Seismic design code
– Part I –

Design provisions for buildings
Reference number P 100-1/2025

Beneficiary:

Ministry of Development, Public Works and Administration

December 2024

This document constitutes the second draft of the technical regulation P100-1, revised following the comments received during the public consultation process, the public inquiry, and the endorsement meeting of the Specialized Technical Committees.

The second draft approved by the specialized technical committees of MDLPA will be subject to the notification procedure with the European Commission, according to the provisions of Government Decision No 1016/2004.

The third draft of the work will be subject to approval by the Technical Committee for General Coordination of the contracting authority and will be published by order of the Minister of Public Works, Development and Administration, issued for the approval of the technical regulation, in the Official Gazette of Romania.

Contents:

1 General information

1.1 Purpose and scope

(1) Seismic Design Code - Part I - Design Provisions for Buildings, reference number P 100-1/2025 shall apply to the seismic design of buildings.

(2) Design of engineering constructions that do not fall under the provisions of (1) is not covered by code P 100-1. In the seismic design of these constructions, depending on their specificity, the provisions of Chapter 3 may be applied to establish the seismic design action.

(3) This technical regulation contains provisions relating to the design of buildings, specific to the ‘resistance, mechanical and stability’ quality requirement, laid down by Law No 10/1995 on quality in construction, republished, as amended.

(4) Seismic design of buildings shall be carried out in accordance with the provisions of this technical regulation in construction and the generally accepted principles of structural mechanics.

(5) Due to the random nature of seismic action, the effectiveness of seismic protection measures for buildings presents a moderate degree of uncertainty. The quality of buildings in terms of seismic performance is assessed according to the way in which the provisions of the applicable technical regulations in construction are met and not by analysing the post-earthquake degradation state.

(6) The design seismic action provided for in this technical regulation is conventional and cannot be associated with or compared to a specific seismic event at a site. Acceleration spectra are used to determine seismic load cases and are not necessarily representative of a particular seismic movement.

(7) The provisions of this technical regulation shall apply to the design of new buildings and to the design of intervention works on existing buildings carried out to reduce the susceptibility to damage from seismic actions.

(8) The provisions of this technical regulation may be applied to the design of safety works for buildings classified as historical monuments only if they do not contradict the concepts, approaches and procedures contained in the normative documents specific to this category of buildings.

(9) The provisions of this technical regulations are minimal. The designer can decide to achieve a quality level higher than the minimum requirements imposed by this technical regulation.

(10) The provisions of this technical regulation are addressed to all factors involved in the quality assurance system in construction, according to Law 10/1995, as amended and supplemented, whose activity directly or indirectly influences the way in which the fundamental requirement "mechanical resistance and stability" is achieved and maintained.

(11) The provisions of Code P 100-1 are harmonised with the provisions of National Standard SR EN 1998-1.

(12) This design code is used in conjunction with other technical regulations in construction.

(13) Structural components of buildings are designed for seismic actions based on the provisions of this technical regulation, Romanian reference standards and, where appropriate, other technical regulations in construction which have provisions regarding the design for seismic actions.

(14) Products and technical solutions for which there are no technical design specifications in the normative documents referred to in (13), shall be selected for incorporation into structural components or structures only on the basis of the provisions of the technical approval concerning the suitability for use of the product under conditions of seismic stress, with dynamic actions applied cyclically, compatible with the basic requirements of seismic design given in this technical regulation.

(15) The provisions of this technical regulation reflect the level of knowledge at the time of its elaboration concerning the actions, principles and rules of calculation and composition of constructions, as well as the performance and requirements for construction and construction products that were used.

(16) As theoretical research and experimental programs obtain additional data and information on the behaviour of buildings during earthquakes and the calculation assumptions used, these will form the basis for substantiating technical amendments to this code, in accordance with the law and the procedure for the revision of technical regulations.

1.2 Structure of the code

(17) This technical regulation contains basic requirements, performance requirements, and prescriptive requirements for the design of buildings to seismic actions, structured in chapters and annexes of a regulatory or informative nature.

(18) The buildings constructed in accordance with this technical regulation shall comply with all the provisions of the normative chapters. The requirements that the structures must meet are drafted in the present tense.

(19) By way of exception to (18), this regulation also includes provisions of recommendatory character, based on general engineering practice, which are distinguished by the use of the term ‘is recommended’. The designer may decide, on a case-by-case basis, a different engineering approach, in compliance with all other mandatory provisions.

(20) In interpreting the provisions, the use of the conjunction ‘and’ indicates that all conditions, requirements, articles, objects or events apply. The use of the conjunction ‘or’ indicates that one of the requirements, conditions, items, objects or events applies. The use of the composite conjunction ‘and/or’ indicates that one or more of the requirements, conditions, articles, objects or events apply. The use of the verb ‘may’ in the reflective impersonal form ‘may’ or ‘is possible’ indicate that the designer has the possibility to use the prescribed solution in a provision without making it compulsory.

(21) In the context of this technical regulation, citations are made as follows:

(a) citations referring to provisions within the same paragraph are drawn up by indicating the number of the subparagraph, the calculation relationship, the figure or the table;

(b) citations referring to provisions in other paragraphs of this technical regulation are drawn up by indicating the paragraph number and the subparagraph number, the calculation relationship, the figure or the table;

(c) citations referring to provisions of other technical regulations are drafted by mentioning the reference number of that technical regulation.

(22) The informative chapters contain provisions of recommendatory character, established on the basis of general engineering practice.

(23) The structure of Code P 100-1 is as follows:

1. General information

2. Basic requirements

3. Design seismic action

4. Seismic design

5. Concrete structures

6. Steel structures

7. Composite structures

8. Masonry structures

9. Timber structures

10. Non-structural components

11. Seismic devices

Annex A - Seismic action. Definitions and additional provisions.

Annex B - Comments

(24) Chapters 1-11 and Annex A are regulatory in nature. Annex B is for information purposes only.

1.3 General definitions

(25) The definitions of the terms specific to the design of buildings for seismic action used in this technical regulation in construction are:

Building: above-ground and, where appropriate, underground construction, which serves to shelter people, materials, machinery, or equipment, etc.

Component: immovable part of the structural, architectural or installation system;

Structural component: a component of a building that ensures the balancing of efforts caused by different types of actions acting on it or on other components of the building.

Non-structural component: component of a building attached to the structure, which has only an architectural or functional role.

Main structural component: component of a structure that is designed to balance the efforts caused by seismic loads acting on all categories of building components.

Secondary structural component: a component of a structure designed to withstand efforts caused by actions other than seismic action affecting all categories of building components.

Content: removable items from the building, introduced by users.

Property: one or more adjacent plots of land, with or without buildings, belonging to the same owner.

Duration of the strong part of an accelerogram: the time interval in an accelerogram between the first absolute value of acceleration greater than or equal to 0.05g and the last absolute value of acceleration greater than or equal to 0.05g.

Reversible plastic deformations: plastic deformations that occur as a result of alternating cyclic loads, where the plastic deformations that occur by loading in one direction are mostly or totally compensated by loading in the opposite direction, so that the accumulation rate from one cycle to another is low.

Horizontal diaphragm: structural element ensuring the connection in a horizontal plane between vertical elements.

Indirect effect: variation of the axial force in a vertical structural element connected by rigid horizontal elements and resistant to other vertical structural elements as a result of horizontal actions.

Gaps: cavities of any form in a structural or non-structural element.

Redundancy: the property of a structure to have two or more ways of balancing inertial seismic forces such that the stability of the structure is preserved in the event of the failure of any structural element.

Repair: the restoration or renewal of any degraded component of a building in order to obtain characteristics similar to those prior to degradation.

Level I supporting: Supporting a structural component on a main vertical structural component that is continuous up to the foundations or supporting a secondary structural component on a secondary vertical structural component that is continuous up to the foundations.

Structure: the set of structural components and the links between them that ensure the stability of the building under different types of actions.

Main structure: the entirety of the main seismic components and the connections between them that ensure the stability of buildings under different types of actions.

Conventional fixation-clamping section: the section from which the horizontal seismic action is considered to be transmitted to the structure.

Critical area: area of a main structural component where plastic deformations may occur as a result of seismic action.

Plastic area: area of a main structural component where plastic deformations develop as a result of seismic action, in accordance with the configuration of the plastic mechanism.

1.4 Symbols

(26) The symbols used in this technical regulation are detailed next to each relationship.

1.5 Units of measurement

(27) Units of the International System shall be used.

(28) For calculations, the following units of measurement shall be recommended:

- dimensions, distances: m, mm;
- stresses and loads: N, kN, kNm, kN/m, kN/m^2;
- unified efforts: N/mm^2;
- mass: kg;
- specific mass (density): kg/m^3;
- specific weight: kN/m^3;
- speeds: m/s;
- accelerations: m/s^2.

1.6 Normative reference documents

(29) The reference normative documents are those included in Table 1.1 and Table 1.2.

Table 1.1 Reference technical regulations

No	Regulation
1.	Design code. Construction design bases, reference number CR 0-2012, approved by Order No 1530/2012 of the Minister for Regional Development and Tourism and supplemented by Order No 2411/2013 of the Minister for Regional Development and Public Administration, hereinafter referred to in this document as Design Code CR 0
2	Rules on the design and verification of timber constructions, reference number NP 005-2022, approved by Order No 227/2023 of the Minister for Development, Public Works and Administration, hereinafter referred to in this normative document as NP 005
3.	Design Code for Reinforced Concrete Frame Structures, reference number NP 007-2025
4	Code for the design of buildings with reinforced concrete structural walls, reference number CR2-1-1.1/2022, approved by Order No 171/2023 of the Minister for Development, Public Works and Administration, hereinafter referred to in this normative document as CR2-1-1.1
5.	Design code for masonry structures, reference number CR 6-2013
6.	Guide for the calculation and design for seismic action of shelving-type metal structures for presentation and storage in commercial spaces, reference number GP 128-2014, approved by Order No 393/2015 of the Minister for Regional Development and Public Administration, hereinafter referred to in this normative document as GP 128
7.	Rules for the production and execution of concrete, reinforced concrete and prestressed concrete works-Part 1: Concrete production, reference number NE 012/1-2022, approved by Order No 30/2023 of the Minister of Development, Public Works and Housing, hereinafter referred to in this normative document as NE 012/1
8.	Rules for the production and execution of concrete, reinforced concrete and prestressed concrete works-Part 2: Execution of concrete works, reference number NE 012/2-2022, approved by Order No 28/2023 of the Minister for Development, Public Works and Administration, hereinafter referred to in this normative document as NE 012/2
9.	Rules on the design of surface foundations, reference NP 112-2014, approved by Order No

	2352/2014 of the Minister for Transport, Construction and Tourism, hereinafter referred to in this normative document NP 112
10.	Regulation on the geotechnical design of foundations on pilots, reference number NP 123-2022, approved by Order No 2405/2022 of the Minister for Development, Public Works and Administration, hereinafter referred to in this normative document as NP 123

Table 1.2 Romanian reference standards:

