3$, where:

$N_d$ Design value of the axial force for the column.
$n$ is the number of piles.

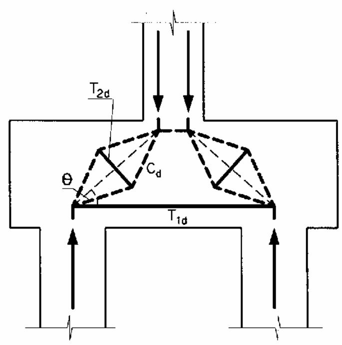



Figure 59.4.1.2.2.2

**COMMENTS**

The criterion employed to obtain vertical secondary reinforcement is that which covers the secondary tension $T_{2d}$ assumed as equal to 0.3$C_d$ , with $\theta$=45$^o$ and vertical reinforcement (figure 59.4.1.2.2.2).

## 59.4.2 Flexible foundations

The general theory of bending applies to this type of foundation.

### 59.4.2.1 Flexible footings and pile caps

Except where a precise study of ground-foundation interaction is carried out, the simplified criteria described below may be used.

#### 59.4.2.1.1 Bending design

##### 59.4.2.1.1.1 Reference section S1

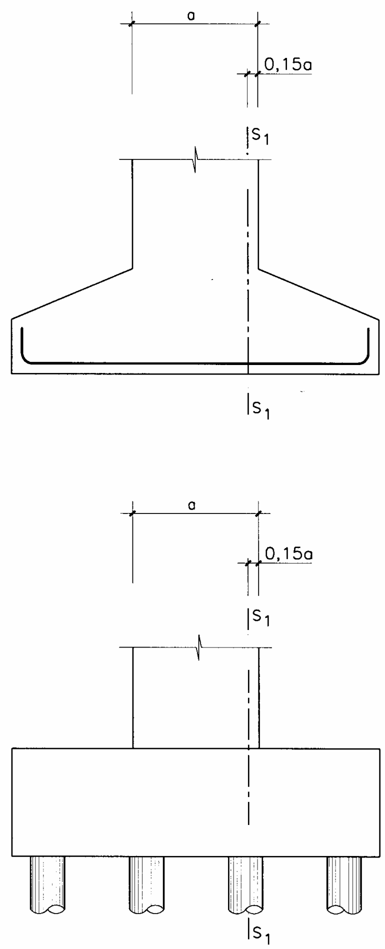
The reference section that is to be considered for the bending design may be defined as follows: it is flat, perpendicular to the base of the footing or pile cap, and takes into account the total section of this footing or pile cap. It is parallel to the face of the column or wall and is located behind this face, at a distance equal to 0.15*a*, where *a* is the dimension of the column or wall measured orthogonal to the section being considered.

The effective depth of this reference section should be taken as being equal to the effective depth of the section parallel to the section $S_1$ which is located at the face of the column or wall (figure 59.4.2.1.1.1.a).

All the above assumes that the column or wall is a concrete element. If it is not, then the quantity 0.15*a* should be replaced with:

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

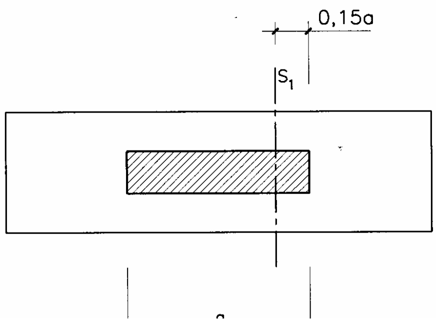
Figure 59.4.2.1.1.1.a



**COMMENTS**

The reference section defined thus takes into account the fact that the bending mechanism is only produced in the offset zone, since the ground stress under the column is directly transmitted by compression to the column, this effect is more acute in the case of long, thin supports, when the reference section is perpendicular to the longest support dimension (figure 59.4.2.1.1.1.b).

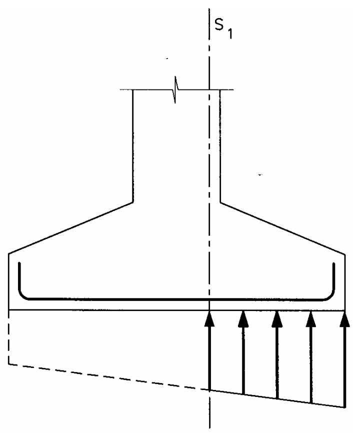
Figure 59.4.2.1.1.1.b




## 59.4.2.1.1.1.1 Calculation of the bending moment

The greatest moment to be considered in the design of flexible footings and pile caps is that contained in the reference section $S_1$ as defined in the previous sub-section (figure 59.4.2.1.1.1.1).

Figure 59.4.2.1.1.1.1

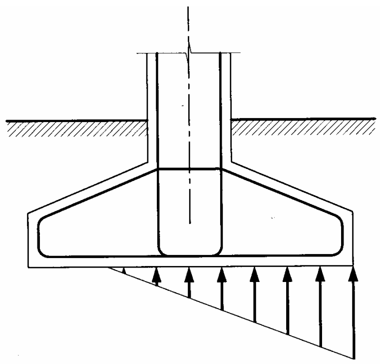



## 59.4.2.1.1.1.2 Determination of the reinforcement

The reinforcement required in the reference section should be determined from a design for pure bending, in accordance with the general design principles for sections subjected to bending, as given in Article 42.

If the stress distribution in the ground is a triangular law as shown in figure 59.4.2.1.1.1.2, then it may occur that the increased moment of the absolute value in the reference section, due to the footing's own weight and that of the ground it supports, is greater than the absolute value of the moment due to the reactions corresponding to the weighted values of the stresses transmitted by the supports, plus the footing's own weight and that of the ground it supports. Therefore, in this situation, it would be necessary to include upper reinforcement capable of supporting the difference of the absolute values of the previously mentioned moments.

