ANNEX 10

Special requirements recommended for structures subject to seismic actions

1 Scope

This Annex sets out the special requirements which are recommended for structural concrete structures subject to seismic actions, in addition to the provisions laid down in the specific regulations on earthquake-resistant construction which apply depending on the type of structure concerned (Earthquake-Resistant Construction Standard NCSE-02 General part and buildings, the NCSE-Bridges or the Guidelines on actions to be taken into account for road bridges - IAP).

Seismic action must be defined as indicated in the applicable earthquake resistance regulations. As a general rule, this will involve elastic response spectra. During a strong earthquake, it is expected that the structure will enter a non-linear range which may dissipate part of the energy introduced by the earthquake. Accordingly, the response spectra to be taken into account in the design may be substantially modified, bearing in mind the structure’s capacity to behave in a ductile manner, i.e. to work within a non-linear range of behaviour without any significant loss of strength.

Standard NCSE-02 lays down the following levels of ductility: Very high (μ=4), High (μ=3), Low (μ=2) and No ductility (μ=1). Corresponding to these levels of ductility are behaviour factors (factors used to reduce the elastic spectrum) which may be treated differently in the various seismic regulations, although they are all completely equivalent.

The level of ductility of a structure depends on the structural type, materials, geometric characteristics, regularity in plan and elevation of the masses and distribution of the load bearing elements. Furthermore, the use of structural and construction details is important as these guarantee adequate confinement of the concrete in the zones where the formation of plastic hinges may be expected, prevent buckling of the longitudinal reinforcements in the compression zone and improve the ductile fracture characteristics of critical sections. In high seismicity zones, the use of the "Capacity-based design" philosophy is recommended through which the fracture mode of the structure is controlled, thereby ensuring that, in all cases, the location of critical zones subject to ductile fracture is guaranteed and avoiding these in zones with ductile fracture modes (failures due to shear, torsion, axial compression forces, etc.). This Annex sets out recommendations on construction details, arrangement of reinforcements and design criteria for concrete structures which are appropriate for seismic zones.

For the purposes of earthquake behaviour, it is recommended that the structural types, construction details, etc. which provide the structure with the highest possible ductility are used, particularly if the calculated seismic acceleration is high.

2 Basis of design

2.1 Fundamental requirements

The basis of design for structures subject to seismic actions are as laid down in Title1, Basis of design, of this Code. In Article 13, Combination of Actions, the combination of seismic action with other actions is regarded as a special accidental situation defined as a seismic situation.

Those values indicated in the various action rules will be taken as the representative quasi-permanent values of the variable actions, $\psi_{2,i}Q_{k,i}$. For the purpose of calculating the masses acting during the seismic action, the fraction corresponding to the overload indicated in the applicable seismic regulations or that corresponding to the quasi-permanent value of the overload, $\psi_{2,i}Q_{k,i}/g$, must be included.

2.2 Definitions

Ductility:

Capacity of materials and structures to deform in a non-linear range without suffering any substantial deterioration of the loadbearing capacity. In structural terms, this is defined as the ratio between the ultimate strain at failure and the plastic deformation and may be related to any kinematic reinforcement ratio such as deformation properly speaking, ductility of sections, rotations or displacement of a structure.

Capacity-based seismic design:

Seismic design philosophy in ultimate limit states which is based on protecting the fragile elements and regions of the structure, thereby giving them adequate overstrength with regard to the ductile elements and improving the ductile fracture mechanisms.

Coupled core walls:

Structural element formed of two or more core walls connected in a regular manner at height via coupling beams which have sufficient stiffness to reduce by at least 25% the sum of the fixed-end moments of all the core walls if these were separate.

Plastic hinge:

Area of a structural element where the tension reinforcement has plasticised and where energy may be dissipated through plastic deformation of this reinforcement.

Critical zone:

Region of a primary seismic element where the worst load combinations occur and where a plastic hinge may form.

2.3 Partial safety factors for materials

The partial safety factors for materials, γ_c and γ_s, must take account of the possible deterioration of materials due to cyclic deformations. If there is no detailed information on this aspect, values of γ_c and γ_s corresponding to the persistent or transient situation must be adopted. If the deterioration effect on the strength is explicitly taken into account, the values corresponding to the accidental situation may be used.

2.4 Primary and secondary elements

It is possible to designate certain structural elements as secondary in terms of the earthquake resistance system. These elements will not be regarded as part of the structural system for withstanding seismic actions and therefore do not have to comply with special detailing such as that indicated in section 6 of this Annex.

However, these elements must be dimensioned, according to the capacity-based design criteria, to support the corresponding gravitational load taking into account the maximum displacements produced during the most unfavourable seismic action and bearing in mind the second-order effects. Any structural element which is not dimensioned as secondary must be regarded as primary and, therefore, must be dimensioned to withstand the seismic action and must comply with the details required for the chosen degree of ductility.

The lateral stiffness of all secondary elements must not exceed 15% of that of all primary elements.

For the purposes of the seismic calculation, the stiffness and strength of secondary elements must be ignored. However, the mass of these must be taken into account.

3 Materials

In order to guarantee structural behaviour with high ductility, high-ductility weldable (SD) steels must be used. The characteristics of these are set out in Article 32.2 of this Code.

The use of smooth bars is not permitted. Bars must meet the bond requirements, minimum mechanical characteristics, fatigue characteristics and high-amplitude cyclic load characteristics mentioned in the main articles.

The concrete used must have sufficient compressive strength. The strain at failure of the concrete (ε_u) must exceed the strain under maximum stress (ε_0) by an adequate margin.

If high-strength concretes are used, it must be borne in mind that these have ultimate strain values lower than conventional concretes. In this case, the ductile fracture of the cross-sections must be guaranteed in the calculation by using compression reinforcement providing the appropriate level of ductility.