No	Standard	Name
1.	SR EN 771-1:2011	Specification for masonry units. Part 1: Clay masonry units
2.	SR EN 771-4:2011	Specification for masonry units. Part 4: Autoclaved aerated concrete masonry units
3.	SR EN 772-1:2016	Methods of test for masonry units. Part 1: Determination of compressive strength
4.	SR EN 845-1:2013	Specification for ancillary components for masonry. Part 1: Wall ties, tension straps, hangers and brackets
5.	SR EN 998-2:2016	Specification for mortar for masonry. Part 2: Masonry mortar
6.	SR EN 1015-11:2002	Methods of test for mortar for masonry. Part 11: Determination of flexural and compressive strength of hardened mortar
7.	SR EN 1052-1:2001	Methods of test for masonry Part 1: Determination of compressive strength
8.	SR EN 1052-2:2001	Methods of test for masonry Part 2: Determination of flexural strength
9.	SR EN 1052-3:2003	Methods of test for masonry Part 3: Determination of initial shear strength
10.	SR EN 1090-2:2018	Execution of steel structures and aluminium structures. Part 2: Technical requirements for steel structures
11.	SR EN 1337-1:2003	Structural bearings. Part 1: General design rules
12.	SR EN 1992-1-1:2004	Eurocode 2: Design of concrete structures. Part 1-1: General rules and rules for buildings
13.	SR EN 01/01/1993:2006	Eurocode 3: Design of steel structures. Part 1-1: General rules and rules for buildings
14.	SR EN 03/01/1993:2007	Design of steel structures. Part 1-3: General rules. Additional rules for structural elements and cold rolled sheet metal
15.	SR EN 05/01/1993:2007	Eurocode 3: Design of steel structures. Part 1-5: Structural elements made of plane slabs stressed within their plane
16.	SR EN 08/01/1993:2006	Eurocode 3: Design of steel structures. Part 1-8: Design of joints
17.	SR EN 10/01/1993:2006	Eurocode 3: Design of steel structures. Part 1-10: Material toughness and through-thickness properties
18.	SR EN 1994-1-1:2004	Eurocode 4: Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings
19.	SR EN 1995-1-1:2004	Eurocode 5: Design of timber structures. Part 1-1: General. Common rules and rules for buildings

20.	SR EN 01/01/1996:2006	Eurocode 6: Design of masonry structures. Part 1-1: General rules for reinforced and unreinforced masonry structures
21.	SR EN 1998-1:2004	Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings
22.	SR EN 1998-3:2004	Eurocode 8: Design of structures for earthquake resistance. Part 3: Assessment and retrofitting of buildings
23.	SR EN 10025	Hot rolled steel products of structural steels.
24,	SR EN 10164: 2019	Steel products with improved deformation properties perpendicular to the surface of the product. Technical delivery conditions
25.	SR EN 12845:2020	Fixed fire fighting systems. Automatic sprinkler systems. Design, installation and maintenance
26.	SR EN 15129:2018	Anti-seismic devices
27.	SR EN ISO 6892-1:2016	Metallic materials - Tensile testing - Part 1: Method of test at room temperature

(30) The list of reference technical regulations given in this technical regulation shall be consulted together with the list of regulatory documents in force published by the relevant regulatory authorities.

(31) The latest editions of the Romanian reference standards shall be used together with, as the case may be, the national annexes, amendments and errata published by the national standardisation body.

(32) Where separate provisions are identified in this technical regulation or in the applicable regulatory reference documents in a particular design situation, the provisions leading to the highest level of performance in relation to the basic design requirements specified in Chapter 2 shall apply.

2 Basic requirements

2.1 General information

(33) The seismic design of buildings is intended that in the event of an earthquake the following are ensured:

(d) controlling the deterioration of buildings;

(e) ensuring uninterrupted operation for buildings with essential functions, for which maintaining integrity during earthquakes is vital for civil protection.

(34) Seismic safety of buildings is achieved differently depending on the class of importance and earthquake exposure to which they belong.

(35) The basic requirements of seismic design are defined by associating a seismic hazard level with the accepted response of the building to seismic action.

(36) The basic requirements of seismic design for a building are:

(f) Following the occurrence of a rare earthquake, the state of degradation of the building may be significant, and the function of the building may be interrupted for a long time, but the overall stability and non-structural components are preserved.

For restoring the function after the earthquake, structural components may require repair, and non-structural components may require repair or replacement. In some situations, it may not be possible to repair the building under economic conditions.

This requirement shall be ensured by checks at the ultimate limit state.

(g) Following the occurrence of a frequent earthquake, the degradation state of the building is limited, the function can be resumed immediately or after a short interruption and the overall stability and the stability of the non-structural components are preserved.

To ensure functionality, the structure does not require immediate repair; non-structural components may require repair.

This requirement shall be ensured by checks at the serviceability limit state.

(37) In order to define the limit states at which seismic actions are checked, the level of seismic hazard at a site is described by the probability of exceeding, in 50 years, the horizontal spectral accelerations caused by seismic action on the surface of the land. The accepted response is described by the accepted degradation state of the building.

2.2 Classes of importance and exposure to earthquakes

(38) Buildings are divided into classes of importance and exposure to earthquakes according to the socio-economic consequences that can be caused by a major natural and/or anthropogenic hazard, as well as their role in society's post-hazard response activities.

(39) The division of constructions into importance and exposure classes shall be made according to the provisions of the CR 0 code.

(40) Each class of importance and exposure to earthquakes, defined according to the provisions of Table 2.1, is associated with a factor of importance and exposure, $\square_{I,e}$. Through the factors of importance and exposure, the level of seismic hazard for

buildings is differentiated, depending on the class of importance and exposure to earthquakes.

Table 2.1 Classes of importance and seismic exposure

Importance-exposure class	Type of building:
Class I	Buildings with essential functions, whose integrity must be preserved during earthquakes to ensure civilian protection, such as: (h) hospitals and other buildings in the healthcare system, which are equipped with emergency/ambulance services and surgical departments; (i) fire stations, police and gendarmerie headquarters, multi-storey above-ground car parks and garages for emergency services vehicles of various types; (j) stations producing and distributing energy and/or providing essential services for the other categories of buildings mentioned here; (k) buildings containing toxic gases, explosives and/or other hazardous substances; (l) emergency communication and/or coordination centres; (m) shelters for emergency situations; (n) buildings with essential functions for public administration; (o) buildings with functions essential for public order, emergency management, defence and national security; (p) buildings housing essential emergency water tanks and/or pumping stations; (q) buildings with a total above-ground height of more than 45 m; and other buildings of the same type.
Class II	Buildings which would pose a major threat to public safety in the event of their collapse or severe damage, such as: (r) hospitals and other health buildings, other than those belonging to class I, which have a capacity of over 100 persons in the total exposed area; (s) schools, high-schools, universities, or other education buildings, which have a capacity of over 250 persons in the total exposed area; (t) nursing homes, crèches, kindergartens or other similar facilities for the care of persons; (u) multi-storey residential, office and/or commercial buildings with a capacity of more than 300 people in the total exposed area; (v) conference, performance or exhibition rooms with a capacity of more than 200 persons in the total exposed area, stadium stands or sports halls; (w) national cultural heritage buildings, museums, etc. ; (x) ground floor buildings, including mall-type buildings, with more than 1,000 people in the total exposed area; (y) multi-level above-ground parking facilities with a capacity of more than 500 vehicles, other than those belonging to class I; (z) penitentiaries; (aa) buildings whose function discontinuation can have a major impact on the population, such as: buildings directly serving power plants, treatment, purification, water pumping stations, energy production and distribution stations, telecommunications centres, other than those of Class I; (bb) buildings with a total above-ground height between 28 and 45 m and other buildings of the same type.

Class III	Common buildings which do not belong to any of the other classes
Class IV	Buildings of low importance for public safety, with a low level of occupancy and/or reduced economic importance, agricultural structures, temporary structures, etc.

(41) In interpreting the provisions of Table 2.1, the total exposed area represents the built-up area of the building and the area adjacent to it, where emergency access and evacuation are conducted through the building.

(42) In interpreting the provisions of Table 2.1, in the case of multi-section buildings, the number of persons in the total exposed area may be determined separately for each section if the sections are independent from the point of view of the organisation of emergency evacuation movements and fire safety installations, if required by specific technical regulations.

(43) In interpreting the provisions of Table 2.1,, the number of people in the total exposed area refers to the projected capacity of the building.

(44) In interpreting the provisions of Table 2.1,, total height above ground is the height of the building measured as the difference between the elevation of the highest point of the building, considering all structural and non-structural components, and the lowest elevation of the landscaped land on the perimeter of the building.

2.3 Limit states

2.3.1 Ultimate limit state

(45) The design value of the horizontal spectral acceleration for checks at the ultimate limit state, for a class III building of importance and exposure to earthquake, corresponds to a probability of exceedance in 50 years equal to 10 %.

(46) The condition of a building, following the incidence of seismic action defined according to (45), beyond which it is considered that the requirements of this limit state are no longer met, is:

(cc) the structure is moderately degraded, has residual horizontal and/or vertical displacements, and is stable under gravitational loads;

(dd) non-structural components are severely degraded but are stable under gravitational loads.

(47) The design value of the horizontal spectral acceleration for checks at the ultimate limit state, for buildings of different classes of importance and earthquake exposure, shall be modified by multiplying the value referred to in (45) by the factor of importance and earthquake exposure.

(48) When checking buildings at the ultimate limit state, the effects of seismic action are combined with the effects of other actions that may occur simultaneously with the seismic action, considering the partial safety coefficients for the seismic grouping established according to the provisions of the technical regulation CR 0.

2.3.2 Service limit state

(49) The design value of horizontal spectral acceleration for serviceability limit state checks, for a class III building of importance and earthquake exposure, corresponds to a probability of exceedance in 50 years equal to 70 %.

(50) The condition of a building, following the incidence of seismic action defined according to (45), beyond which it is considered that the requirements of this limit state are no longer met, is:

(ee) the structure is slightly degraded, has negligible residual horizontal and/or vertical displacements, and is stable under gravitational loads;

(ff) non-structural components have moderate degradation and are stable under their own weight.

(51) The design value of horizontal spectral acceleration for serviceability limit state checks for buildings of different importance and earthquake exposure classes shall be modified by multiplying the value referred to in (49) by the importance and earthquake exposure factor.

(52) When checking buildings at the serviceability limit state, the effects of the seismic action are combined with the effects of other actions that may occur simultaneously with the seismic action, considering the partial safety factors for checks at the serviceability limit state in the quasi-permanent combination established according to the provisions of the technical regulation CR 0. In this check, the effects of the design seismic action, A_{Ed}, shall be combined considering a partial safety coefficient equal to 1.00.

3 Design seismic action

(53) Design seismic action is a conventional simplified representation of seismic action for use in building design.

(54) Seismic action at the ground surface of a site shall be represented for design by:

(gg) the elastic response spectra of absolute accelerations in two horizontal orthogonal directions and in the vertical direction, to be determined in accordance with the provisions of 3.1,

or

(hh) the variation over time of the ground acceleration, which shall be determined in accordance with the provisions of 3.2.

(55) To determine the design value of seismic action, A_{Ed}, in accordance with the provisions of 3.1, the design values of the elastic response spectra of the absolute accelerations for the horizontal and vertical components of the ground movement at the ground surface at the site shall be used.

(56) The design value of seismic action given in this technical regulation is a minimum design value.

3.1 Elastic response spectrum

(57) The design seismic action is described by the design values of the ordinates of the elastic response spectra of the absolute accelerations for the horizontal or vertical components of the site ground movement, $S_e(T)$, hereinafter referred to as design acceleration spectra.

(58) Horizontal seismic action for design is described by the spectrum of horizontal accelerations for design, noted $S_{e,h}(T)$.

(59) Vertical seismic action for design is described by the spectrum of vertical accelerations for design, noted $S_{e,v}(T)$.

(60) For checks at the ultimate limit state and the serviceability limit state, different spectra of horizontal or vertical accelerations are provided for the design.

(61) The spectrum of horizontal design accelerations for ultimate limit state checks of class III buildings of importance and earthquake exposure corresponds to a probability of exceedance of 10 % in 50 years.

(62) The spectrum of horizontal design accelerations for serviceability limit state checks of class III buildings of importance and earthquake exposure corresponds to a probability of exceedance of 70 % in 50 years.