Figure 59.4.2.1.1.1.2



## 59.4.2.1.1.2 Layout of reinforcement

In flexible footings and pile caps which are linear, working in a single direction, and in square foundation elements working in two directions, the reinforcement may be distributed evenly across the whole width of the foundation.

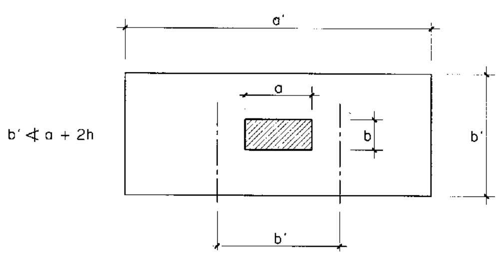
In rectangular foundation elements working in two directions, the reinforcement parallel to the longer side of the foundation, of length *a'*, may be uniformly distributed across the whole width *b'* of the foundation base. The reinforcement that is parallel to the shorter side *b'* should be arranged in such a way so that a fraction of the total steel area $A_s$ equal to $2b' / (a' + b')$ is uniformly distributed throughout a middle strip, coaxial with the column, and having a width equal to *b'*. The remainder of the reinforcement should be uniformly distributed in the two resulting side strips.

The width of strip *b'* should not be less than $a + 2h$, where:

*a* The side of the column or wall parallel to the foundation base's longer side.
*h* Overall foundation depth.

In the situation whereby *b'* is smaller than $a + 2h$, then *b'* should be replaced with $a + 2h$ (figure 59.4.2.1.1.2.a).

Figure 59.4.2.1.1.2.a



The reinforcement that has been calculated as established in 59.4.2.1.1.1.2 should be anchored in accordance with the less favourable of the following two criteria:

- The reinforcement should be anchored in accordance with the conditions stated in Article 66, beyond a section $S_2$ located a distance of one useful depth from the reference section $S_1$.
- The reinforcement should be anchored beyond section $S_3$ (figure 59.4.2.1.1.2.b) for a force of:

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

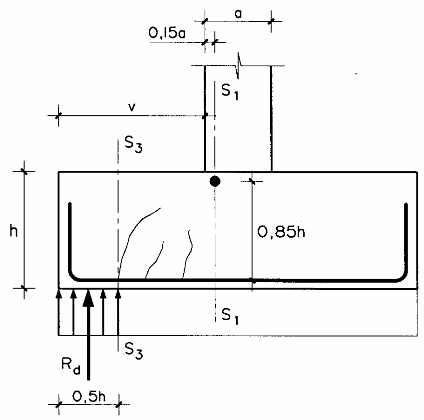



Figure 59.4.2.1.1.2.b

**COMMENTS**

In the case of rectangular footings, the location of the reinforcement parallel to the shorter side *b'* of the footing may be simplified by distributing it uniformly along the entire width *a'* if an area is employed that is greater than required by the calculation given by the following expression:

$$A_{sfic} = \frac{2\ A_s \cdot a'}{a' + b'}$$

with $b' \nless a+2h$, as established in the article, and $A_s$ is the strict reinforcement obtained in 59.4.2.1.1.1.2.

## 59.4.2.1.2 Design for tangential stresses

The resistance to tangential stresses in flexible footings and pile caps, in the vicinity of concentrated loads or reactions (columns and piles), should be verified for shear as a linear element and for punching shear.

**COMMENTS**

This instruction tells the ~~a~~ difference in the shear behaviour between a long, narrow footing that essentially behaves like a beam, and a flexible footing working in two directions, in that a failure may occur due to punching.

#### 59.4.2.1.2.1 Design for shear

In this case the footing or pile cap should be verified for shear stresses in the reference section $S_2$, in accordance with the provisions given in Article 44.

Reference section $S_2$ should be located at a distance that is equal to the effective depth from the face of the column, wall or pedestal, or from the mid-point between the face of the column and the edge of the steel plate, in the case of metal columns on steel bearing plates. This reference section is flat, perpendicular to the base of the footing or pile cap, and takes into account the total section of the same foundation element.

**COMMENTS**

In this situation, the footing or pile cap is considered as being a conventional wide beam with a potential inclined crack that extends along a plane over the foundation width.

#### 59.4.2.1.2.2 Design for punching shear

This Limit State should be verified in accordance with Article 46.

**COMMENTS**

In this situation, the footing or pile cap is considered as working in two directions, with possible inclined cracking along the surface of a cone or pyramid stem around the concentrated load or that of reaction.

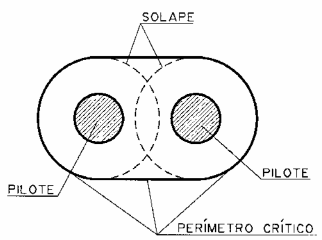
In accordance with this section, it will be necessary to verify the punching ~~shear~~ for the values of the loads transmitted by isolated, most loaded piles. When several piles are sufficiently close together, so that the smaller casing of the individual critical perimeters has a perimeter value that is smaller than the sum of the individual critical perimeters, then the critical perimeter that is taken into consideration for the calculation should be that one with the smallest perimeter, and this will be calculated with the reaction transmitted by the group of piles under consideration. An example of this situation is shown in figure 59.4.2.1.2.2.

Figure 59.4.2.1.2.2

OVERLAPPING

PILE

CRITICAL PERIMETER




If the reactions of either the ground or the piles are not uniformly distributed in the area of the footing or the pile cap, then the area enclosed within the critical perimeter that is under consideration for the dimensioning of the foundation element of the punching shear reinforcement, will be that corresponding to the greatest ground pressures or the greatest pile reactions.

### 59.4.2.1.3 Cracking verification

Whenever required, the Cracking Limit State should be verified in accordance with Article 49.

### 59.4.2.2 Raft foundations

This section applies to broad reinforced or prestressed concrete elements (slabs) used as the foundations for several supports.