The strength and ultimate strain of the concrete may be increased by incorporating transverse confinement reinforcement. The strength of confined concrete may be determined using Article 40.3.4 and the peak and ultimate strains (ε_{cc0} and ε_{ccu}) of confined concrete may be determined using the following expressions:

$$\varepsilon_{cc0} = \varepsilon_{c0}\left[1 + 5\left(\frac{f_{ccd}}{f_{cd}} - 1\right)\right]$$

$$\varepsilon_{ccu} = \varepsilon_{cu} + 0{,}1\alpha\,\omega_w$$

where α and ω_w are the parameters defined in Article 40.3.4.

4 Structural analysis

4.1 Calculation methods

The structural analysis methods to be used to study the effects of seismic action are:

Linear methods:

- Modal spectral analysis using a standard response spectrum.
- Equivalent static analysis.

Non-linear methods:

- Non-linear dynamic calculation in the time domain, using a series of accelerograms representing the zone.
- Non-linear static method or incremental pressure method.

In principle, all these methods are applicable to structural concrete structures bearing in mind the requirements and comments in Title 2: Structural analysis. The specific application criteria for each method must be consulted in the applicable seismic regulations.

When considering a ductile behaviour for the structure, the second-order effect caused by the deformations, assessed bearing in mind the deterioration in stiffness suffered by the structure, must particularly be checked.

The stiffness conditions of a structure and, as a result, the stresses induced by the seismic action can vary considerably due to the influence of non-structural elements such as partitions or enclosing walls. The model used for the stress analysis must take account of this effect and the design must define all the details required to guarantee that the collaboration or non-collaboration of these elements in terms of the loadbearing capacity of the structure will be reproduced in the structure, as anticipated in the design.

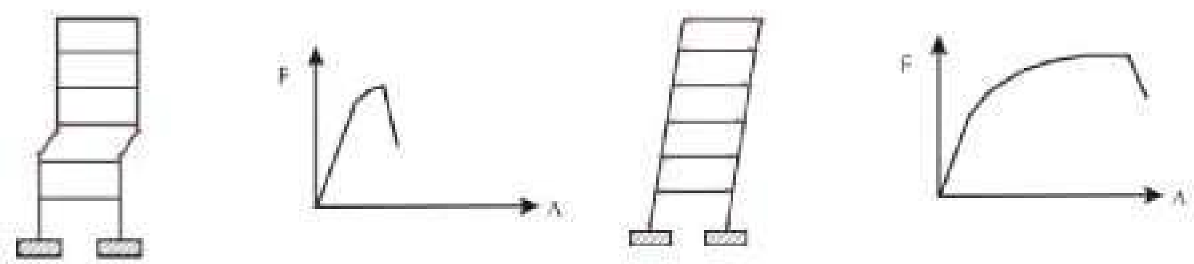
5 Considerations on the ultimate limit states

5.1 Capacity-based design

During major seismic actions, the capacity to dissipate energy possessed by structures with ductile behaviour reducing the stresses which the elements must withstand is normally used. In this way, at a reasonable cost, the collapse of the structure can be avoided and the lives of the occupants of the structure can be saved. It must be borne in mind that this practice involves accepting significant damage to the structure and therefore a non-linear response which will produce different stresses from those predicted in the elastic calculation.

The capacity-based design criterion is intended to prevent the occurrence of fragile fracture modes which can prevent the structure from behaving correctly, such as transforming the structure into a premature mechanism causing a collapse. The following are some of the effects to be avoided:

- Compression fracture in concrete sections without reaching a plastic stage of the tension reinforcements.
- Fracture by shear fracture or primary torsion.
- Fracture of joints between elements or nodes in frames with rigid nodes
- Plasticisation of the foundations or any element which should remain within the elastic range.
- Buckling failures.
- Concentration of plastic hinges on one storey of a multi-storey structure (see Figure A.10.1).
- Etc.



Undesirable fragile behaviour

Desirable ductile behaviour

Figure A.10.1

To avoid undesirable failure modes, the design actions of the elements must be determined using equilibrium conditions, by isolating the element or zone of the structure to be protected from premature failure. The formation of the plastic hinges anticipated in the critical zones is then assumed, taking into account the possible material overstrength factors. The isolated zone must withstand, using the limit state criterion and the corresponding partial safety factors, the stresses deriving from this situation.

Note that, using this criterion, the region or element dimensioned for the stresses thus determined is stronger than the plastic hinges assumed to form at its ends which behave in a ductile non-linear manner and whose plasticisation when subject to a major earthquake is desired. In this way it is guaranteed that the plastic hinge can develop and deform during the seismic action, with the fragile region thus maintaining essentially elastic behaviour.

Below, rules will be given for determining the design stresses in any structural elements according to the capacity-based design criterion.

5.1.1 Shear stress in beams

The shear fracture of beams must be prevented as this can stop the whole ductile bending behaviour of the element from developing. To this end, the design shear stresses, for beams supporting a distributed gravitational load, must be determined based on the system indicated in the following figure. The element is isolated and it is assumed that the end sections have plasticised, thereby forming plastic hinges at the joints; the sign of the stress at each end must be taken into account according to the possible directions of the seismic action (Figure A.10.2).