(63) Spectrum of accelerations for design, $S_e(T)$, is defined generically as:

$$S_e(T)=\begin{cases}\gamma_{I,e}\eta F_T S_{ap}\dfrac{(0{,}6T+0{,}4T_B)}{T_B}, for\ 0\le T\le T_B\\ \gamma_{I,e}\eta F_T S_{ap}, for\ T_B<T\le T_C\\ \gamma_{I,e}\eta F_T S_{ap}\dfrac{T_C}{T}, for\ T_C<T\le T_D\\ \gamma_{I,e}\eta F_T S_{ap}\dfrac{T_C T_D}{T^2}, for\ T>T_D\end{cases} \qquad (3.1)$$

where:

T the horizontal vibration period of the system with a degree of dynamic freedom and elastic response (expressed in seconds);

$\gamma_{I,e}$ importance factor and earthquake exposure; the values of the importance factor and exposure to the earthquake are given at (65);

η correction factor taking into account the critical damping fraction of the design structure; the calculation relationships of the correction factor are given at (66);

F_T topographic amplification factor; the values of the topographic amplification factor are given at (67)...(69);

S_{ap} the value of the absolute spectral acceleration corresponding to the range between the corner periods T_B and T_C of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 %; values S_{ap} for the horizontal components of ground movement for checks at SLU and SLS are provided in Annex A;

T_B, T_C, T_D corner (control) periods of the design acceleration spectrum; values T_C for the horizontal components of ground movement for checks at SLU and SLS are provided in Annex A.

(64) In order to select the values of the importance factor and exposure to earthquakes, the territory of Romania is conventionally divided into two zones:

(ii) Zone 1, which includes all the administrative territorial units in the counties of Alba, Arad, Bihor, Bistrița-Năsăud, Brașov, Caraș-Severin, Cluj, Hunedoara, Maramureș, Mureș, Sălaj, Satu Mare, Sibiu, and Timiș;

(jj) Zone 2, which includes all territorial administrative units in the counties of Argeș, Bacău, Botoșani, Brăila, Bucharest, Buzău, Călărași, Constanța, Covasna, Dâmbovița, Dolj, Galați, Giurgiu, Gorj, Harghita, Ialomița, Iași, Ilfov, Mehedinți, Neamț, Olt, Prahova, Suceava, Teleorman, Tulcea, Vâlcea, Vaslui, Vrancea.

(65) Importance and earthquake exposure factor values for SLU and SLS checks, $\gamma_{I,e}^{SLU}$ and $\gamma_{I,e}^{SLS}$, are set out in Table 3.1.

Table 3.1 Factors of importance and exposure to earthquakes

Importance and earthquake exposure class of the building	SLU		SLS	
	For sites located in counties in:			
	Zone 1	Zone 2	Zone 1	Zone 2
I	1.50	1.25	1.55	1.35
II	1.15	1.10	1.25	1.15
III	1.00	1.00	1.00	1.00
IV	0.70	0.80	0.75	0.80

(66) The values of the correction factor taking into account the critical depreciation fraction of the building shall be determined using the relation:

$$\eta = \begin{cases} \sqrt{\dfrac{10 + \left(1 - \dfrac{T}{T_B}\right)^3 (\xi - 5)}{\xi + 5}}, for\ T \le T_B \\ \sqrt{\dfrac{10}{\xi + 5}}, for\ T > T_B \end{cases} \tag{3.2}$$

where

ξ the fraction of the critical depreciation of the building to be selected in accordance with the provisions of Chapters 5-9, specific to structures made of different materials, expressed as %.

For buildings characterised by a critical depreciation fraction equal to 5 %, the value of the factor η is equal to 1.0 for the entire range of vibration periods.

The value of the correction factor η shall be limited lower at 0.55.

(67) Topographic amplification factor F_T is equal to 1.00 if the location of the building:

(kk) is characterized by values of the corner period T_C^{SLU} greater than or equal to 1.20 s,

or

(ll) are located on flat ground or on slopes with an average slope of less than 15⁰ or a height of less than 30 m, or on their ridges.

(68) For locations characterised by corner period values T_C^{SLU} less than 1.20 s, which are not covered by the provision at (67), (ll), the value of the topographic amplification factor shall be determined as follows:

(mm) if the sites are situated on the ridges of the slopes with an average slope between 15⁰ and 30⁰, the value of the topographic amplification factor F_T is 1.20;

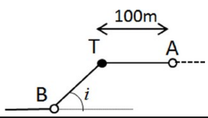
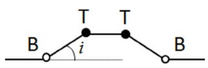
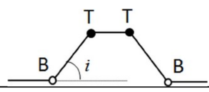
(nn) if the sites are located on the ridges of slopes with an average slope of more than 30⁰, the value of the topographic amplification factor F_T is 1.40;

(oo) if the sites are situated on the slope, the value of the topographic amplification factor shall be determined by linear interpolation between the value 1.00 at the base of the slope and the value 1.20 or 1.40, determined according to (68) or (nn), depending on the height of the site from the base of the slope and the total height of the slope;

(pp) if the sites are situated on the top of the slope at a distance of 100 m or more from its ridge, the value of the topographic amplification factor shall be 1.00. For sites situated on the top of the slope at a distance of less than 100 m from its ridge, the value of the topographic amplification factor shall be obtained by linear interpolation between the value 1.00, at a distance of 100 m from the ridge of the slope, and the value 1.20 or 1.40, determined according to (68) or (nn), on the ridge of the slope, depending on the distance of the site from the ridge of the slope and 100 m.

(69) How to determine the values of the topographic amplification factor for any location characterised by corner period values T_C^{SLU} less than 1.2 s is shown in Table 3.2.

Table 3.1 Topographic amplification factor values for T_C^{SLU} <1.20 s

Description of the topography	F_T	Drawing
For any location on flat ground or on slopes with an average slope of less than 15⁰ or a height of less than 30 m, or on their ridges	1.00	-
For locations on the ridge of the slopes with an average slope between 15⁰ and 30⁰	1.20	
Ridges with much smaller peak width than at the base and average slopes 15° < *i* < 30°	1.20	
For sites located on the ridges of slopes with an average gradient of more than 30⁰	1.40	

Note: Values F_T refer to the locations situated on the ridge, denoted by T in the drawings. For sites located at the base, denoted by B in the drawings, or at a distance of 100 meters from the ridge, marked by A in the drawings, the value F_T is equal to 1.00. For sites between points B and T, or T and A, the value of F_T shall be determined by linear interpolation.

(70) Spectrum of design accelerations established according to the relation (3.1) is represented in Figure 3.1.

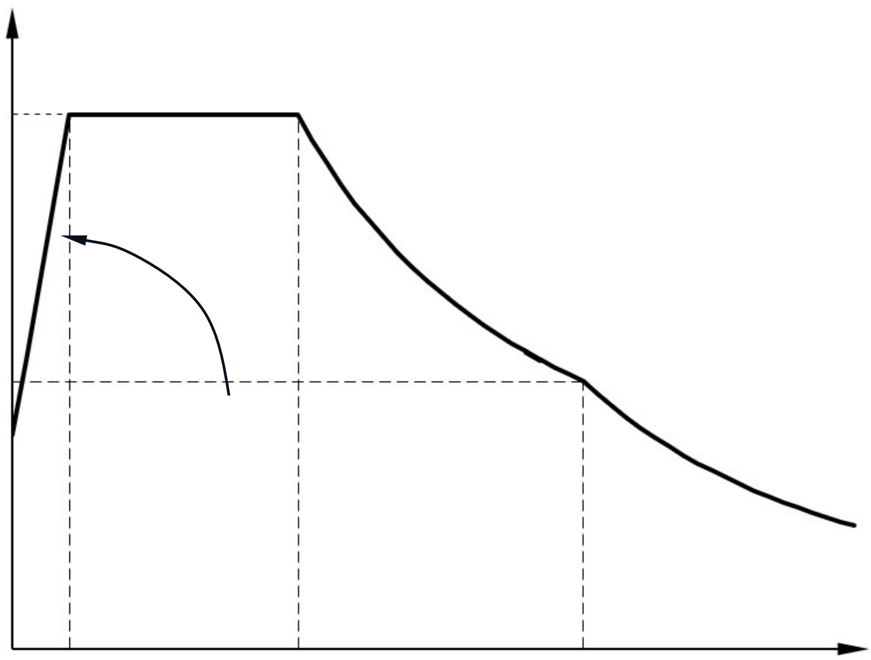


Figure 3.2 Acceleration spectrum for design

(71) In this technical regulation, local terrain conditions are described in a simplified manner by the corner period values T_C of the spectra of horizontal accelerations for design at the location under consideration.

3.1.2 Horizontal acceleration spectra for design

(72) Values of the horizontal acceleration spectrum ordinates for on-site design for checks at SLU, $S_{e,h}^{SLU}(T)$ (in m/s^2), are determined according to the relation 3.1.1.1(3.1) whereby:

$$T_C = T_C^{SLU} \tag{3.1}$$

$$T_B = T_B^{SLU} = \begin{cases} 0{,}10\,s, \text{if } T_C^{SLU} < 1{,}20\,s \\ 0{,}20\,s, \text{if } T_C^{SLU} \geq 1{,}20\,s \end{cases} \tag{3.2}$$

$$T_D = T_D^{SLU} = 2\,T_C^{SLU} \tag{3.3}$$

$$S_{ap} = S_{ap,h}^{SLU} \tag{3.4}$$

where:

T_B^{SLU}, T_C^{SLU}, T_D^{SLU} corner periods (control) of the design horizontal acceleration spectrum for checks at the SLU; values T_C^{SLU} are laid down in Annex A;

$S_{ap,h}^{SLU}$ the value of the absolute horizontal spectral acceleration corresponding to the range between the corner periods T_B^{SLU} and T_C^{SLU} of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 %, for checks at the SLU, provided for in Annex A.

(73) Values of the horizontal acceleration spectrum ordinates for on-site design for checks at SLS, $S_{e,h}^{SLS}(T)$ (in m/s^2), shall be established according to the relation 3.1.1.1(3.1) whereby:

$$T_C = T_C^{SLS} \tag{3.5}$$

$$T_B = T_B^{SLS} = 0{,}10\,s \tag{3.6}$$

$$T_D = T_D^{SLS} = 2T_C^{SLS} \tag{3.7}$$

$$S_{ap} = S_{ap,h}^{SLS} \tag{3.8}$$

where:

T_B^{SLS}, T_C^{SLS}, T_D^{SLS} corner periods (control) of the horizontal acceleration spectrum for design checks at SLS; values T_C^{SLS} shall be determined in accordance with the provisions of Annex A;

$S_{ap,h}^{SLS}$ the value of the absolute horizontal spectral acceleration corresponding to the range between the corner periods T_B^{SLS} and T_C^{SLS} of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 %, for checks at SLS, determined according to the provisions of Annex A.

(74) The elastic response spectrum of relative displacements for horizontal components of ground movement (hereinafter referred to as the design horizontal displacement spectrum), $S_{De}(T)$ (in metres), is defined generically as:

$$S_{De}(T) = S_{e,h}(T)\left(\frac{T}{2\pi}\right)^2 \tag{3.9}$$

(75) Values of the horizontal displacement spectrum ordinates for on-site design for checks at SLU, $S_{De}^{SLU}(T)$ (in metres), shall be determined according to the relation (3.11) where:

$$S_{e,h}(T) = S_{e,h}^{SLU}(T) \tag{3.10}$$

(76) Values of the horizontal displacement spectrum ordinates for on-site design for checks at SLS, $S_{De}^{SLS}(T)$ (in metres), shall be determined according to the relation (3.11) whereby:

$$S_{e,h}(T) = S_{e,h}^{SLS}(T) \tag{3.11}$$

3.1.3 Spectra of vertical accelerations for design

(77) For the determination of the ordinates values of the spectra of vertical accelerations for on-site design, the two zones defined in 3.1, (8) shall be used.