The models described in Article 22 may be used to calculate the stresses involved.

In order to verify the various Limit States, the different combinations of factored actions should be checked, in accordance with the criteria given in Article 13.

The Ultimate Limit State of bending should be verified in accordance with Article 42, taking into consideration an equivalent bending moment 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 should be verified in accordance with the indications contained in Article 44.

The Punching Limit State should be verified in accordance with the indications contained in Article 44.

In addition, whenever applicable, the Cracking Limit State should be verified in accordance with Article 49.

The reinforcement distribution should comply with the provisions of Article 66 for reinforcing steel and 67 for prestressing steel.

## 59.5 Centring and tie beams

Centring beams are linear elements that may be used to resist construction eccentricities or moments in pile heads, in the case of pile caps for one or two piles, where the piles do not have a specific capacity for resisting these actions, or in offset footings.

Tie beams are linear elements connecting superficial or deep foundations, and are a particular requirement for foundations in areas prone to earthquakes.

In general, these elements should satisfy the requirements for beams as given in Article 54.

## 59.6 Piles

Verification for a pile is similar to that for a column as stated in Article 55, in which the soil prevents buckling at least partially.

In all cases, a minimum eccentricity, as defined in accordance, with tolerances should be taken into consideration.

For the dimensioning of piles concreted *in situ* without any pile casing, a design diameter of $d_{cal}$ should be used that is equal to 0.95 times the nominal diameter of the pile, $d_{nom}$, and which satisfies the following conditions:

$$d_{nom} - 50\ mm \leq d_{cal} = 0,95\ d_{nom} \leq d_{nom} - 20\ mm$$

## 59.7 Plain concrete footings

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

The reference section $S_1$ that is to be considered in bending calculation may be defined as follows:

It is flat, perpendicular to the base of the footing, and takes into account the total section of the same footing. It is parallel to the face of the column or wall and is located behind this face, at a distance equal to 0.15*a*, where *a* is the dimension of the column or wall measured orthogonal to the section being considered. The total depth *h* of this reference section should be taken as being equal to the total depth of the section parallel to section $S_1$ which is located at the face of the column or wall. In the above, it is assumed that the column or wall is a concrete element; if it is not, the quantity 0.15*a* should be replaced with:

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

The reference section being considered for shear design should be located at a distance that is equal to the depth from the face of the column, wall or pedestal, or from the mid-point between the face of the column and the edge of the steel plate in the case of metal columns on steel load distribution plates. This reference section is flat, perpendicular to the base of the footing, and takes into account the total section of the same footing.

The reference section to be considered in punching shear design should be perpendicular to the footing base and should be defined so that its perimeter is minimum and is not located closer than half the total depth of the footing from the perimeter of the column, wall or pedestal.

The factored bending moment and shear force in the corresponding reference section will cause tensile stresses due to bending and mean tangential stresses, the value of which should be less than the design virtual strength of the concrete to flexural tension and shear.

Bending design should be carried out on the basis of a state of plane stress and strain and on the assumption that the concrete within the section is whole and not cracked.

The footing should be checked for shear and punching shear stresses in the reference sections, as defined above, with the shear strength being governed by the more restrictive of the conditions.

The design tensile and shear strength of the concrete should be taken as being the value $f_{ct,d}$ as stated in 52.2.

For the purposes of the verification for punching shear, the value $2f_{ct,d}$ should be taken.

**COMMENTS**

In accordance with that already established, there is no requirement to carry out any verification of shear or punching shear in footings that are supported on the ground where the offset, as measured from the column face in the two main directions, is less than half the total depth.

The design of plain concrete pile caps supported by piles is forbidden.

## 59.8 Minimum dimensions and reinforcement for footings, pile caps and raft foundations

### 59.8.1 Minimum depths and dimensions

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

The minimum total depth at the edge of reinforced concrete foundation elements should not be less than 25 cm if they are supported by the ground or 40 cm in the case of pile caps. Additionally, in the latter case the thickness at any point should not be less than the diameter of the pile.

The distance between any point on the pile's perimeter and the outer limit of the pile cap base should not be less than 25 cm.

### 59.8.2 Layout of reinforcement

The longitudinal reinforcement should meet the provisions in Article 42. The minimum quantity applies to the sum of the reinforcement in the upper face, the top face and the side walls in the direction under consideration.

The reinforcement distributed in the top, bottom and side faces should have a spacing with intervals no greater than 30 cm.

COMMENTS

It is recommended that the minimum diameter of the reinforcement be distributed in a foundation element be not less than 12 mm.

### 59.8.3 Minimum vertical reinforcement

In flexible footings and pile caps it will not be necessary to arrange for any transverse reinforcement provided that this is not required by the design and the concrete is placed without any form of discontinuity.

If the footing or pile cap essentially behaves as a wide beam and is designed as a linear element in accordance with 59.4.2.1.2.1, then the transverse reinforcement should be in agreement with the provisions of Article 44.

If the footing or pile cap behaviour is rather in two directions, and is designed for punching shear in accordance with 59.4.2.1.2.2, then the transverse reinforcement should be in agreement with the provisions of Article 44.

## Article 60 Concentrated loads on solid blocks

### 60.1 General

A concentrated load that is applied to a solid block constitutes a D region.

Because it is a D region, the general method of analysis is the one indicated in Article 24. The verifications of struts, ties and nodes, and the properties of the materials that are to be taken into consideration, are those established in Article 40.

The equivalent strut-and-tie model, in the case of a centred load as shown in figure 60.1.a, is that indicated in figure 60.1.b.

**COMMENTS**

Just as was defined in Article 24, a concentrated load on a solid block produces a static type of discontinuity.

From the results of a linear ~~condition~~ analysis, it may be seen that if the concentrated load is applied to a solid block having a width of *l*, the load will be uniformly distributed at a depth *h* that is approximately equal to *l* (figure 60.1.c).