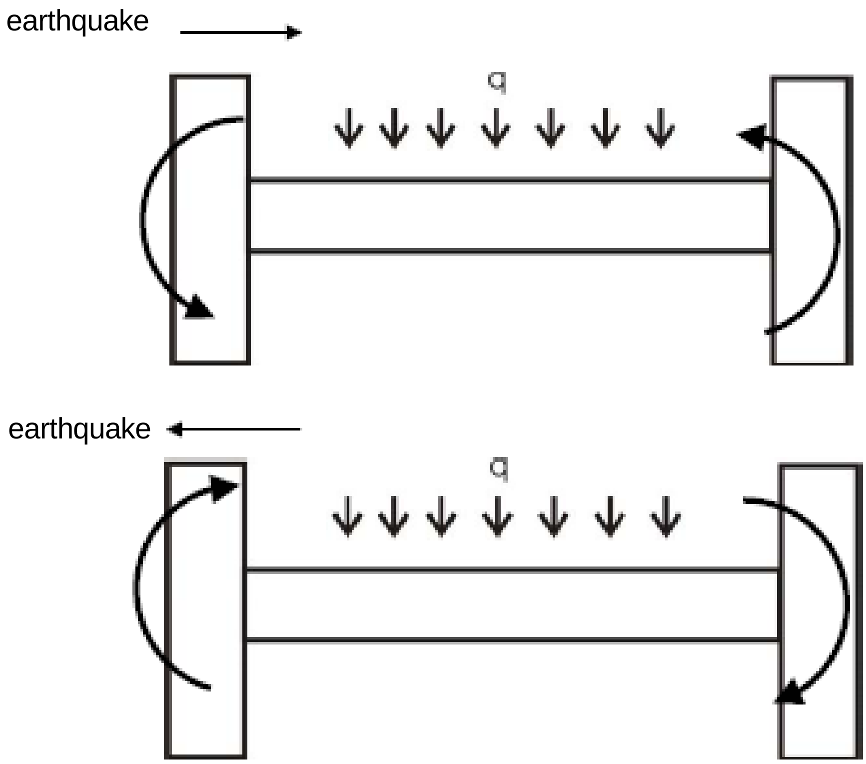


Figure A.10.2

The design shear stress will be the greater of the following possible situations:

$$V_{d1} = \frac{qL}{2} + \gamma_{SR} \frac{\left(M_u^{1-} + M_u^{2+}\right)}{L}$$

$$V_{d2} = \frac{qL}{2} + \gamma_{SR} \frac{\left(M_u^{1+} + M_u^{2-}\right)}{L}$$

where:

q	Distributed load which the beam must support during the earthquake.
L	Clear span of the beam.
M_u^{1+}, M_u^{2+}	Positive resisting bending forces in the end sections of the beam.
M_u^{1-}, M_u^{2-}	Negative resisting bending forces, as an absolute value, in the end sections of the beam.

γ_{SR}	Overstrength factor for the end moments with a value of 1,35. This parameter takes account of the actual strength of the steel, bearing in mind the plastic hardening.

5.1.2 Bending moments in supports

To prevent fracture modes such as those indicated in Figure A.10.1 in multi-storey structures, it must be guaranteed, at the beam-column joints, that the plastic hinges form in the beams instead of in the supports. This requirement must be met on all storeys apart from the top storey.

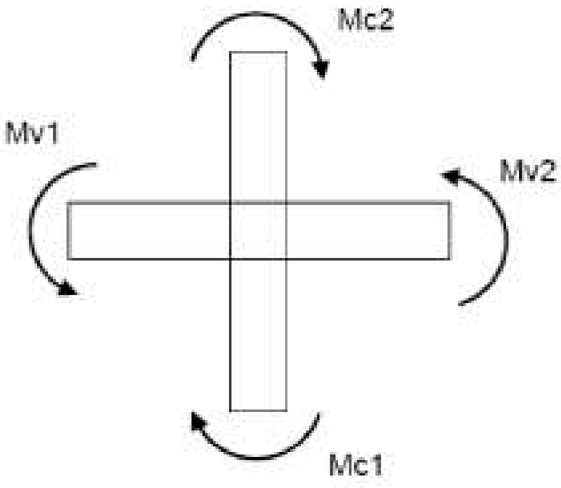


Figure A.10.3.

This requirement is regarded as met if, for each studied direction of the seismic action, it is confirmed that the sum of the ultimate moments in the columns is higher than the sum of the ultimate moments in the beams:

$$\Sigma \, Mu \geq \gamma_{SR} \, \Sigma \, Mu$$

where

γ_{SR} is the overstrength factor with a value of 1,35.

In the above check, the maximum and minimum values which may be taken by the axial force of the supports under the seismic action must be taken into account.

5.1.3 Shear stress in supports

The shear fracture of supports must be prevented and it must be guaranteed that, if the supports fracture, this is due to bending. The design shear stress may be determined for these elements by using similar criteria to that indicated in paragraph 5.1.1, bearing in mind that there is no distributed load in this element and the value of the corresponding axial force. The overstrength factor may be taken as 1,35 for structures with high ductility or as 1,2 for other cases.

5.2 Failure limit state under shear stress

Linear elements cannot be designed without shear reinforcement.

The concrete's contribution to the shear strength *(V_{cu})* is reduced according to the level of ductility required for the section. It is therefore recommended that the independent part of the axial force in the equation corresponding to V_{cu} from Article 44.2.3.2.2 is changed as follows:

$$[\, V_{cu} = \left[\frac{0{,}15\kappa}{\gamma_C} \xi \, (100 \, \rho_l f_{ck})^{1/3} + 0{,}15\alpha_l \, \sigma'_{cd} \right] \beta b_0 \, d$$

where the coefficient κ affecting the term 0,15/γ takes the following values:

- Low or moderate ductility structures: 0,8
- High ductility structures: 0,5
- Very high ductility structures: 0,2

6 Structural details of primary elements

6.1 General

Set out below are certain reinforcement arrangement and dimensional requirements which ensure high ductility behaviour for the various magnitudes of the seismic action, according to the available experimentation and the actual behaviour of structures subject to earthquakes.

The requirements for minimum dimensions or maximum quantities are generally established to prevent excessive concentration of reinforcements or inadequate execution of the zones with greatest structural responsibility.

The requirements for longitudinal reinforcements, as regards minimum quantities in sections and distribution throughout the element, are established bearing in mind, in the main, the reversibility of moments and the change in the stress laws throughout the element due to the assumed non-linear behaviour.

The requirements for transverse reinforcements are primarily established in order to confine the compressed concrete, prevent buckling of the compression reinforcement and increase the shear strength.

Finally, the general criteria for the anchoring conditions are established to take account of the deterioration in these loadbearing characteristics due to the action of alternating cyclic loads.