(78) For the territorial administrative units located in the counties of Zone 1, the values of the ordinates of the spectrum of vertical accelerations for on-site design for checks at the SLU, $S_{e,v}^{SLU}(T)$ (in m/s^2), shall be established according to the relation (3.1) whereby:

$$T_C = T_{C,v}^{SLU} = 0{,}40\,s \tag{3.1}$$

$$T_B = T_{B,v}^{SLU} = 0{,}05\,s \tag{3.2}$$

$$T_D = T_{D,v}^{SLU} = 2\,T_{C,v}^{SLU} \tag{3.3}$$

$$S_{ap} = S_{ap,v}^{SLU} = 0{,}70\,S_{ap,h}^{SLU} \tag{3.4}$$

where:

$T_{B,v}^{SLU}$, $T_{C,v}^{SLU}$, $T_{D,v}^{SLU}$ corner periods (control) of the vertical acceleration spectrum for design verifications at the SLU;

$S_{ap,v}^{SLU}$ the value of the vertical spectral acceleration corresponding to the range between the corner periods $T_{B,v}^{SLU}$ and $T_{C,v}^{SLU}$ of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 %, for checks at the SLU.

(79) For the territorial administrative units located in the counties of Zone 1, the values of the ordinates of the spectrum of vertical accelerations for on-site design for checks at SLS, $S_{e,v}^{SLS}(T)$ (in m/s²), shall be established according to the relation (3.1) whereby:

$$T_C = T_{C,v}^{SLS} = 0{,}30\,s \tag{3.5}$$

$$T_B = T_{B,v}^{SLS} = 0{,}05\,s \tag{3.6}$$

$$T_D = T_{D,v}^{SLS}\,¿\,2\,T_{C,v}^{SLU} \tag{3.7}$$

$$S_{ap} = S_{ap,v}^{SLS} = 0{,}60\,S_{ap,h}^{SLS} \tag{3.8}$$

where:

$T_{B,v}^{SLS}$, $T_{C,v}^{SLS}$, $T_{D,v}^{SLS}$ corner periods (control) of the design vertical acceleration spectrum for checks at SLS;

$S_{ap,v}^{SLS}$ the value of the vertical spectral acceleration corresponding to the range between the corner periods $T_{B,v}^{SLS}$ and $T_{C,v}^{SLS}$ of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 % for checks at SLS.

(80) For the administrative territorial units located in the counties of Zone 2, the values of the ordinates of the spectrum of vertical accelerations for on-site design for checks at the SLU, $S_{e,v}^{SLU}(T)$ (in m/s²), shall be determined according to the relation (3.1) whereby:

$$T_C = T_{C,v}^{SLU} = 0{,}60\,s \tag{3.9}$$

$$T_B = T_{B,v}^{SLU} = 0{,}05\,s \tag{3.10}$$

$$T_D = T_{D,v}^{SLU} = 2\,T_{C,v}^{SLU} \tag{3.11}$$

$$S_{ap} = S_{ap,v}^{SLU} = 0{,}60\,S_{ap,h}^{SLU} \tag{3.12}$$

where:

$T_{B,v}^{SLU}$, $T_{C,v}^{SLU}$, $T_{D,v}^{SLU}$ corner periods (control) of the vertical acceleration spectrum for design verifications at the SLU;

$S_{ap,v}^{SLU}$ the value of the vertical spectral acceleration corresponding to the range between the corner periods $T_{B,v}^{SLU}$ and $T_{C,v}^{SLU}$ of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 % for checks at the SLU.

(81) For the administrative territorial units located in the counties of Area 2, the values of the orders of the spectrum of vertical accelerations for on-site design for checks at SLS, $S_{e,v}^{SLS}(T)$ (in m/s²), shall be determined according to the relation (3.1) whereby:

$$T_C = T_{C,v}^{SLS} = 0{,}50\,s \quad (3.13)$$

$$T_B = T_{B,v}^{SLS} = 0{,}05\,s \quad (3.14)$$

$$T_D = T_{D,v}^{SLS} = 2\,T_{C,v}^{SLS} \quad (3.15)$$

$$S_{ap} = S_{ap,v}^{SLS} = 0{,}50\,S_{ap,h}^{SLS} \quad (3.16)$$

where:

$T_{B,v}^{SLS}$, $T_{C,v}^{SLS}$, $T_{D,v}^{SLS}$ corner periods (control) of the design vertical acceleration spectrum for checks at SLS;

$S_{ap,v}^{SLS}$ the value of the vertical spectral acceleration corresponding to the range between the corner periods $T_{B,v}^{SLS}$ and $T_{C,v}^{SLS}$ of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 % for checks at SLS.

3.2 Accelerograms

(82) On-site seismic design action is represented by accelerograms describing the variation over time of the ground acceleration compatible with a target spectrum.

(83) The target spectrum for checks at the ultimate limit state is the spectrum of horizontal accelerations for on-site design for checks at the ULS, set out in 3.1.2, (1). The target spectrum for checks at the service limit state is the spectrum of horizontal accelerations for on-site design for checks at SLS, set out in 3.1.2, (2).

(84) Artificial, recorded and/or simulated accelerograms may be used.

(85) Seismic action at the site is represented by pairs of accelerograms acting simultaneously in two horizontal orthogonal directions. Different accelerograms are used in both directions.

(86) The calculation shall be carried out using a set of at least seven pairs of accelerograms. The effects of seismic action shall be determined as the arithmetic mean of their maximum values determined by dynamic calculation for each pair of accelerograms.

(87) For the selection, simulation or generation of accelerograms compatible with the building site, the response spectra of the accelerograms shall be calculated for a critical damping fraction equal to 5 % and compared to the target spectrum.

3.2.1 Artificial accelerograms

(88) Artificial accelerograms are accelerograms generated in accordance with the target spectrum.

(89) The duration of the strong part of artificial accelerograms shall be determined in accordance with the provisions of Table 3.3. Values T_C^{SLU} are characteristic of the site for which the artificial accelerograms are generated.

Table 3.1 Conventional values of earthquake magnitude and duration of the strong part of artificial accelerograms

For sites located in counties in:	The magnitude of a moment, M_w	Duration of the strong part of artificial accelerograms		
		T_C^{SLU} = 0.80 s	T_C^{SLU} = 1.20 s	T_C^{SLU} = 1.80 s
Zone 1	6.5	8.00 s	10.0 s	12.0 s
Zone 2	7.5	25.0 s	30.0 s	35.0 s

(90) By way of exception from (2), the duration of the strong part of the artificial accelerograms may be chosen from on-site seismic hazard studies, if available.

(91) Artificial accelerograms shall be generated in such a way that the elastic response spectra of a set of accelerograms cumulatively meet the following conditions:

(qq) the arithmetic mean of the maximum values of the ordinates of the elastic response spectra of the absolute accelerations corresponding to all the artificial accelerograms in the set is greater than or equal to the maximum value in the target spectrum;

(rr) for each spectral period between $0.2T_1$ and $1.5T_1$, where T_1 is the fundamental period of vibration of the structure, the mean value of the ordinates of the elastic response spectra of the absolute accelerations corresponding to all the artificial accelerograms in the set is greater than or equal to 90 % of the corresponding value of the target spectrum.

3.2.2 Recorded accelerograms

(92) In order to perform the structural calculation by a dynamic calculation method, it is recommended to represent the seismic action by recorded accelerograms, which are available in accepted databases, according to the provision 3.3, (9).

(93) When selecting recorded accelerograms, account shall be taken of the compatibility of the terrain conditions at the site of the building and at the seismic stations that made the recordings. The magnitude shall be determined in accordance with the provisions of Table 3.3. The class of terrain shall be determined in accordance with the specific provisions of 3.3.

(94) To achieve compatibility of a recorded accelerogram with the target spectrum, acceleration values can be multiplied by a single scaling factor between 0.50 and 2.00.

(95) The set of recorded accelerograms shall be considered compatible with the target spectrum if all of the following conditions are met:

(ss) in the periods range between $0{,}2T_1$ and $1{,}5T_1$, where T_1 is the fundamental period of vibration of the system in the direction in which the accelerogram is applied, the ratio of the mean spectrum values calculated by arithmetic averaging of the ordinates of the elastic response spectra of absolute accelerations corresponding to all recorded and possibly scaled accelerograms to the target spectrum values is between 0.75 and 1.30, and the mean value of the ratio is equal to or greater than 0.95;

(tt) in the periods range between $0{,}2\,T_1$ and $1{,}5\,T_1$, where T_1 is the fundamental period of vibration of the structure in the direction to which the accelerogram is applied, the response spectra values for each recorded and possibly scaled accelerogram are greater than or equal to 50 % of the corresponding values of the target spectrum.

3.2.3 Simulated accelerograms

(96) Simulated accelerograms shall be used when available recorded accelerograms do not cover the variety of source mechanisms and source-to-site directivity conditions. The accelerograms shall be simulated so as to cover the range of periods of interest for the projected building defined in (97).

(97) The set of simulated accelerograms shall be considered compatible with the target spectrum if all of the following conditions are met:

(uu) in the periods range between $0{,}2\,T_1$ and $1{,}5\,T_1$, where T_1 is the fundamental period of system vibration in the direction to which the accelerogram is applied, the ratio of the mean spectrum values calculated by arithmetic averaging of the ordinates of the elastic response spectra of absolute accelerations corresponding to all simulated accelerograms to the target spectrum values is between 0.75 and 1.30, and the mean value of the ratio is equal to or greater than 0.95;

(vv) in the periods range between $0{,}2\,T_1$ and $1{,}5\,T_1$, where T_1 is the fundamental period of vibration of the structure in the direction to which the accelerogram is applied, the response spectra values for each simulated accelerogram are at least equal to 50 % of the corresponding values of the spectrum.

3.3 Other provisions

(98) Conventionally, the territory of Romania is divided into seismic zones delimited according to the seismicity levels defined by the values $S_{ap,h}^{SLU}$, as follows:

(ww) low seismicity – values $S_{ap,h}^{SLU} \leq 3.00$ m/s^2;

(xx) moderate seismicity – values 3.00 m/s^2 < $S_{ap,h}^{SLU}$ < 7.50 m/s^2;

(yy) high seismicity – values $S_{ap,h}^{SLU} \geq 7.50$ m/s^2.

(99) The affiliation of the administrative-territorial units to these zones is established in accordance with the provisions of Annex A.

(100) In locations with high seismicity, class I buildings of importance and exposure to earthquakes shall be seismically instrumented with minimally placed digital accelerometers:

(zz) on the floor above the top floor of the building;

(aaa) on the floor at elevation ±0.00 of the building;

(bbb) in an open field, at a minimum distance of 10.0 m from the outer perimeter of the building's footprint on the ground.

Records obtained during earthquakes with a moment magnitude greater than or equal to 4.00, as reported by the National Research-Development Institute for Earth Physics, shall be made available to the construction regulator.

(101) For class I buildings of importance and exposure to earthquakes, specific studies are recommended for the seismic characterization of the terrain conditions at the site. These studies must include:

(ccc) shear wave velocity profile V_s and compression wave profile V_p, for all terrain layers from the surface to the seismic bedrock; the profile can be determined, in a simplified and conventional way, for a depth of 30 metres;

(ddd) the stratigraphy of the site: thickness, density and type of ground;

(eee) the weighted mean velocity of the shear waves for the stratigraphy taken into consideration, $\overline{V_S}$:

$$\overline{V_S} = \frac{\sum_{i=1}^{n} h_i}{\sum_{i=1}^{n} \frac{h_i}{V_{s,i}}} \tag{3.1}$$

where:

h_i thickness of the layer i;

$V_{s.i}$ shear waves velocity for layer i.

(102) Size $\overline{V_S}$ shall be calculated for a depth equal to the lesser of 30 m and H_{800}, where H_{800}(m) is the depth at which seismic bedrock occurs, conventionally identified by a value $V_{s,i}$ greater than or equal to 800 m/s.

(103) Based on the values H_{800} and the weighted average velocity in superficial stratigraphy $\overline{V_S}$, the terrain conditions shall be classified into classes according to the provisions of Table 3.4.