The compression flow deviation produces transversal stresses that condition the reinforcement dimensioning.

### 60.2 Verifying nodes and struts

The maximum compressive force acting at the Ultimate Limit State may act on a restricted surface (figure 60.1.a) of area $A_{c1}$ located concentrically and homothetically within another area $A_c$, that is assumed to be flat, and may be calculated from 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 that the element on which the load is acting has no internal cavities and has a thickness $h$ such that $h \geq 2A_c / u$, with $u$ being the perimeter of $A_c$.

If the two surfaces $A_c$ and $A_{c1}$, do not have the same centroid, then the outline of $A_c$ should be replaced with an internal outline, homothetic to $A_{c1}$, which delimits an area $A_c'$ having a centroid at the point of application of the stress $N$, with the formulae given above applying to areas $A_{c1}$ and $A_c'$.

### 60.3 Transverse reinforcement

The ties $T_d$ indicated in figure 60.1.b should be dimensioned for the design tension forces given 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

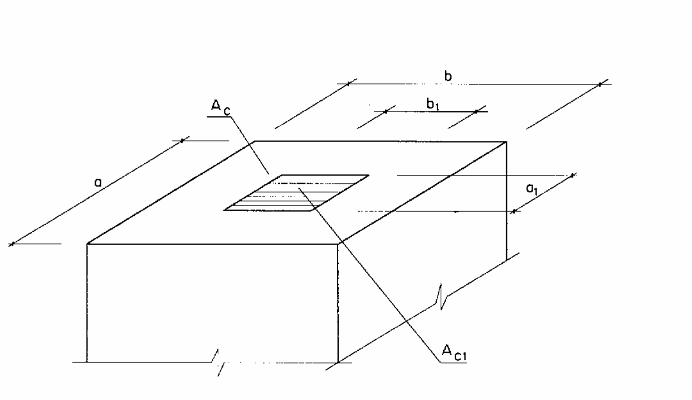



Figure 60.1.a

COMPRESSION
TENSION

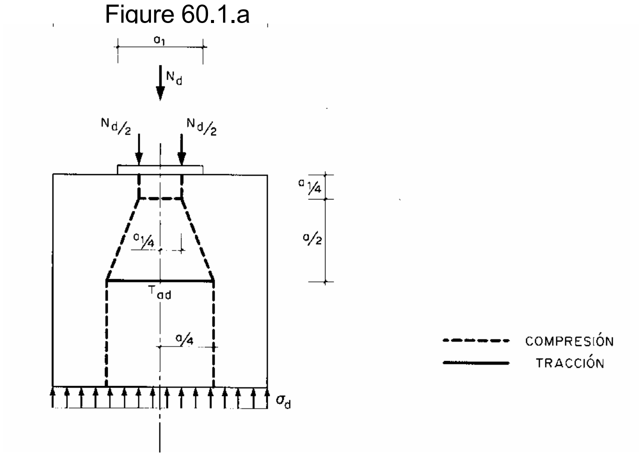



Figure 60.1.b

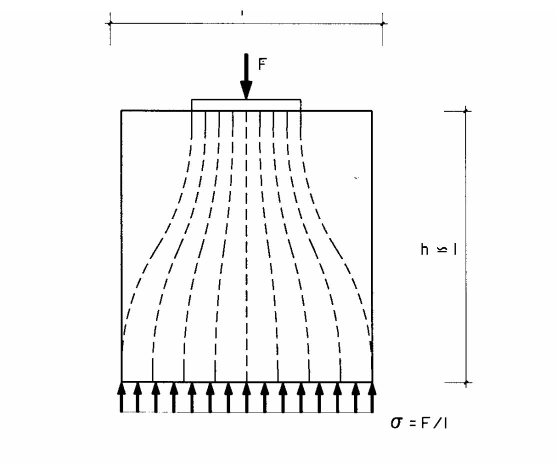



Figure 60.1.c

$$T_{bd} = 0,25\, N_d \left( \frac{b - b_1}{b} \right) = A_s\, f_{yd}$$

in a direction parallel to $b$, with $f_{yd} \ngtr 400$ N/mm$^2$ (40.2).

## 60.4 Reinforcement distribution criteria

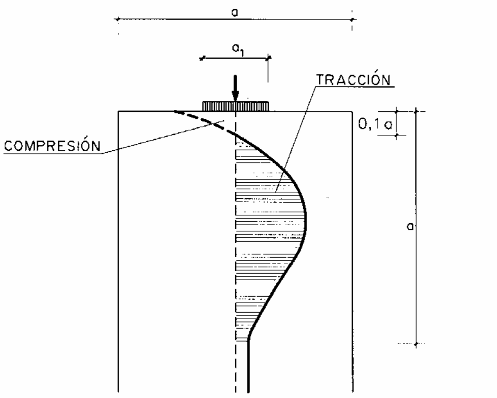
The corresponding reinforcement should be distributed at a distance of between 0.1*a* and *a*, and 0.1*b* and *b*, respectively. These distances should be measured perpendicularly the surface $A_c$.

The use of stirrups which improve the concrete confinement is recommended.

COMMENTS

Figure 60.4 shows the distribution of transverse forces parallel to side *a* (an identical distribution would correspond to the transverse forces parallel to side *b*).

COMPRESSION




TENSION

Figure 60.4

## Article 61 Anchoring zones

The anchorage of prestressing steel makes up a D region in which the distribution of strains is not linear on a section level. Therefore, the general method established in Articles 24 and 40 or the results of experimental studies should be employed in its design.

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 prestressing only produces its full effect from the transmission length.

#### COMMENTS

Just as was stated in Article 42, an anchorage zone is a static "D" region due to the appearance of concentrated loads.

In general, high tensile stresses are produced in the distribution zone of concentrated forces, making it necessary to limit their values in order to prevent any possible cracking. When necessary, the transversal dimensions of the member should be increased, especially in the case of relatively powerful tendons.