6.2 Beams

This section covers elements which fundamentally work by bending and which meet the following conditions:

- The reduced design axial compressive force, due to the seismic situation, complies with:

$$\frac{N_d}{A_c f_{cd}} \leq 0{,}10$$

- The width/depth ratio will not be less than 0,3.
- The span of the beam will not be less than four times the effective depth of the element.
- If there is a concrete top slab, the effective width of this will be as defined below. The reinforcement of the slab contained within this width forms part of the upper reinforcement of the beam and must therefore be taken into account for the purposes of the permitted maximum reinforcement ratio and the calculation of the shear stress by capacity-based design criteria.

    - For external beam-column nodes without transverse beams, this will be the column width.
    - For external beam-column nodes with transverse beams, this will be the column width plus double the slab depth on each side of the beam where there is a slab.
    - For internal beam-column nodes, the above widths may be increased to double the slab depth.

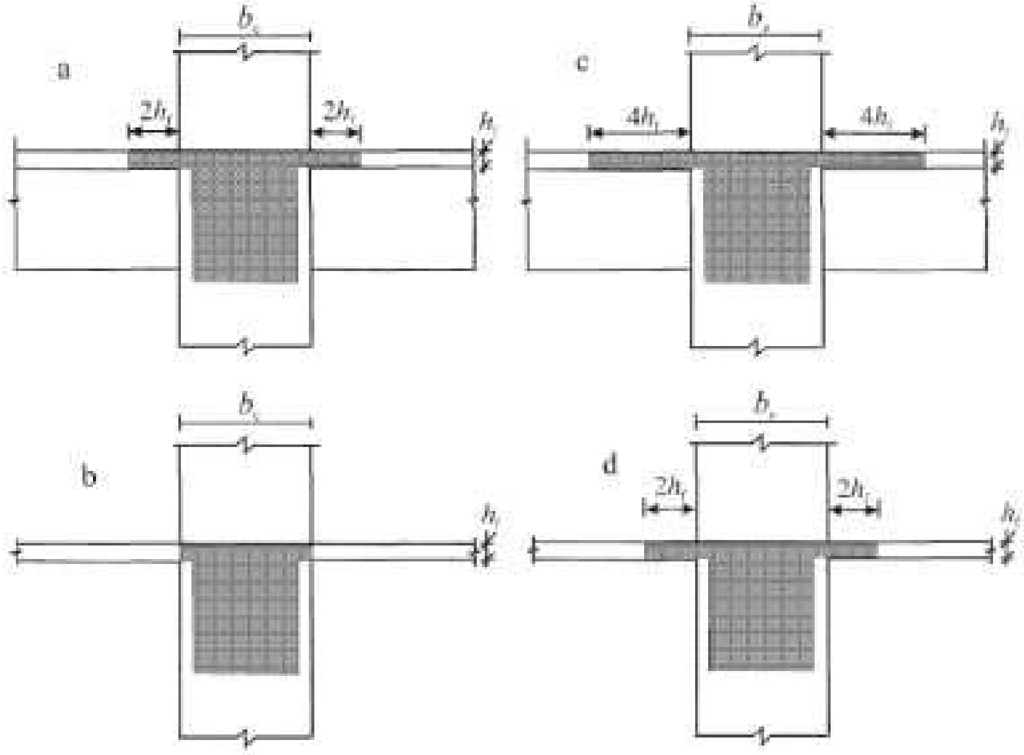


Figure A.10.4

With regard to the anchorage and overlap of reinforcements, the following indications shall be observed:

- The anchorage lengths of the reinforcements shall be increased by 10Ø with regard to those defined for static loads as indicated in the main articles of this Code for cyclic loads.
- The joints of the reinforcements will, as far as possible, be kept away from the zones close to the ends, by a length of double the beam depth, or from the zones where the formation of plastic hinges is anticipated.

The length of critical zones or those where plastic hinges are likely to form must be taken as:

- For frames with rigid nodes, double the beam depth measured from the face of the supporting elements to halfway along the span.
- Double the camber of the element on both sides of a section where the steel may plasticise under seismic loading conditions.
- For beams which support significant point loads, the zone situated directly below the load and double the beam depth on both sides of this.

Structural systems involving discontinuous pillars supported on lateral beams are not recommended in seismic areas. In all cases, these beams must be dimensioned with special care and the capacity-based design rules must be used. The vertical component of the seismic accelerations must be included in the structural analysis.

6.2.1 High ductility

General provisions for the whole beam:

- Beams must be set back from the slab edge. This set-back must be greater than the depth of the neutral fibre in the support zone under the negative failure moment. The width of the set-back must be at least 200 mm.
- Along its whole length there must be a longitudinal reinforcement of at least 2ϕ14 or 25% of the maximum reinforcement ratio of negative reinforcement in any section between supports. In all cases, the minimum quantity set in the main articles of this Code must be respected.
- The maximum reinforcement ratio under tension in any section of the beam shall be less than:

$$\rho_{\max} = \rho' + 72 \frac{f_{cd}}{f_{yd}^2} [MPa]$$

- Transverse reinforcement of at least ϕ6 will be provided in the form of closed hoops set along the whole length of the beam. Their spacing will be no more than h/2.

Provisions to be met in critical zones of the beam where a plastic hinge may form:

- The compression reinforcement will be at least 50% of the tension reinforcement placed in the same section.
- The transverse reinforcement will be at least ϕ6 in the form of closed hoops. In the support zone, the first transverse reinforcement must be placed 50 mm from the support. The maximum spacing of this reinforcement must be less than:
    - d/4
    - 6 times the smallest diameter of the longitudinal reinforcement.

- 24 times the diameter of the hoop reinforcement.
- 200 mm

6.2.2 Very high ductility

General provisions for the whole beam:

- Beams must be set back from the slab edge. This set-back must be greater than the depth of the neutral fibre in the support zone under the negative failure moment. The width of the set-back must be at least 250 mm.
- Along its whole length there must be a longitudinal reinforcement of at least 2ϕ14 or 33% of the maximum reinforcement ratio of negative reinforcement in any section between supports. In all cases, the minimum reinforcement ratio set in the main articles of this Code must be respected.
- The maximum reinforcement ratio under tension in any section of the beam shall be less than:

$$\rho_{\max} = \rho' + 50 \frac{f_{cd}}{f_{yd}^2} \left[N/mm^2 \right]$$

- Transverse reinforcement of at least ϕ6 will be provided in the form of closed hoops set along the whole length of the beam. Their spacing will be no less than h/2.