Table 3.1 Classes of terrain according to values H_{800} and $\overline{V_S}$

H_{800}			
$H_{800} \leq 5m$	A	A	E
$5m < H_{800} \leq 30m$	B	E	E
$30m < H_{800} \leq 100m$	B	C	D

$100\,m < H_{800}$	B	F	F

(104) Estimation of the fundamental vibration frequency of the package of terrain layers of thickness $h = minim(30\,m, H_{800})$from the surface of the land, f_0, can be simplified using the relationship:

$$f_0 = \frac{\overline{V_s}}{4h} \quad (3.2)$$

(105) If there is no information on the value H_{800} for the site, the classes of terrain can be determined by simultaneously considering the values $\overline{V_S}$ and f_0 from the site, in accordance with the provisions of Table 3.5.

Table 3.1 Classes according to values f_0 and $\overline{V_S}$

Values f_0 (Hz) and $\overline{V_S}$ (m/s)	Classes of terrain
$f_0 \geq 10$ Hz and $\overline{V_S} \geq 250$ m/s	A
$f_0 < 10$ Hz and 400 m/s $\leq \overline{V_S} < 800$ m/s	B
$\overline{V_S}/250 \leq f_0 < \overline{V_S}/120$ and 250 m/s $\leq \overline{V_S} < 400$ m/s	C
$\overline{V_S}/250 \leq f_0 < \overline{V_S}/120$ and 150 m/s $\leq \overline{V_S} < 250$ m/s	D
$\overline{V_S}/120 \leq f_0 < 10$ Hz and 150 m/s $\leq \overline{V_S} < 400$ m/s or $f_0 \geq 10$ Hz and 150 m/s $\leq \overline{V_S} < 250$ m/s	E
$f_0 < \overline{V_S}/250$ and 150 m/s $\leq \overline{V_S} < 400$ m/s	F

(106) The following databases shall be considered accepted databases for the selection of recorded accelerograms: Engineering Strong Motion Database (https://esm-db.eu/#/home), PEER Strong Motion Database (https://peer.berkeley.edu/peer-strong-ground-motion-databases), Italian Accelerometric Archive (https://itaca.mi.ingv.it/ItacaNet_40/), Turkish Accelerometric Database and Analysis System (https://tadas.afad.gov.tr/login), Strong-motion Seismograph Networks (https://www.kyoshin.bosai.go.jp/).

4 Seismic design

4.1 General information

(107) Seismic performance criteria for buildings are set out in this chapter.

(108) Buildings shall be designed for seismic actions in order to meet the basic requirements set out in Chapter 2.

4.1.1 Building components

(109) From a seismic protection point of view, the seismic performance criteria for a building are established in a differentiated manner for three categories of building components:

(fff) main structural components;

(ggg) secondary structural components;

(hhh) non-structural components.

(110) In the application of this technical regulation in construction, a component of a building falls into a single category.

(111) The rigidity and resistance of secondary structural components and non-structural components interacting with the structure are neglected in the calculation of the main structure under seismic actions if their influence on the response of the structure favours the fulfilment of the basic requirements of seismic design. Otherwise, their influence is taken into account in the calculation of the main structure.

4.1.2 Ductility classes

(112) Seismic performance criteria for buildings are established differently according to the ductility class for which they are designed.

(113) A building as a whole shall be designed for one of the following classes of ductility:

(iii) high ductility class (DCH);

(jjj) moderate ductility class (DCM);

(kkk) low ductility class (LDC).

(114) The ductility class is associated with the capacity of the main structure to dissipate energy through plastic deformations, the resistance capacity of the structure to horizontal seismic actions, the level of seismic degradation accepted and the seismicity of the site.

(115) In the case of the high ductility class, DCH, or the moderate ductility class, DCM, the main structure shall be constructed in accordance with the principles of the capacity design method, ensuring that a favourable seismic response is achieved through the formation of an optimal plastic mechanism with adequate energy dissipation capacity induced by horizontal seismic action.

Note: Structures for buildings can be designed for one of the two ductility classes, high ductility class (DCH) or moderate ductility class (DCM), depending on the energy dissipation capacity and resistance to horizontal forces. Structures designed for DCH have overall and local ductility greater than those designed for DCM. To reduce the ductility requirements, structures in the moderate ductility class have a resistance capacity superior to those in the DCH.

(116) The optimal plastic mechanism of the structure is established in accordance with the capacity of the main structural components to dissipate seismic energy through plastic deformations, plastic deformation capacity and their overstrength, according to the provisions of the chapters 5-9, specific to structures made of different materials.

(117) Structural components that deform plastically as a result of seismic action undergo degradation that may require post-earthquake repair work.

(118) In buildings of the DCL ductility class, the design shall be carried out in order to obtain and control a quasi-elastic structural response to the design seismic action, corresponding to the ultimate limit state.

(119) All the main structural components of a building shall be designed for the same class of ductility.

4.2 Criteria for the composition of structures

4.2.1 Structure configuration

(120) The structure of the building consists of complete structural subsystems resistant to horizontal and vertical forces.

(121) The structure shall be constructed such as to ensure a continuous path of balancing loads from the point of application to the supports, with adequate rigidity and strength.

(122) It is recommended that the main structure be minimally organized according to two main horizontal orthogonal directions, forming vertical bracing planes according to these directions.

(123) It is recommended that the floors be made as rigid diaphragms.

(124) The main structure shall be designed to exhibit predictable behaviour under seismic action, with cumulative compliance with the verification provisions:

(lll) by calculation of the structure;

(mmm) conditions of composition.

(125) The rigidity, strength and energy dissipation capacity of the main structure shall be adjusted in conjunction to ensure an adequate response to design seismic action, within the permissible limits of deformations and forces.

(126) The main structures of buildings shall be constructed in accordance with the structural types and limitations on their use defined in Chapters 5-9, specific to structures made of different materials.

(127) Buildings located in areas with moderate or high seismicity shall be designed for an elasto-plastic response to design seismic action, with appropriate rigidity, strength and ductility, for the ductility class DCH or DCM.

(128) Buildings where the main structures are not composed in accordance with the structural types, structural materials and, where appropriate, production technologies

defined in Chapters 5-9, specific to structures made of different materials, shall be designed for the ductility class DCL.

(129) By way of exception from (127), buildings which, due to their architectural composition, cannot meet the design criteria specific to the ductility class DCH or DCM shall be designed for the ductility class DCL in such a way that their overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations, irrespective of the location.

4.2.2 Regularity of structure

4.2.2.1 Regularity in horizontal plane

(130) In order to ensure a favourable response and to increase the predictability of the building's behaviour during seismic activity, it is recommended that it be regulated in terms of its composition in the horizontal plane through an architectural design that optimally satisfies the conditions given at (131), (nnn) and (ooo).

(131) A building may be considered regular in terms of its horizontal composition if, at each level above the conventional embedding level, the following cumulative conditions are met:

(nnn) the building is approximately horizontally symmetrical, in terms of floor shape and mass distribution, in relation to two horizontal orthogonal directions;

(ooo) the area between the contour of each floor and its convex polygonal envelope does not exceed 10 % of the total area of the floor;

(ppp) The structure is approximately horizontally symmetrical in relation to two horizontal orthogonal directions in terms of the distribution of rigidity and resistance to horizontal actions;

(qqq) the main structure is organized in such a way that the horizontal seismic action is balanced by at least two vertical bracing planes located to the left and right of the centre of mass, aligned with two horizontal orthogonal directions, with similar rigidity, resistance and ductility properties;

(rrr) the main structural components are directly supported;

(sss) floors are constructed as rigid diaphragms, in accordance with 4.2.6;

(ttt) on the perimeter of the building at any level, the maximum horizontal movement in the seismic grouping in the direction of the force shall not exceed by more than 30 % the average of the maximum and minimum horizontal movements.

(132) Buildings that do not meet the provision of (131) are irregular in the horizontal plane.

(133) Irregular buildings in the horizontal plane shall be designed to ensure greater resistance capacity. It is recommended to verify them through linear or non-linear dynamic structural calculation.

4.2.2.2 Vertical regularity

(134) In order to ensure a favourable response and to increase the predictability of the building's behaviour during seismic activity, it is recommended that it be

regularized in terms of its vertical composition through an architectural design that optimally satisfies the conditions given at (135), (uuu) and (vvv).

(135) A multi-storey building may be considered regular from the point of view of its vertical composition if at each level the following cumulative conditions are met:

(uuu) the level area of the building does not vary by more than 20 % from the area of the building at the adjacent, upper, or lower level;

(vvv) the level mass of the building does not vary by more than 20 % from the mass of the adjacent upper or lower level;

(www) the rigidity at the level of horizontal actions of the building does not vary by more than 20 % from the rigidity of the building at the adjacent upper or lower level;

(xxx) the resistance capacity of the building to horizontal actions does not vary by more than 20 % from the resistance capacity at the adjacent upper or lower level, except as explicitly provided for in this technical regulation;

and

(yyy) the main vertical structural components are continuous from the foundations to the top of the building;

(136) When establishing the vertical regularity of multi-storey buildings, in order to verify the conditions (uuu), (vvv), (www) and (xxx) of (135), only the part of the structure situated above the conventional fixation-clamping section defined in accordance with 4.2.7 is considered.

(137) In application of the provision of 4.2.2.2, (135), (www), the level rigidity shall be determined as the ratio of the level shear force to the relative displacement of the level at the centre of mass determined by calculation of the structure using the method of equivalent static seismic forces, in the direction of calculation.

(138) In the application of the provision of 4.2.2.2, (135), (www), the decrease in level rigidity from the level immediately above the conventional fixation-clamping section to the one above it may be neglected, if this variation is determined by the nature of the connections that the structural components have at the conventional fixation-clamping section.

(139) When establishing the vertical regularity of multi-storey buildings, the composition of the top level can be neglected if it exclusively houses technical spaces and its area is less than 30 % of the area of the building at the next lower level.

(140) When establishing the vertical regularity of buildings with a ground floor height regime, the partial intermediate floor can be neglected if its area is less than or equal to 10 % of the built area of the building and its structure is made with beams hinged on the pillars that constitute the main structural components.

(141) When determining the vertical regularity of buildings, the composition of the levels below the conventional fixation-clamping section may be neglected if their composition ensures, at each level, a rigidity and resistance to horizontal actions greater than that of the levels above the conventional fixation-clamping section.

(142) Constructions which do not fulfil the condition of (135) are irregular in the vertical plane.

(143) Irregular buildings in the vertical plane are designed to ensure a greater resistance capacity. It is recommended to verify them through linear or non-linear dynamic structural calculation.

(144) In addition to the provision of (142), the following types of buildings are considered to be vertically irregular:

(zzz) buildings with a frame structure that have partition and enclosing walls made of masonry of any type or concrete, if open commercial spaces or parking lots are organized on the ground floor;

(aaaa) buildings with a walled structure, if there are walls that are interrupted and supported indirectly, above the conventional fixation-clamping section.

(145) Buildings located in areas of high or moderate seismicity where the level rigidity at horizontal actions of the level immediately above the conventional embedding level is less than 70 % of the rigidity of the adjacent upper level shall not be permitted.

4.2.3 Overall torsional rigidity

(146) In order to ensure a favourable response and to increase the predictability of behaviour during seismic action, it is recommended that the building does not have high flexibility in overall torsion.

(147) A structure shall have high overall torsion flexibility if either of the following two conditions is met:

(bbbb) for each horizontal main direction, the highest modal mass does not belong to the first or second mode of vibration;

(cccc) the period of the first proper mode of torsional vibration is greater than the periods of the proper modes of translational vibration in the two main horizontal directions.

(148) In verifying the conditions of (147), local or equipment vibration modes shall not be considered.