It is accepted that, as from a certain distance $h$ from the far face in the case of post-tensioned reinforcement, the stress distribution is uniform. This distance $h$ is assumed as being equal to the longest dimension of the member's transverse section.

In the case of prestressed reinforcement, it is accepted that the stress distribution is uniform at a distance of $h+l_{bpt}$ where $l_{bpt}$ is the transfer length of the prestressing steel.

## Article 62 Deep beams

### 62.1 General

Deep beams are straight beams that, in general, have a constant cross-section where the ratio of span $l$ to total depth $h$ is less than 2 for simply supported beams or 2.5 for continuous beams.

In deep beams, the span of a bay is taken as being:

- The distance between support axes, provided this distance is not more than 15% greater than the free distance between the faces of the supports.
- This is taken as being 1.15 times the free span in all other cases.

With this type of element, the Bernouilli-Navier hypotheses do not apply, and therefore the method indicated in Articles 24 and 40 should be used for their design.

#### COMMENTS

In accordance with the given definition, the concept of a deep beam (or wall beam) does not possess an absolute character, but instead, depends on the depth/span ratio of the member.

In accordance with that stated in Article 24, a deep beam is a generalised "D" region.

## 62.2 Minimum width

The minimum width is limited by the maximum value for compression in the nodes and struts in accordance with the criteria given in Article 40. Possible out-of-plane buckling of compression fields should be analysed where necessary in accordance with Article 43.

## 62.3 Simply supported deep beams

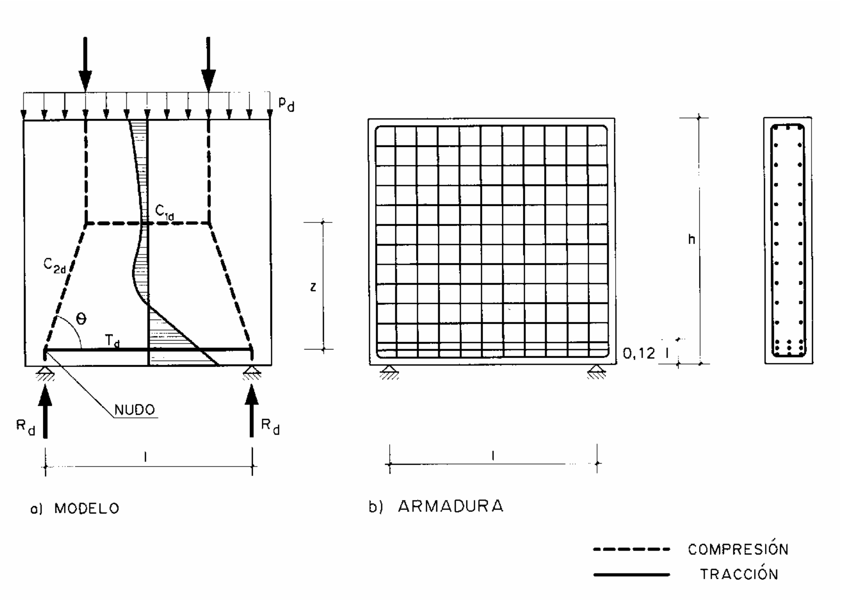
### 62.3.1 Determination of the reinforcement

In the case of a uniformly distributed load applied to the upper surface, the model is the one shown in figure 62.3.1.a and the main reinforcement should be calculated by taking the position of the lever arm as $z$ = 0.6$l$, for a tensile force that is equal to:

$$T_d = 0,2\, p_d\, l = 0,4\, R_d = A_s\, f_{yd}$$

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

The support node should be verified in accordance with the model shown in figure 62.3.1.b.




a)MODEL b) REINFORCEMENT TENSION COMPRESSON

*Figure 62.3.1 a.*

In addition to the main reinforcement corresponding to $T_d$, a minimum of 0.1% reinforcement should be placed in each direction in each face of the element:

Special attention should be paid to the anchorage of the main reinforcement (see figure 62.3.1.c), the anchorage length of which shall lie between the support axis and the end of the member.

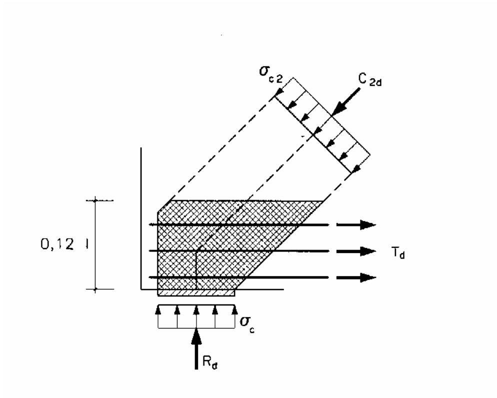
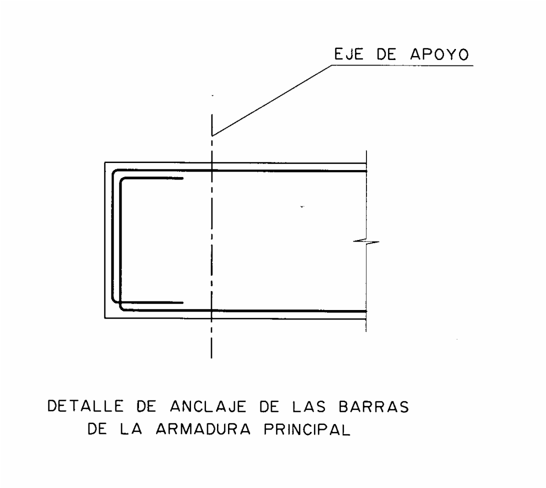