Provisions to be met in critical zones of the beam where a plastic hinge may

- The compression reinforcement will be at least 33% of the tension reinforcement placed in the same section.
  - The transverse reinforcement will be at least ϕ6 in the form of closed hoops. In the support zone, the first transverse reinforcement must be placed 50 mm from the support. The maximum spacing of this reinforcement must be less than:
  - d/4
  - 8 times the smallest diameter of the longitudinal reinforcement.
  - 24 times the diameter of the hoop reinforcement.
  - 200 mm

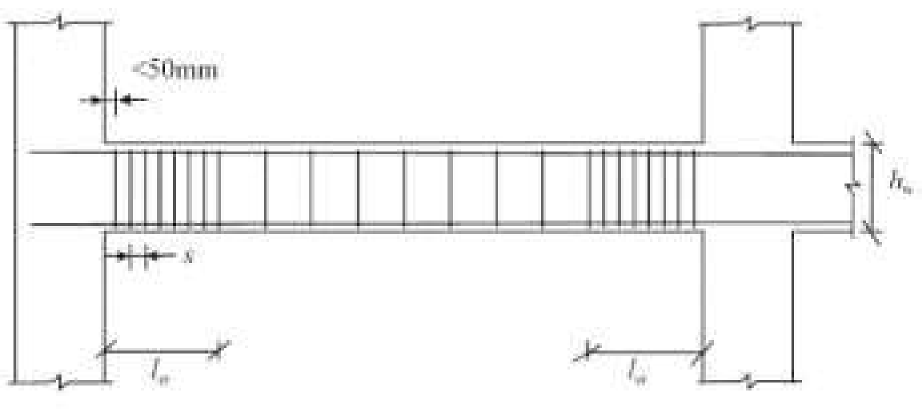


Figure A.10.5

6.3 Supports

This section covers elements which fundamentally work by compound compression and which meet the following conditions:

- The reduced design axial compressive force, due to the seismic situation, is:

$$\frac{N_d}{A_c\, f_{cd}} \geq 0{,}10$$

- The supports forming part of the primary earthquake resistance system, designed with a level of ductility other than essentially elastic, must meet the following condition for the design axial force:

$$\frac{N_d}{A_c\, f_{cd}} \leq 0{,}65$$

- The ratio between the largest and smallest dimensions of the rectangle in which the cross-section is inscribed must not exceed 2.5.

With regard to the anchorage and overlap of reinforcements, the following indications shall be observed:

- The anchorage lengths of the reinforcements shall be increased by 10Ø with regard to those defined for static loads as indicated in the main articles of this Code for cyclic loads.
- The joints of the reinforcements will, as far as possible, be kept away from the zones close to the ends or from the zones where the formation of plastic hinges is anticipated.

The zones contained within the plastic hinge lengths at both ends of a column must be regarded as critical zones. In the absence of more precise information, the length of the plastic hinges shall be taken as the maximum of the following values:

- the maximum dimension of the column cross-section
- 1/6 of the free length of the column
- 450 mm

If the free length of the column is less than 3 times the largest dimension of its cross-section, the whole column must be regarded as a critical zone and must comply with the corresponding minimum structural details.

6.3.1 General provisions

These provisions apply to any column forming part of a primary earthquake resistance system designed with a type of behaviour better than essentially elastic.

The reinforcement ratio of longitudinal reinforcement must not be less than 1% or more than 6%. If the cross-section is symmetrical, longitudinal reinforcement which is also symmetrical must be used.

The longitudinal reinforcement will consist of at least three bars in each face. In the case of circular sections, at least six bars in total must be used.

The transverse reinforcement will consist of closed hoops and, where applicable, additional bands of at least ϕ6. The arrangement of the transverse reinforcements will be such that this ensures effective confinement of the cross-section.

Throughout the critical zones there must be a minimum mechanical reinforcement ratio of transverse reinforcement with a value of:

$$\omega_{W,\ min} = 0{,}08$$

Outside the critical zones, transverse reinforcement of at least ϕ6 must be used with a spacing of not more than 15 times the diameter of the smallest longitudinal reinforcement or 150 mm.

In high or very high ductility structures, the provisions indicated below must also be observed.

6.3.2 Provisions for high ductility

The minimum section of the cross-section shall be 250 mm.

The maximum reinforcement ratio of longitudinal reinforcement shall be 4%.

The distance between longitudinal reinforcements shall not exceed 200 mm. Along the whole length of the column, transverse support must be provided for the longitudinal reinforcements using additional hoops or hooks, at least alternately and at the corner bars.

In critical zones where a plastic hinge may form, a reinforcement ratio of transverse reinforcement must be provided which is equal to or greater than:

$$\omega_{W,\min} = \frac{1}{\alpha}\left(\frac{\nu_d f_{yd}}{1333}\frac{b_c}{b_0} - 0{,}035\right)$$

where

$$\nu_d = \frac{N_d}{A_c f_{cd}}$$

b_c Width of the cross-section

b_0 Width of the confined core (measured between the central lines of the confining hoops).

α Confinement effectiveness factor, defined in Article 40.3.4 of this Code.

The maximum spacing between transverse reinforcements in critical zones shall be the smallest of the following values: $b_0/3$, 150 mm or 8 times the diameter of the smallest longitudinal reinforcement.

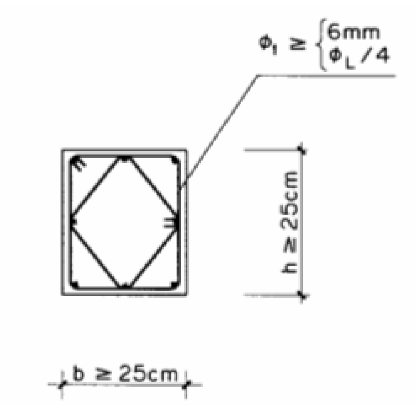


Figure A.10.6

6.3.3 Provisions for very high ductility

The minimum section of the cross-section shall be 300 mm.