(149) By way of exception from (147), structures which meet all of the following conditions may be considered as not having high overall torsion flexibility:

(dddd) comply with the provisions set out in 4.2.2.1, (131), (sss) and (ttt);

(eeee) comprise, in each main direction, at least two structural vertical bracing subsystems located to the left and right of the centre of mass, with a minimum distance between them greater than or equal to 0.50 of the maximum building size measured perpendicular to the direction considered;

(ffff) the structural vertical bracing subsystems located to the left and to the right of the centre of mass, considered in verifying compliance with the provision of (eeee), ensure at least 30 % of the rigidity of the structure against horizontal forces in the direction considered.

Note: Vertical bracing structural subsystems are planar or spatial structural subsystems with high rigidity and resistance to horizontal actions, such as frames with rigid joints, braced frames or structural walls.

(150) Buildings with high overall torsional flexibility shall be designed to provide greater resistance to horizontal seismic actions and verified by dynamic calculation.

4.2.4 Structural redundancy

(151) Seismic design recommends creating structures with high redundancy.

(152) Structures with a higher degree of static indeterminacy have higher redundancy. In determining the degree of static indeterminacy, for the purposes of this paragraph, only the main structural components shall be considered.

(153) In the case of a building seismically designed for ductility class DCH or DCM, the plastic mechanism shall be carried out with sufficient plastic areas of adequate ductility, allowing exploitation of the structural strength reserves and advantageous dissipation of seismic energy.

(154) In the case of multi-storey buildings, it is recommended that the horizontal seismic action be balanced by at least two structural vertical bracing subsystems located to the left and right of the centre of mass, in relation to each direction of seismic action, with similar properties of rigidity, resistance and ductility.

4.2.5 Distances between buildings

(155) Buildings and/or sections of buildings shall be spaced to allow independent horizontal oscillation or to limit the effects of any collisions caused by seismic action.

(156) The location of buildings shall be determined in such a way that the minimum distance between adjacent buildings or sections of buildings that are part of the same property is greater than the maximum value of the square root of the sum of the squares of their maximum horizontal displacements under the design seismic action corresponding to the ultimate limit state at each level.

$$d_{min} \geq \sqrt{d_{Ed,1}^{SLU\,2} + d_{Ed,2}^{SLU\,2}} \tag{4.1}$$

d_{min} the minimum distance between two buildings or sections of buildings that are part of the same property;

$d_{Ed,1}^{SLU}$, $d_{Ed,2}^{SLU}$ design values of the maximum horizontal displacement, caused by the design seismic action corresponding to the ultimate limit state, for each section or building, established in accordance with 4.3.1.2.2, (216).

(157) The location shall be established in such a way that the horizontal projection of the building at ground level, in the situation of maximum horizontal oscillation caused by the design seismic action corresponding to the ultimate limit state, in any direction, is located within the property limit of the building.

(158) In the case of neighbouring multi-storey buildings or sections of buildings having the same height regime, floors at the same vertical elevations and similar dynamic characteristics, the minimum value of the distance between buildings determined in accordance with (156) may be reduced by 30 %. In order to assess the similarity of dynamic characteristics, minimum account shall be taken of the vibration periods and the own vibration forms in the modes with a significant contribution to the total seismic response of the structure established according to 4.5.1.4, (301).

(159) The joints and joint masking devices between two adjacent buildings and/or their attachments shall be made in such a way as not to influence the oscillations of the buildings caused by seismic action.

4.2.6 Horizontal diaphragms

(160) Horizontal diaphragms of a building fall into one of the following categories:

(gggg) rigid diaphragms;

(hhhh) semi-rigid diaphragms;

(iiii) flexible diaphragms.

(161) When calculating the structure, the influence of diaphragm rigidity on the overall structural response to seismic actions is considered.

(162) By way of exception from (161), the influence of flexible diaphragms on the overall response of the structure to horizontal seismic actions may be neglected in the calculation.

(163) By way of exception from (161), rigid diaphragms may be modelled in the calculation to determine the overall response of the structure to horizontal seismic actions as infinitely rigid and resistant to actions in their plane. In this situation, the building masses that generate the inertia forces in the horizontal direction can be considered simplified as being concentrated in the centres of mass of the diaphragms.

(164) Diaphragms where the maximum inherent deformation in the horizontal direction under seismic action applied in the same direction is more than twice the average relative displacement of the main vertical structural components at the immediately lower level fall under the category of flexible diaphragms.

(165) Diaphragms where the maximum inherent deformation in the horizontal direction under seismic action applied in the same direction is less than half of the average relative displacement of the main vertical structural components from the upper limit of the diaphragm fall into the category of rigid diaphragms.

(166) Diaphragms that do not meet the conditions (164) and (165) fall into the category of semi-rigid diaphragms.

(167) Diaphragms made of metal plating without over-concreting, in steel or composite structures braced in the vertical plane or in structures with reinforced concrete walls or steel-concrete composites, may be considered flexible in the calculation, without the assessment provided for in (164).

(168) Diaphragms made of reinforced concrete slabs executed monolithically, with a thickness of more than 100 mm or prefabricated with monolithization with a thickness of more than 60 mm, which are not weakened by large gaps, may be considered rigid in the calculation. Exceptions are diaphragms located just below the conventional fixation-clamping section in reinforced concrete-walled structures, composite-walled structures and braced metal structures that are modelled for calculation with their actual rigidity.

(169) In the case of structures with cantilevered pillars made of concrete, steel and/or composites, horizontal diaphragms that are made of beams arranged in one or two orthogonal directions and metallic plating, without bracings arranged in a horizontal plane, cannot be classified as rigid diaphragms regardless of the results of the assessment by calculation according to (165).

4.2.7 Conventional fixation-clamping section

(170) The conventional building fixation-clamping section is the section from which horizontal seismic action is considered to be transmitted to the structure.

(171) The conventional fixation-clamping section is established by engineering reasoning according to:

- the type of foundations;
- the existence of underground levels with a structure significantly more rigid and resistant than that of the above-ground levels;
- the total height of the underground levels;
- the distance between the ground level and the level of the horizontal diaphragms in the building near the ground;
- the characteristics of the terrain in the vicinity of the building;
- location and type of seismic joints;
- distance from neighbouring buildings and the nature of their infrastructure;
- slope of the land at the site.

(172) Levels fully above the ground shall be above the conventional fixation-clamping section.

(173) If the rigidity and resistance of the structure below ground level are not significantly higher than those of the structure above, the building conventional fixation-clamping section shall be considered at foundation level.

(174) In the case of buildings designed for class DCH or DCM, the designed plastic mechanism shall engage the entire part of the structure above the conventional fixation-clamping section.

(175) The part of the structure located below the conventional fixation-clamping section is designed for quasi-elastic response to design seismic action only.

(176) Further details on how to determine the conventional fixation-clamping section are given in chapters 5-9, specific to structures made of different materials or in the comments section.

4.2.8 Optimal plastic mechanism

(177) In the case of buildings designed for the ductility class DCH or DCM, structures shall be designed in such a way that they can develop an optimal plasticisation mechanism under horizontal seismic action.

(178) The optimal plastic mechanism develops through the appearance of plastic deformations in the main structural components or in the links between them.

(179) The optimal plastic mechanism develops exclusively in the part of the structure located above the conventional fixation-clamping section. The part of the structure located below the conventional fixation-clamping section elastically responds to the design seismic action.

(180) The optimal plastic mechanism with an optimal energy dissipation capacity induced by horizontal seismic action has the following characteristics:

(jjjj) drives the structure as a whole above the conventional fixation-clamping section;

(kkkk) plastic deformations are moderate and evenly distributed throughout the main structure;

(llll) plastic deformations occur in the main structural components or in the links between them that have sufficient plastic deformation capacity, in relation to the plastic deformations caused by the seismic design action, under stable hysteretic behaviour;

(mmmm) plastic deformations are reversible;

(nnnn) the main structural components and the links between them shall be constructed in such a way as to avoid any kind of fragile breakage.

(181) The main structural components that respond elastoplastically to the design seismic action corresponding to the ultimate limit state shall be selected so that they can be inspected after the earthquake to establish the degradation state and carry out the necessary repairs.

(182) The main structural components which cannot be inspected or repaired after the earthquake shall be designed for elastic response to design seismic action corresponding to the ultimate limit state.

(183) The structures of multi-storey buildings shall be designed in such a way that they do not form level mechanisms.

(184) Specific provisions on the configuration of the optimal plastic mechanism for different types of structures are given in chapters 5-9, specific to structures made of different materials.

4.3 Seismic performance criteria for the main structure

(185) This paragraph contains provisions on seismic performance criteria for the main structure, the main structural components and the connections between them.

(186) The main structure shall be made so that it meets the seismic performance criteria given in this paragraph for any horizontal direction of seismic action.

(187) The design value of a seismic effect is the maximum value of that effect caused by the design seismic action, including the effects of other actions concurrent with it, in accordance with the provisions of the technical regulation CR 0.

(188) In the case of structures with an elasto-plastic response under design seismic action, the design values of seismic action effects shall be established by:

(oooo) transforming the values resulting from the calculation of the structure by a linear calculation method to quantify the non-linearity of the structural response caused by the design seismic action, in accordance with the principles of the resistance capacity hierarchy method, according to the provisions specific to each ductility class,

or

(pppp) by calculating the structure using a non-linear calculation method.

(189) In the case of structures with exclusively elastic response under design seismic action, the design values of the effects of seismic action shall be established by calculating the structure through a linear calculation method.

(190) Design value of the seismic action effect, E_d, shall be limited to the permissible design value of the action effect, R_d, according to the relationship:

$$E_d \leq R_d \tag{4.1}$$

(191) In order to determine the design values of the effects of seismic action, the calculation of the structure shall be carried out according to the provisions of 4.5.

4.3.1 Ultimate limit state

(192) Structures of buildings required for seismic actions and their reliance on the ground shall be verified at the ultimate limit state in accordance with the provisions of 2.3.1.

(193) When designing seismic structures for buildings at the ultimate limit state, the following characteristics shall be verified:

(qqqq) resistance;

(rrrr) ductility;

(ssss) stability.

(194) By way of exception from (193), the seismic design of structures for buildings of the ductility class DCL at the ultimate limit state shall verify their resistance and stability.

(195) Verification of the characteristics of the structure according to (193) and (194) shall be carried out considering the design seismic action for the ultimate limit state, established according to the provisions of chapter 3.

(196) The main structure shall be designed to the ultimate limit state according to the specific provisions given in chapters 5-9, for structures made of different materials.

(197) When seismic designing buildings to the ultimate limit state, their resistance, rigidity and ductility shall be adjusted in conjunction, taking into account the cumulative influence of these properties on the overall structural response, in accordance with the ductility class in which the building is framed.

(198) Verification of the reliance on the ground at the ultimate limit state shall be carried out in accordance with the specific technical regulations and the principles and additional provisions given in this technical regulation.

4.3.1.1 Resistance

(199) The structure shall be constructed so that the design value of the resistance capacity of the main structure to horizontal actions is greater than or equal to the base shear force corresponding to the design seismic action determined in accordance with the provisions of 4.5.1.3, (293).

For this verification, the design value of the resistance capacity of the structure as a whole to horizontal actions corresponds to the design values of the resistances of the materials and the action of horizontal forces applied statically, distributed according to the results of the modal analysis for the fundamental mode of vibration, in each direction considered, and is consistent with the seismic action grouping situation.

Note: This condition can be considered fulfilled if for all the main structural components and for the links between them the condition (4.1) is fulfilled, expressed in terms of resistance, where the value E_d is derived from the calculation of the structure by the method of equivalent static lateral forces according to 4.5.1.3.

(200) The provision of (199) does not apply to seismically isolated structures.

(201) The structure shall be constructed such that for each main structural component the condition (4.1) is expressed in terms of resistance, where E_d is the design value of the seismic grouping effort, also taking into account second-order effects when significant, and R_d is the design value of the resistance capacity, calculated with the design values of the resistances of the materials, based on specific mechanical models.

(202) Condition of (201) shall be fulfilled for all main structural components along their entire length and for all their connections.