Figure 62.3.1.b




BEARING AXIX

ANCHORAGE OF PRINCIPAL
REINFORCEMENT

Figure 62.3.1.C

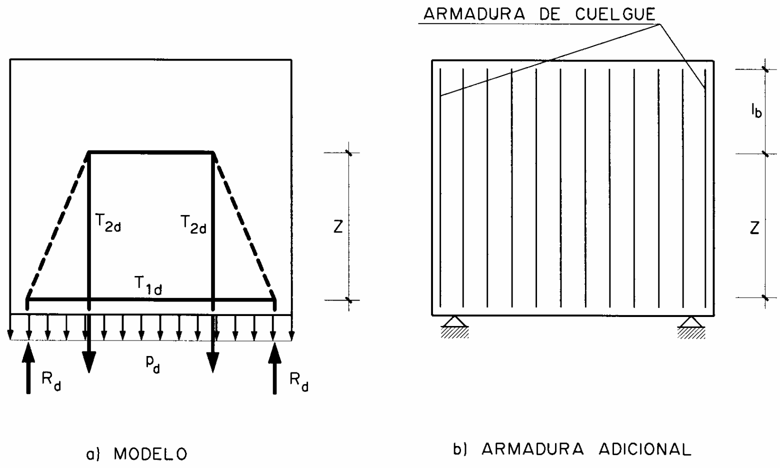
Where required, additional reinforcement may be distributed in the supports in accordance with Article 60.

**COMMENTS**

In the case of other loading configurations, or where cavities are present, the equivalent strut and tie model should be studied in accordance with that established in Article 24 and 40.

In the special case of uniform load applied to the lower part of the beam, it is sufficient to add to the previously calculated reinforcement the suspended reinforcement $T_{2d}$ indicated in figure 62.3.1.d, which is distributed uniformly over the two faces of the beam and anchored from an upper level of the lever arm *z*.

Figure 62.3.1.d HANGING REINFORCEMENT




a) MODEL b) ADDITIONAL REINFORCEMENT

### 62.3.2 Verifying nodes and struts

When verifying nodes and struts, it is sufficient to check that the compression at the supports is as follows:

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

where:

*a*, *b* The support's dimensions.

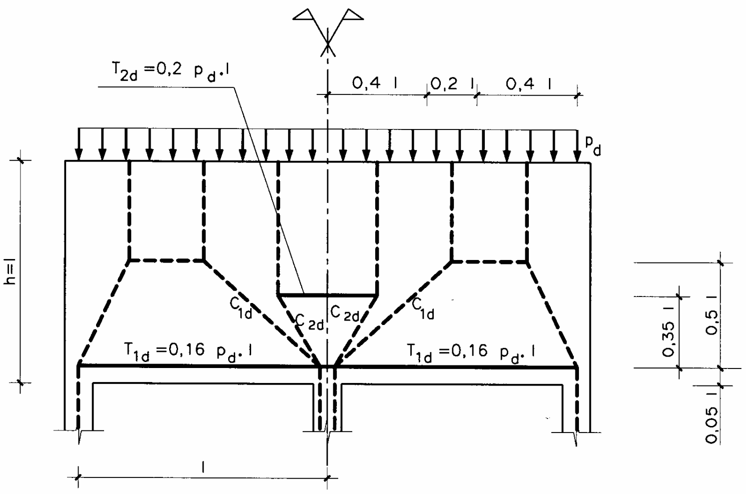
$f_{2cd}$ Design value of concrete cylinder compressive strength.

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

## 62.4 Continuous deep beams

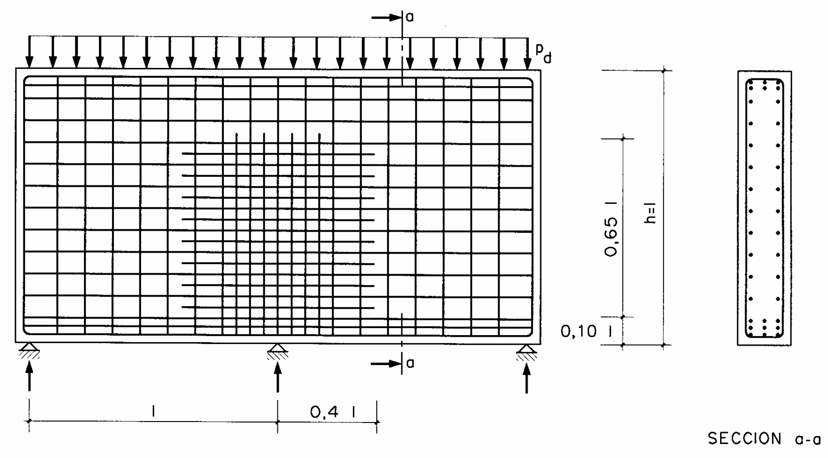
In the case of a uniformly distributed load applied on the upper surface, the model is the one described in figures 62.4.a and b.

Figure 62.4.a




a) MODEL

a) MODELO

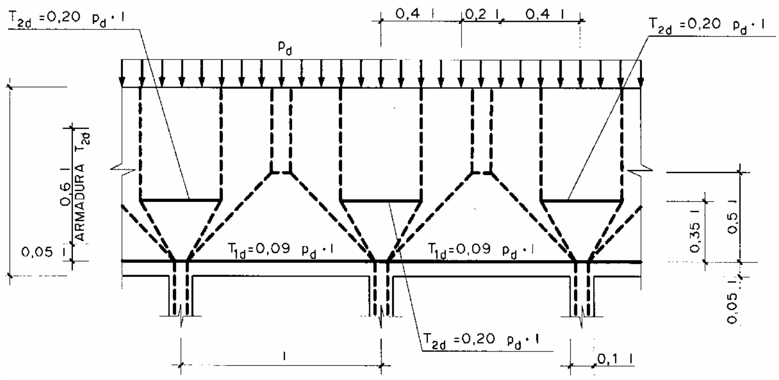



b) ARMADURA

b) REINFORCEMENT

SECTION

Figure 62.4.b




## 62.4.1 Dimensioning the reinforcement

In accordance with the above models, the reinforcement in the zone of intermediate supports should be designed for a tensile force of:

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

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

The lower reinforcement in end bays should be designed for a force equal to:

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

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

The lower reinforcement in intermediate bays should be designed for a force equal to:

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

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

In addition to the main reinforcement described in the previous paragraph, a minimum of 0.1% reinforcement should be placed in each direction and in each face of the element.