The maximum reinforcement ratio of longitudinal reinforcement shall be 4%.

The minimum diameter of the transverse reinforcement shall be $\phi 8$.

The distance between longitudinal reinforcements shall not exceed 150 mm. Along the whole length of the column, transverse support must be provided for the longitudinal reinforcements using additional hoops or hooks, at least alternately and at the corner bars.

In critical zones where a plastic hinge may form, a reinforcement ratio of transverse reinforcement must be provided which is equal to or greater than:

$$\omega_{W,\min} = \frac{1}{\alpha}\left(\frac{\nu_d f_{yd}}{950}\frac{b_c}{b_0} - 0{,}035\right)$$

where the parameters of the formula have the same meanings as in the previous section.

The maximum spacing between transverse reinforcements in critical zones shall be the smallest of the following values: $b_0/4$, 100 mm or 6 times the diameter of the smallest longitudinal reinforcement.

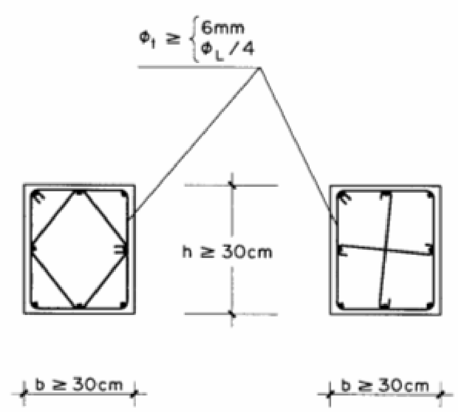


Figure A.10.7

6.4 Nodes

In order to check the coupling conditions, a strut and tie rod model must be used, defined in accordance with the general criteria in Article 24 and with the various elements being checked according to the indications in Article 40.

The beam-column nodes shall be dimensioned to withstand the shear stress determined according to the capacity-based design criteria as indicated in section 5 of this Annex. In addition, transverse reinforcement must be provided in order to ensure adequate confinement of the core to the concrete. This reinforcement shall be parallel to the horizontal reinforcement of the columns.

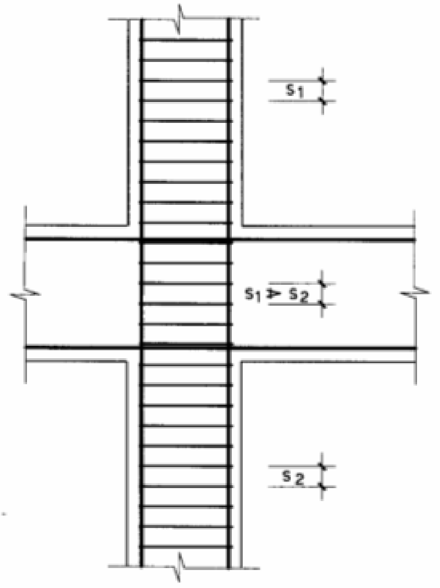


Figure A.10.8

In general, the transverse confinement reinforcement at the coupling shall not be less than that specified for the critical zones of the columns. As an exception, if the coupling receives beams on all four faces and the width of these is at least ¾ of the parallel dimension of the column, the spacing of the confinement hoops may be double that specified above, but never greater than 150 mm.

6.5 Core walls

This section covers highly rigid elements whose fundamental function is to withstand the horizontal stresses produced by the seismic action and which meet the following conditions:

- The minimum thickness of the core wall shall be 150 mm and no more than 5% of the clear height of the floor.
- The main reinforcement shall be placed in both faces.
- Core walls forming part of the primary earthquake resistance system, designed with a level of ductility other than essentially elastic, must comply with the following condition for the design axial force:

$$\frac{N_d}{A_c\, f_{cd}} \leq 0{,}40$$

- The geometric reinforcement ratio of longitudinal reinforcement shall be 4%.
- The stiffness conditions and therefore the dimensions shall not vary significantly through the height.
- Where there are openings, these shall be vertically aligned.
- With regard to the anchorage and overlap of reinforcements, the following indications shall be observed:

- The anchorage lengths of the reinforcements shall be increased by 10Ø with regard to those defined for static loads in the main articles of this Code.
- The groups of box-section core walls connected together on one floor forming L-, T-, U-, double-T or similar sections shall be regarded as integral units formed of webs and flanges.

The effective width of the flanges shall be taken from the edge of the webs over a length no greater than the actual length of the flange, half the distance between adjacent webs or 25% of the total height of the wall above the level in question. In all cases, the reduced axial force mentioned in this section shall be standardised with regard to the web of the cross-section.

The length of the critical zone where a plastic hinge may form shall be taken as the maximum value of the horizontal length of the core wall or the total height of the wall. However, the length of the critical zone shall not exceed double the horizontal length of the core wall, the clear height of the floor for buildings with 6 floors or less or double the clear height of the floor for buildings with more than 6 floors.

Where the reduced design axial force under the seismic action is equal to or greater than 0,15, the following mechanical reinforcement ratio of horizontal confinement reinforcement must be placed in the critical zone:

- in high ductility primary elements

$$\omega_{W,\min} = \frac{1}{\alpha}\left[\frac{(\nu_d + \omega_v) f_{yd}}{1333}\frac{b_c}{b_0} - 0{,}035\right]$$

- in very high ductility primary elements

$$\omega_{W,\min} = \frac{1}{\alpha}\left[\frac{(\nu_d + \omega_v) f_{yd}}{950}\frac{b_c}{b_0} - 0{,}035\right]$$

where:

ω_V Mechanical reinforcement ratio of vertical reinforcement in the web, standardised with regard to the web of the core wall.