(203) The design values of the efforts shall be established from equilibrium conditions considering the formation of plastic zones according to the configuration of the optimal plastic mechanism and the mobilization of overstrength in plastic zones.

(204) The design effort values shall be determined, taking into account the uncertainty of the assessment, by multiplying them by partial safety coefficients greater than one. Exceptions are the efforts that produce plastic deformations in plastic areas positioned according to the configuration of the optimal plastic mechanism, for buildings designed for the DCH or DCM ductility class.

(205) Further provisions on how to determine the design values of efforts for ultimate limit state checks are given in chapters 5-9, specific to structures made of different materials, for different types of structures and structural calculation methods.

(206) Provisions on how to determine the design values of resistance capabilities are given in chapters 5-9, for different structural materials and types of structures.

(207) The main structural components shall be designed with sufficient resistance to ensure a complete, uninterrupted and as short as possible route of loads from the point of application to the foundation ground.

4.3.1.2 Ductility

(208) Ductility of the main structure under horizontal seismic actions shall be ensured by:

(tttt) the ranking of the resistance capacities of the structural components;

(uuuu) limiting plastic deformations of the main structure caused by the design seismic action;

(vvvv) ensuring the local ductility of the main structural components that deform in the plastic domain.

4.3.1.2.1 Hierarchy of resistance capacities

(209) The hierarchy of resistance capacities limits the efforts that can produce brittle fractures and conveniently directs the position of plastic deformation zones, in accordance with the configuration of the optimal plastic mechanism.

(210) For any type of brittle fracture, the resistance capacities of the main structural components or their connections shall be ensured to exceed the stresses corresponding to each type of fracture, which may develop at the incidence of design seismic action.

4.3.1.2.2 Limitation of relative level displacements

(211) For the main seismic structure, the condition 4.3.1.1(4.1) shall be met, expressed in terms of relative level displacements, as follows:

$$d_{Ed,r}^{SLU} \le d_{Rd,r}^{SLU} \qquad (4.1)$$

where:

$d_{Ed,r}^{SLU}$ the design value of the relative level displacement in the horizontal direction, in the seismic combination of actions at the ultimate limit state, also taking into account second-order effects when significant;

$d_{Rd,r}^{SLU}$ the design value of the relative displacement level permitted for checks at the ultimate limit state.

(212) Design value of the allowed relative level displacement for checks at the ultimate limit state, $d_{Rd,r}^{SLU}$, is equal to $0.025h_s$, where h_s isthe total height of the level.

(213) By way of exception from (212), particular design values of the relative level displacement allowed for checks at the ultimate limit state may be provided for in Chapters 5-9, specific to structures made of different materials.

(214) The relative level displacement is equal to the absolute value of the difference between the horizontal displacements of the points of intersection of a vertical line with the two consecutive horizontal diaphragms bordering above and below the level.

In the case of the level immediately above the conventional fixation-clamping section, in buildings where the deformations of the infrastructure and foundations in the horizontal direction can be neglected, the relative level displacement is equal to the horizontal displacement of a point situated at the intersection of a vertical line with the horizontal diaphragm that borders the level above.

(215) Fulfilment of the condition (4.1) shall be checked at all levels and at any point of the diaphragms, regardless of their type.

(216) If the calculation of the structure is carried out by a linear static calculation method, the design value of the horizontal displacement, d_{Ed}^{SLU}, of a point in the structure shall be determined by the relation:

$$d_{Ed}^{SLU} = cq\,d'^{SLU}_{Ed} \qquad (4.2)$$

where:

d_{Ed}^{SLU} the design value of the horizontal point displacement caused by the design seismic action corresponding to the ultimate limit state;

d'^{SLU}_{Ed} the value of the point displacement determined by calculating the structure using a linear static calculation method at the ultimate limit state;

q behaviour factor used to calculate the design value of the seismic force, determined in accordance with 4.5.1.1, for the ultimate limit state;

c displacement amplification factor for the ultimate limit state, as set out in Chapters 5-9, specific to structures made of different materials.

(217) If the calculation of the structure is performed by the linear dynamic calculation method, the design value of the horizontal displacement of a point in the structure, d_{Ed}^{SLU} ,, is the maximum absolute value of the horizontal displacement of that point determined by calculation under design seismic action, corresponding to the ultimate limit state, multiplied by the factor c established in accordance with (220).

(218) Where the calculation of the structure is performed by the non-linear dynamic calculation method, the design value of the horizontal displacement of a point in the structure, d_{Ed}^{SLU} , is the maximum absolute value of the horizontal displacement of that point determined by calculation under design seismic action, corresponding to the ultimate limit state.

(219) In the case of applying the non-linear static calculation method, the design value of the horizontal displacement of a point in the structure, d_{Ed}^{SLU} ,, is the value of the displacement of that point associated with the deformation of the building caused by the design seismic action, corresponding to the ultimate limit state.

(220) Provisions on how to determine the displacement amplification factor, c, for checks at the ultimate limit state are given in chapters 5-9, for different structural materials and types of structures.

(221) When checking glazed curtain facades or other facades hanging from the structure, the design value of horizontal displacements shall be considered to be 30 % higher than that determined according to the provisions of (216), (217), (218) or (219). The design values of the horizontal displacements thus established constitute design data for the designer of the facade system.

(222) If the calculation of the structure is done considering the terrain-structure interaction, the design values of the horizontal displacements also include the component caused by the ground rotation of the building.

4.3.1.2.3 Limitation of deformations of the main structural components

(223) For the main seismic structural components or, where appropriate, their connections, the condition 4.3.1.1(4.1) expressed in terms of deformations is fulfilled, where E_d represents the design value of the deformation in the seismic combination, taking into account second-order effects when significant, and R_d represents the design value of the permissible deformation at the considered limit state.

(224) The provision of (223) shall be applied differently depending on the method of structural calculation, the material from which the structure is made and the type of structure, as provided for in Chapters 5-9.

(225) Verification provided for in (223) shall apply to:

(wwww) reinforced concrete structures, regardless of the structural calculation method used in the design;

(xxxx) composite, steel, masonry or wooden structures, if a non-linear calculation method has been used in the design.

(226) If, for a reinforced concrete structure, the calculation of the structure is carried out by a linear static calculation method, the design value of the rotation of the main

structural components that deform plastically from bending may be determined in a simplified manner using the relation:

$$\theta_{Ed}^{SLU} = cq\theta'^{SLU}_{Ed} \tag{4.1}$$

where:

c the displacement amplification factor determined in accordance with 4.3.1.2.2, (220).

θ_{Ed}^{SLU} the bar rotation represented by the angle between the secant and the bar axis at the end where the flow produced by the design seismic action occurs, corresponding to the ultimate limit state;

θ'^{SLU}_{Ed} bar rotation represented by the angle between the secant and the bar axis at the end where flow occurs as determined by linear static calculation in the seismic grouping:

$$\theta'^{SLU}_{Ed} = \frac{d_V}{L_V} \tag{4.2}$$

L_V the distance from the considered end of the element to the inflection point of its deformation;

d_V displacement at the inflection point measured perpendicular to the axis of the undeformed element from the considered end of the element.

In the case of beams of frame structures, sizes θ'^{SLU}_{Ed} can be approximated by the ratio of the relative displacement of the level $d_{Ed,r}^{SLU}$ and the level height h_s:

$$\theta'^{SLU}_{Ed} = \frac{d_{Ed,r}^{SLU}}{h_s} \tag{4.3}$$

(227) The design values of the permissible deformations of structural components or, where appropriate, their connections, for checks at the ultimate limit state, shall be established in accordance with the provisions of Chapters 5-9.

(228) Ensuring the local ductility of structural elements that deform in the plastic range shall be achieved through constructive design measures and by limiting the stresses that reduce the plastic deformation capacity.

4.3.1.3 Stability

(229) The structure as a whole, the various subassemblies and structural components shall be made in such a way as to be geometrically stable. For this purpose, structural components and structures shall be constructed with appropriate shapes and sizes, in accordance with the design values of the actions.

(230) The structure shall be designed to be stable against overturning and slipping by using a foundation system appropriate to the physico-mechanical characteristics of the foundation ground.

4.3.2 Service limit state

(231) The structures of the buildings required for seismic actions and their propping on the ground shall be checked at the limit state of service under the conditions referred to in 2.3.2.

(232) When designing structures for buildings for the serviceability limit state , the rigidity of the structure shall be checked.

(233) Verification of the rigidity of the structure according to (232) shall be carried out considering the design seismic action corresponding to the service limit state, established according to the provisions of Chapter 3.

(234) The verification conditions for resistance and stability at serviceability limit state shall be deemed to be fulfilled if they are fulfilled at the ultimate limit state, as laid down in 4.3.1.1 and 4.3.1.3.

(235) The verification of the propping on the ground at the serviceability limit state shall be carried out in accordance with the specific technical regulations and the principles and additional provisions set out in this technical regulation.

(236) The main structure shall be designed at the serviceability limit state according to the provisions of Chapters 5-9, specific to structures made of different materials.

4.3.2.1 Limitation of relative level displacements

(237) The main structure shall be constructed with sufficient rigidity to horizontal actions to limit its horizontal displacements corresponding to the elastic or quasi-elastic response, by fulfilling the condition 4.3.1.1(4.1) expressed in terms of relative level displacements:

$$d_{Ed,r}^{SLS} \leq d_{Rd,r}^{SLS} \qquad (4.1)$$

where:

$d_{Ed,r}^{SLS}$ the design value of the relative level displacement in the horizontal direction in the seismic grouping, at the serviceability limit state, also taking into account second-order effects when significant;

$d_{Rd,r}^{SLS}$ the design value of the relative level displacement permitted for checks at the serviceability limit state.

Note: This condition ensures that at the level of the building as a whole, the degradation of non-structural elements is limited. However, greater degradations of non-structural components may occur locally due to the uneven distribution of deformations at each level.

(238) The design value of the permissible relative level displacement for service limit state checks shall be:

(yyyy) $0.005h_s$ for buildings containing non-structural components that may experience significant degradation due to horizontal deformations of the structure;

(zzzz) $0.0075h_s$ for buildings that are not of the type (yyyy).

where

h_s the total level height.

(239) Buildings with non-structural masonry walls fall within the category referred to in (238), (yyyy), except those where non-structural masonry walls are used only locally for the delimitation of ventilation recesses or for installations.

(240) Buildings with curtain façades, attached to the structure, and/or other non-structural components which, by the nature of their own construction system, including structural attachments, can follow the horizontal deformations of the

structure under conditions of limited degradation, fall under the category set out in (238), (zzzz).

(241) In interpreting the provisions of (238), clamps are part of non-structural components.

(242) The design value of the relative level displacement in the horizontal direction shall be determined as specified in 4.3.1.2.2, (214).

(243) Fulfilment of the condition (4.1) shall be checked at all levels and at any point of the diaphragms, regardless of their type.

(244) By way of exception from (238), specific design values of the relative permissible level displacement for service limit state checks may be provided for in Chapters 5-9, specific to structures made of different materials.

(245) If the calculation of the structure is carried out by a linear static calculation method, the design value of the horizontal displacement of a point in the structure shall be determined by the equation:

$$d_{Ed}^{SLS} = q\, d'^{SLS}_{Ed} \tag{4.2}$$

where:

d_{Ed}^{SLS} the design value of the horizontal displacement of the point caused by the design seismic action corresponding to the serviceability limit state;

d'^{SLS}_{Ed} the value of the point displacement determined by linear static structural calculation in the seismic grouping at the serviceability limit state;

q the behaviour factor used to calculate the design value of the seismic force, corresponding to the serviceability limit state.

(246) Where the calculation of the structure is carried out by the linear or non-linear dynamic calculation method, the design value of the horizontal displacement of a point in the structure shall be the maximum absolute value of the horizontal displacement of that point caused by the design seismic action corresponding to the service limit state.