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.

Where required, additional reinforcement may be distributed in the supports in accordance with Article 60.

### 62.4.2 Verifying nodes and struts

When verifying nodes and struts, it is sufficient to check the compression at the supports.

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

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

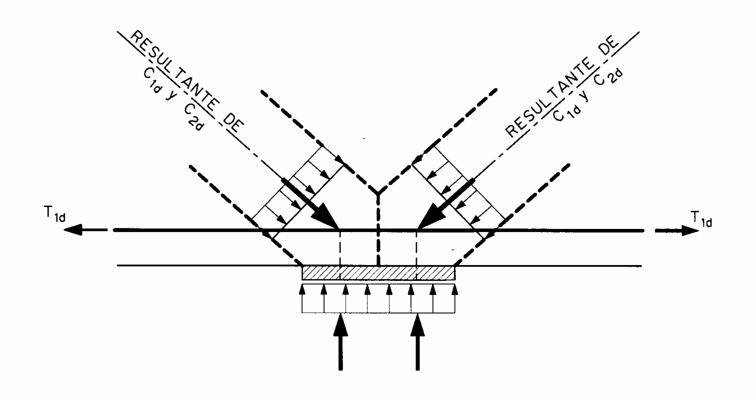
where:

| | |
|---|---|
| $R_{ed}$ | Design reaction at an end support. |
| $R_{id}$ | Design reaction at an inner support. |
| $a_e$ , $b_e$ | Dimensions of the end support (figure 62.3.1.b). |
| $a_i$ , $b_i$ | Dimensions of the inner support (figure 62.4.2). |
| $f_{2cd}$ | Concrete strength for bi-axial compression states. |

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

Figure 62.4.2

RESULTANT



# Article 63 Corbels

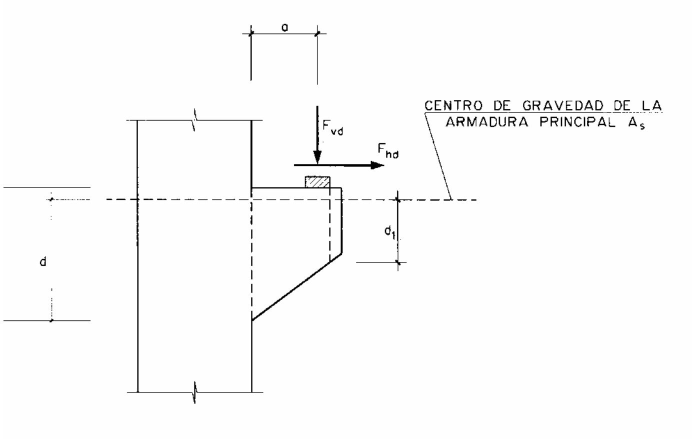
## 63.1 Definition

Corbels are defined as short cantilever beams where the distance *a* between the line of action of the main vertical load and the section adjacent to the support is less than, or equal to, the effective depth *d* at that section (figure 63.1).

The effective depth $d_1$ as measured at the outer edge of the area where the load is applied, should be equal to, or greater than, 0.5*d*

Figure 63.1

PRINCIPAL REINFORCEMENT CENTROID




**COMMENTS**

A corbel is a static type (concentrated loads)and geometric type (sharp changes in the geometry of the member) “D” region.

## 63.2 Verifying elements and reinforcement dimensioning

Since it is a D region, the general method of analysis is the one described in Article 24.

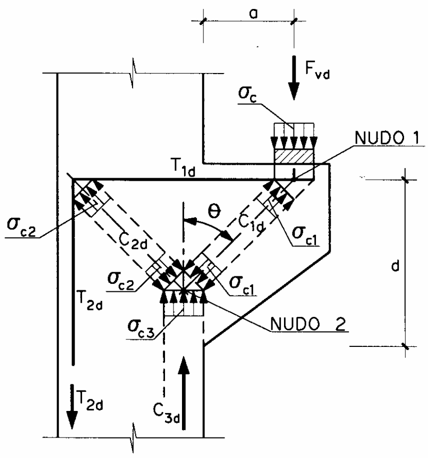
The verifications of struts, ties and nodes and the properties of the materials to be taken into consideration are those established in Article 40.

**COMMENTS**

The general method described in Articles 24 and 40 allows the specific case of figure 63.2 to be tackled in addition to the favourable or unfavourable effects of the forces existing in the section of column located above the cantilever.

Figure 63.2

NODE




The application of the general method is always desirable since it enables the general operation of the "D" region to be controlled, together with its connection to the adjacent "B" regions.

## 63.2.1 Verifying nodes and struts and the reinforcement design

The equivalent strut-and-tie model could be that described in figure 63.2.

The slope angle $\theta$ of the diagonal compressions (struts) may adopt the following values:

cotg $\theta$ = 1.4 if the corbel is monolithically concreted with the column.
cotg $\theta$ = 1.0 if the corbel is concreted onto the hardened concrete of the column.
cotg $\theta$ = 0.6 as for the previous case if the hardened concrete surface has a low level of roughness.