This confinement must be placed at the ends of the core wall, in the form of hoops, at a horizontal distance (l_c) measured from the cover of the reinforcements to the point where the unconfined concrete may slip. This distance may be determined as:

$$l_c = x_u\left(1 - \frac{\varepsilon_{cu}}{\varepsilon_{cu,c}}\right)$$

where:

ε_{cu} Crushing strain of the concrete for the corresponding characteristic strength.

$\varepsilon_{cu,c}$ Crushing strain of the confined concrete which can be determined as: $\varepsilon_{cu,c} = \varepsilon_{cu} + 0{,}l\ \alpha\ \omega_w$, where α and ω_w are the parameters defined in Article 40.3.4.

X_u Depth of the neutral fibre at fracture after the unconfined concrete slips. In the absence of a rigorous calculation, this may be estimated as: [see original for equation], where b_0 is the width of the confined core of the core wall.

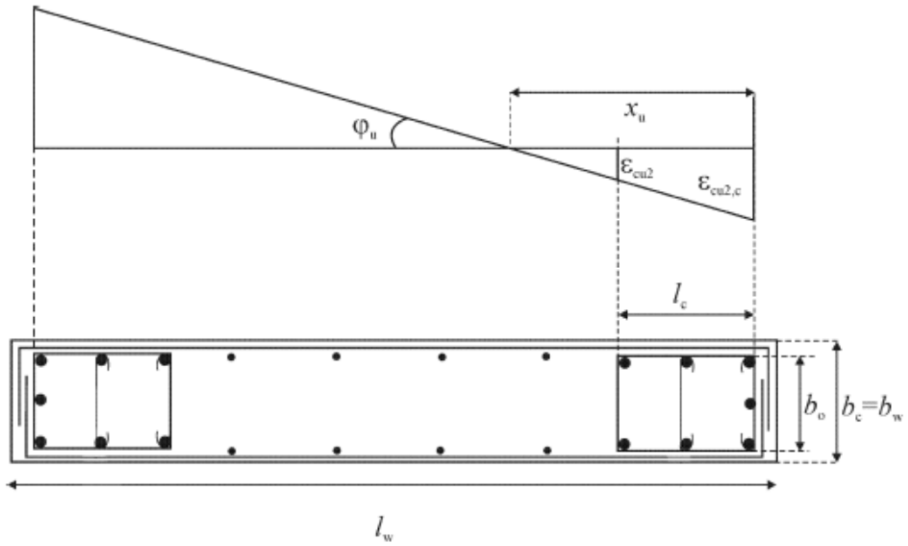


Figure A.10.9

In the confined edge zone, the reinforcement ratio of vertical reinforcement must not be less than 0,005. The thickness of the confined edge zone must not be less than 200 mm in general.

If the length lc does not exceed double the width of the confined zone or 20% of the horizontal length of the wall, the width of the confined zone shall also be more than 10% of the clear height of the floor.

If the length lc does exceed double the width of the confined zone or 20% of the horizontal length of the wall, the width of the confined zone shall also be more than 15% of the clear height of the floor.

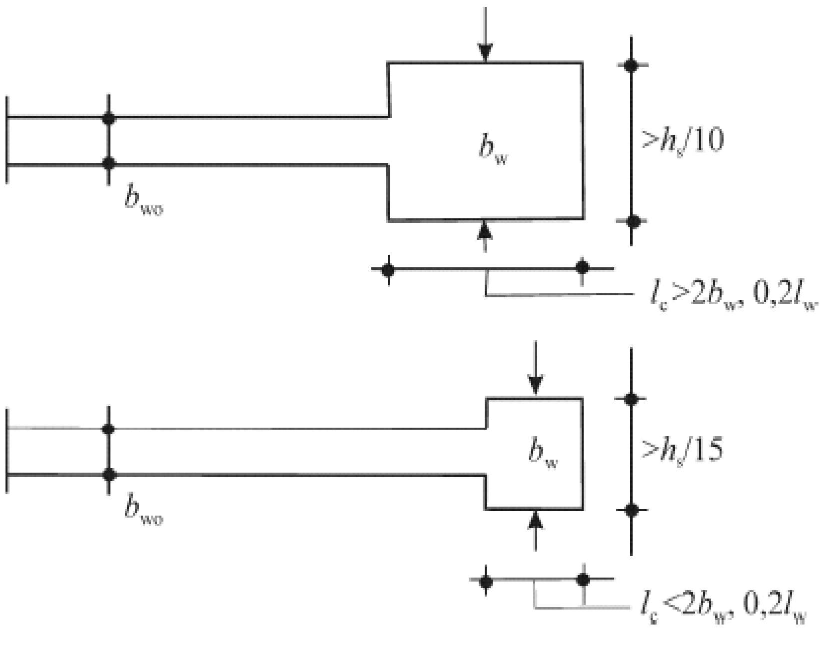


Figure A.10.10

6.6 Connecting elements between coupled core walls

This section covers lintel-type elements or edge beams which join, in their plane, two core walls at different heights. It is understood that these elements are sufficiently rigid to connect the deformation of the core walls in terms of both horizontal displacements and rotation. These elements must be taken into account in the structural model of the core wall. Slabs or floor slabs not contained within the plane of the core wall are not regarded as connecting elements.

These elements may be dimensioned as beams if their length is more than 3 times their depth or where inclined cracking under the seismic action is unlikely, which may be regarded as met if the following condition is satisfied:

$$V_d \leq f_{ctd} bh$$

where:

V_d Design shear stress under the seismic combination.
f_{ctd} Lower design tensile strength of the concrete.

Where the above criteria are not met, these elements must be dimensioned in accordance with the strut and tie rod criteria defined in accordance with the general criteria in Article 24 and with the various elements being checked according to the indications in Article 40.