The effective depth *d* of the corbel (figures 63.1 and 63.2) should meet the following condition:

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

### 63.2.1.1 Dimensioning the reinforcement

The main reinforcement $A_s$ (figure 63.2.1.1) should be dimensioned for a design tension of:

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

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

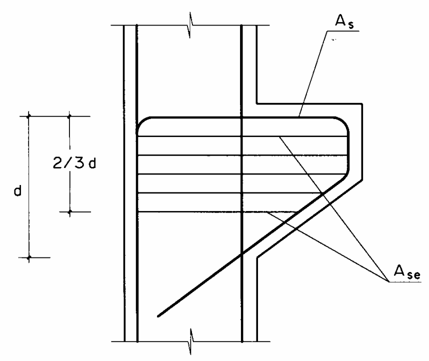



Figure 63.2.1.1

Horizontal stirrups ($A_{se}$) should be evenly distributed in order to resist a total stress of:

$$T_{2d} = 0{,}20\, F_{vd} = A_{se}\, f_{yd}$$

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

#### 63.2.1.2 Verifying nodes and struts

Where the geometric conditions described in 63.2.1 are met, it is sufficient to verify the compression at the support (node 1, figure 63.2).

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

where:

$b$, $c$ Support plan dimensions.
$f_{1cd}$ Concrete strength for bi-axial compression states.

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

**COMMENTS**

The expression given in the article is only valid if $F_{hd} \leq 0.15\, F_{vd}$. If this is not the case, then the node should be verified using the general method as described in Articles 24 and 40.

#### 63.2.1.3 Reinforcement anchorage

Both the main reinforcement and the secondary reinforcement should be suitably anchored at the end of the corbel.

### 63.3 Suspended loads

If a corbel is subjected to a suspended load by means of a beam (figure 63.3.a), various strut-and-tie systems should be studied, in accordance with Articles 24 and 40.

In all cases, horizontal reinforcement should be distributed close to the upper face of the corbel.

**COMMENTS**

The case of corbels subjected to suspended loads may be approached by means of the following simplified manner:

It is assumed that a load fraction of $F_{vd}$ equal to $0,5F_{vd}$ is acting as if applied to the upper part of the cantilever. Another load fraction $F_{vd}$ equal to $0,6F_{vd}$, is assumed to be acting on the lower part of the cantilever. A model of the type described in fFigure 63.3.b is employed to calculate the required reinforcement.

In all cases, secondary reinforcement should be distributed as described in 63.2.1.1.3, together with the suspended vertical reinforcement that is necessary to guarantee transmission of load $0.5F_{vd}$ to the upper part of the cantilever.

The values of 0.5 and 0.6 defined for the load fraction operating on the upper and lower parts, are approximate values.

Figure 63.3.c shows one possible reinforcement distribution.

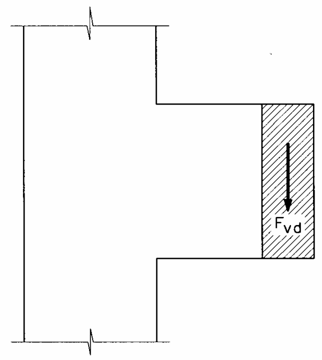



Figure 63.3.a

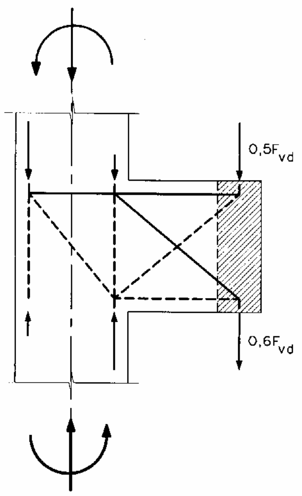



Figure 63.3.b

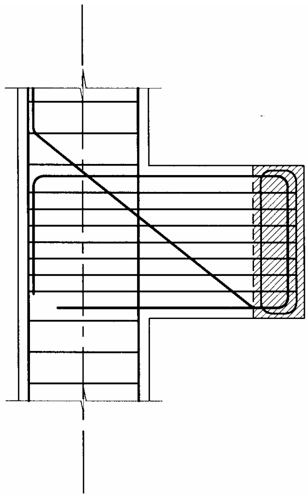
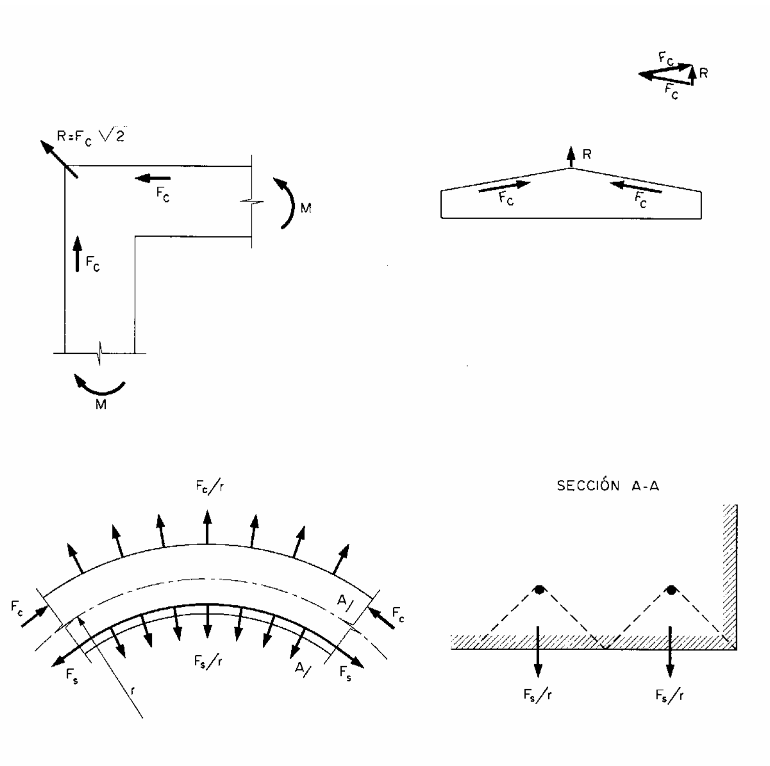


Figure 63.3.c

## Article 64 Members 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).

Figure 64




The binding reinforcement may be designed in general terms based on the indications described in Articles 24 and 40.