The reinforcement of these elements must be arranged by forming two diagonals along the beam as shown in the following figure:

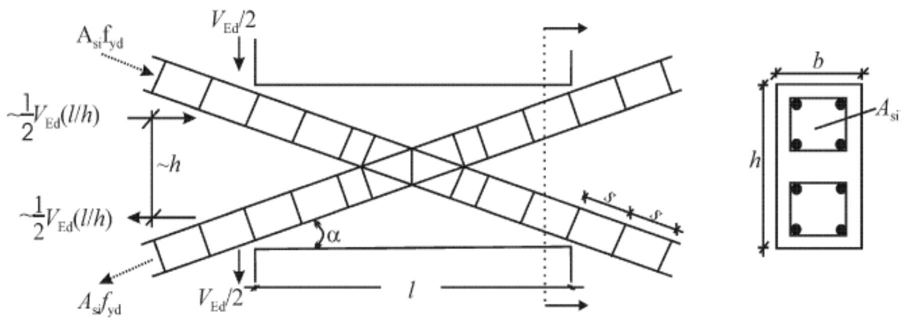


Figure A.10.11

The longitudinal reinforcement in each diagonal must meet the following condition:

$$V_d \leq 2A_{si}\, f_{yd}\, sen\ \alpha$$

where α is the inclination of the diagonals with regard to the horizontal and A_{si} is the area of longitudinal reinforcement in each diagonal.

The diagonals must be reinforced along similar lines to the columns in order to prevent their buckling. The dimension of the diagonal in the plane of the beam shall be at least 50% of the beam width. The anchorage length of the reinforcements must be increased by 50% with regard to that required in the main articles of this Code for static loads. Transverse reinforcement must be provided to prevent buckling of the compression reinforcements, as indicated in section 6.3 of this Annex.

In addition, vertical and horizontal reinforcements must be placed in both faces as laid down in the main articles of this Code for deep edge beams.

6.7 Horizontal diaphragms

Horizontal diaphragms may consist of concrete slabs or the compression layer of one-way or two-way floor slabs where their thickness is greater than or equal to 50 mm, where an intermediate reinforcement is provided in both directions and where an adequate link to the perimeter elements (beams or bands) is guaranteed.

For calculation purposes, diaphragms may be regarded as infinitely rigid elements in their plane, provided that the ratio between the largest and smallest dimensions in plan view is equal to or less than 4. If this ratio is not met throughout the whole floor slab or in any region of this, a more detailed analysis will be needed of the deformability of the floor slab and its effects on the distribution of the seismic action to the primary elements.

Horizontal diaphragms must be dimensioned in accordance with the strut and tie rod criteria defined in accordance with the general criteria in Article 24 and with the various elements being checked according to the indications in Article 40. It must be guaranteed that the diaphragm is capable of distributing the seismic stresses to the primary elements connected via this, by paying attention to the concentration of stresses produced in the opening zone and the possible directions of the seismic action.

The struts must be appropriately confined, using criteria similar to those used for high ductility columns, unless the compression of these is less than $0{,}15\ f_{cd}$ under the design seismic action. If longitudinal reinforcement is required in the compressed struts, appropriate measures must be adopted to prevent buckling of the longitudinal reinforcements as indicated in section 6.3 of this Annex.

For diaphragms formed of precast slabs, the capacity of the longitudinal joints to transmit the shear stress produced in the plane of these must be checked, with the diaphragm being regarded as a beam supported on the elements of the primary system. This shear stress may be withstood using reinforcement which crosses the joint transversely and is anchored in the precast elements (the joint must subsequently be concreted) or using the transverse reinforcement of the site-cast top slab (where this exists).

In the latter case, the top slab must be at least 70 mm thick. The surface of the precast slab on which the top slab is concreted must be rough and clean or there must be shear connectors.

Likewise, the capacity of precast diaphragms to transmit the seismic stresses to the primary elements must be checked.

6.8 Foundation elements

If the design stresses of the foundations are determined using capacity-based design criteria, no significant dissipation of energy in these elements will be expected and therefore a special detailed study will not be required to guarantee a level of ductility. Otherwise, foundation elements must meet the same requirements as indicated above.

In all cases, the solution adopted for the foundations must comply with the following criteria:

- The coexistence of different foundation solutions in one structural unit, understood as the part of the structure separated from the rest by a joint along its whole height, must be avoided.
- If the supporting ground is not generally uniform, the foundations shall be divided into different structural units.
- If liquefaction is likely, superficial foundations must be avoided.
- The end of deep foundations must be below the liquefiable layers.
- Tie elements must be placed under the primary elements in both directions, at the base of tie beams and at the footing height, thus avoiding the formation of short pillars. The minimum dimensions of the tie beams shall be 250 mm for the base and 400 mm for the depth, for structures with up to 3 floors above the basement, or 500 mm for the

depth for higher structures. The axial force which is produced due to the horizontal action must be taken into account.

- If the design acceleration is less than 0,16 g, the tie may be at the base of a foundation slab provided that its depth is at least 150 mm or 1/50 of the distance between pillars.

6.9 Precast elements and joints

Precast beams and supports must meet the requirements indicated in sections 6.1 and 6.2 of this Annex, bearing in mind the actual link between the elements when determining the critical regions.

For frames with rigid couplings, the adequate transmission of moments in positive and negative directions through the joints and embedded supports with an adequate strength must be guaranteed. The design stresses must be determined in accordance with the capacity-based design principles.

If the joint between elements is located within a critical region, this must be over-dimensioned, in accordance with the capacity criteria, in order to guarantee that this does not plasticise, unless it is proven that the joint forms a device with sufficient ductility and energy dissipation capacity and has been considered as such in the design. In all cases, the premature collapse of the joint must be prevented using capacity-based design criteria.

For core walls formed of precast elements, the capacity to transmit the shear stresses produced in the plane of this must be checked using provisions similar to those indicated for horizontal diaphragm joints in section 6.6 of this Annex.

For horizontal diaphragms formed of precast elements, the provisions indicated in section 6.6 of this Annex must be met.

7 Anchorage of reinforcements

Reinforcements must be anchored as indicated in Article 68 of this Code. It should be remembered that, under seismic stresses, the anchorage of reinforcements must be increased by 10ϕ with regard to the value given for static loads.