(247) Where the calculation of the structure is carried out using the non-linear static calculation method, the design value of the horizontal displacement of a point in the structure shall be the value of the displacement of that point associated with the deformation of the building caused by the design seismic action corresponding to the service limit state.

(248) When checking glazed curtain facades or other facades hanging from the structure, the design value of horizontal displacements shall be considered to be 30 % higher than that determined according to the provisions of (245), (246) or (247). The design values of the horizontal displacements thus established constitute design data for the designer of the facade system.

(249) If the calculation of the structure is done considering the terrain-structure interaction, the design values of the horizontal displacements also include the component caused by the ground rotation of the building.

4.4 Seismic performance criteria for other components

4.4.1 Secondary structural components

(250) This paragraph contains provisions on seismic performance criteria for secondary structural components.

(251) The structure shall be designed so that its rigidity and resistance to horizontal actions, determined by also considering the contribution of secondary structural components, are no more than 15 % higher than the rigidity and resistance to horizontal actions of the main structure, determined without considering the contribution of secondary structural components.

(252) The secondary structural components and their connections shall be made such that they:

(aaaaa) do not alter, by reason of their rigidity and resistance, the regularity of the structure or increase its irregularity in plan or elevation;

(bbbbb) do not affect the plastic mechanism of the main structure;

(ccccc) do not favour the development of brittle fractures in the main structural components.

(253) The secondary structural components and their links with the main structural components shall be such that the conditions for limiting the effects of actions can be met, $R_d > E_d$, in the event of structural deformation caused by design seismic action, taking into account second order effects.

(254) Secondary structural components and their connections with the main structural components shall be designed to ensure their balance and stability, even under the incidence of design seismic action.

(255) The functional role of secondary structural components shall be ensured in accordance with the specific requirements of the non-structural components given in Chapter 10.

(256) Secondary structural components shall be designed considering the forces acting directly on them and the forces connecting them with the other structural or non-structural components.

(257) Secondary structural components that deform plastically in the event of deformation of the structure caused by design seismic action shall be detailed to ensure ductility according to the provisions specific to the main structural components made of different materials given in chapters 5-9.

(258) Secondary structural components shall be made in such a way as to meet the specific provisions of the technical regulations in construction for structures made of different materials exposed to actions other than seismic action.

(259) When designing secondary structural components, the effects of vertical seismic action are also considered, according to the provisions of this technical regulation.

4.4.2 Non-structural components

(260) Seismic performance criteria for non-structural components are given in Chapter 10.

4.5 Calculation of the structure

(261) One or more of the following methods of calculating structures shall be used for the design:

(ddddd) static linear calculation;

(eeeee) linear dynamic calculation;

(fffff) non-linear static calculation;

(ggggg) non-linear dynamic calculation.

(262) Static linear calculation can be performed by:

(hhhhh) equivalent static lateral force method;

(iiiii) the modal response spectrum calculation method.

(263) For the design of structures, a linear static calculation method shall be used. In the case of irregular structures in the horizontal plane and/or in elevation and for structures with high torsion flexibility, the modal calculation method with response spectra shall be used.

(264) Linear dynamic, static non-linear, and/or non-linear dynamic calculation methods shall only be used to verify the seismic behaviour of structures with elasto-plastic behaviour under design seismic action.

(265) For buildings in classes I, II, or III of importance and exposure to earthquakes, the calculation of structures shall be carried out using the finite element method with spatial models.

(266) By way of exception from (265), the calculation of the structures of class III buildings of importance and exposure to earthquakes can be done on flat models, for certain types of structural systems and materials, if specific provisions are included in chapters 5-9.

(267) In the calculation of the structure, the directions of seismic action shall be determined in such a way as to establish the most unfavourable values of its effects.

(268) In the calculation of the structure, the horizontal seismic action shall be represented bidirectionally. The bidirectional representation shall be achieved by simultaneously considering the seismic action in two orthogonal horizontal directions, as follows:

(jjjjj) in the case of applying a linear static calculation method, by combining the effects of the actions according to the provisions of 4.5.1.6.

(kkkkk) in the case of the application of a dynamic calculation method, by fulfilling the provision of 3.2, (85).

(269) By way of exception from (268), horizontal seismic action may be represented unidirectionally for:

(lllll) buildings belonging to class III of importance and exposure to earthquakes, located in areas of low or moderate seismicity, which meet the provision of 4.2.2.1, (131), (qqq).

(mmmmm) buildings belonging to class II or III of importance and exposure to earthquakes, regular in the horizontal plane and in the vertical plane.

4.5.1 Static linear calculation

(270) In linear static analysis, the effects of seismic action in the horizontal direction shall be determined by considering that the structure responds exclusively elastically and is seismically driven by:

(nnnnn) static, horizontal and/or vertical forces at the masses liable to oscillate under seismic action;

(ooooo) torsion moments, with the moment vector oriented in a vertical direction.

4.5.1.1 Reduced spectrum

(271) In the calculation of the structure using a linear static calculation method, the design value of the base shear force shall be calculated on the basis of the reduced spectrum of horizontal accelerations, $S_{r,h}(T)$.

(272) The value of an ordinate of the reduced spectrum of horizontal accelerations, $S_{r,h}(T)$, corresponding to a period of vibration T, shall be calculated using the formula:

$$S_{r,h}(T)=\frac{S_{e,h}(T_B)}{q} \text{ for } 0<T\leq T_B; \tag{4.1}$$

$$S_{r,h}(T)=\frac{S_{e,h}(T)}{q} \text{ for } T>T_B; \tag{4.2}$$

where:

q the behaviour factor of the structure for horizontal seismic oscillations, referred to in this technical regulation as the 'behaviour factor';

$S_{e,h}(T)$ the ordinate of the horizontal acceleration spectrum for design, corresponding to the period.

(273) Values of the ordinates of the reduced spectrum of horizontal accelerations $S_{r,h}(T)$ for checks at the ultimate limit state, determined according to the relations (4.1) or (4.2), shall be limited below by cumulative compliance with the conditions:

$$S_{r,h}(T)\geq 0{,}08\,S_{ap,h}^{SLU} \tag{4.3}$$

$$S_{r,h}(T)\geq 0{,}25\,m/s^2 \tag{4.4}$$

where:

$S_{ap,h}^{SLU}$ the value of the absolute horizontal spectral acceleration corresponding to the range between the corner periods T_B^{SLU} and T_C^{SLU}, of constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 %, for checks at the ultimate limit state.

(274) The value of the behaviour factor, q, shall be determined according to the energy dissipation capacity, deformation capacity and overstrength of the structure.

(275) The maximum value of the behaviour factor shall be determined in accordance with the provisions of Chapters 5-9, for different structural materials or types of structures.

(276) In order to establish the behaviour factor, the irregular constructions in the horizontal plane, according to 4.2.2.1, fall into the category of structures with high torsion flexibility.

(277) By way of exception from (276), in the case of horizontally irregular constructions or structures with high torsion flexibility satisfying the conditions of 4.2.2.1, (131), (sss) and (ttt), the maximum value of the behaviour factor, q, is the value determined in accordance with the type of structure as set out in Chapters 5-9, for different structural materials, reduced by 20 %. This reduction shall be applied in addition to any other reductions established in accordance with the provisions of this code.

(278) In the case of vertically irregular buildings, the maximum value of the behaviour factor, q, is the value determined in accordance with the type of structure as set out in Chapters 5-9, for different structural materials, reduced by 20 %. This reduction shall be applied in addition to any other reductions established in accordance with the provisions of this code.

(279) When designing structures with low redundancy for the ductility class DCH or DCM, a higher resistance capacity to horizontal seismic actions is ensured by considering a lower behaviour factor.

(280) In the case of buildings designed for the ductility class DCL located in an area of moderate or high seismicity under the conditions specified in 4.2.1 (128) and (129), the behaviour factor, q, is equal to 1.00.

(281) If the type of the main structural system is different in the two main horizontal orthogonal directions, different behaviour factors may be considered when calculating the structure for the two directions.

(282) The design values of the effects of vertical seismic action shall be calculated on the basis of the reduced spectrum for the vertical component of the ground movement.

(283) Low value of an ordinate of the reduced spectrum of vertical accelerations $S_{r,v}(T)$, corresponding to a period of vibration T, shall be calculated using the formula:

$$S_{r,v}(T)=\frac{S_{e,v}(T)}{q_v} \text{ for } T>T_B; \tag{4.5}$$

q_v the behaviour factor of the structure for seismic oscillations in the vertical direction, which is equal to 1.50;

$S_{e,v}(T)$ the ordinate of the design spectrum of seismic accelerations in the vertical direction.

4.5.1.2 Modelling for calculation

(284) The calculation of the structure shall be carried out using rational methods based on the generally accepted principles of structural mechanics.

(285) In order to verify the structure, a mathematical model shall be constructed and this model shall be evaluated in order to establish the effects of the design seismic action.

(286) All significant sources of rigidity and damping shall be taken into account in the calculation of the structure using the linear static calculation method.

(287) The modelling of the rigidity and damping of the structural components and of the joints between them shall be carried out in accordance with the provisions of the specific technical regulations or technical approvals, as the case may be.

(288) When determining the effects of actions on structural components, the conditions of equilibrium, general stability and geometric compatibility of deformations shall be taken into account.

(289) In determining the effects of actions on structural components, account shall be taken of the short-term and long-term properties of materials, as appropriate.

(290) Where no model is identified with which it is possible to calculate stress and/or deformation coverage values in all structural elements, several models, based on different modelling strategies, shall be used to determine the most unfavourable stress situations of the components of the structure, in strict accordance with the rules of the method of prioritisation of resistance capacities.

4.5.1.3 Equivalent static lateral forces method

(291) The method of equivalent static lateral forces may be applied to buildings which simultaneously meet the following conditions:

(ppppp) horizontal floors are rigid diaphragms, as specified in 4.2.6;

(qqqqq) the masses of the building can be considered concentrated at the level of the floors, in terms of the horizontal oscillations of the building as a whole;

(rrrrr) the building is regular in plane and elevation;

(sssss) the vibration period of the structure in fundamental mode in each horizontal direction of calculation is less than or equal to 1.50 s.

(ttttt) the building is classified in class III or IV of importance and exposure to earthquakes.

(292) In the method of equivalent static lateral forces, the effects of seismic action in the horizontal direction shall be determined by linear static calculation considering the structure driven by:

(uuuuu) horizontal forces, applied statically, in the design direction, arranged at the centre of mass of the floors at each level;

(vvvvv) accidental torsion moments, with the moment vector oriented vertically, located at the centre of mass of the floors at each level.

(293) The design value of the basic shear force caused by seismic action, for each horizontal direction of calculation, shall be determined using the relation:

$$F_b = S_{r,h}(T_1)\lambda m \tag{4.1}$$

where:

T_1 the fundamental vibration period of the building for oscillations in the horizontal direction of calculation;

$S_{r,h}(T_1)$ the ordinate of the reduced spectrum of horizontal accelerations corresponding to the fundamental period, T_1;

m total mass of the building;

λ correction factor:

$\lambda = 0,85$ if $T_1 \leq minim(T_C, 1,20\,s)$ and the building has more than two levels

or (4.2)

$\lambda = 1,00$ in all other situations.

(294) The period of vibration of the structure in each main direction shall be determined in accordance with the general principles of the dynamics of structures, through the dynamic analysis of the structure as a whole.

(295) The horizontal static seismic force that applies to the floor at each level of the building is determined by the relation:

$$F_i = F_b \frac{m_i \varphi_i}{\sum_{j=1}^{n} m_j \varphi_j} \qquad (4.3)$$

where:

F_i the static horizontal seismic force at level i;

F_b the design value of the basic shear force;

φ_i, φ_j the ordinate of the fundamental vibration mode in the direction of calculation at level i or j;

n the number of levels of the building;

m_i mass at the level of i;

m_j mass of the level j.

(296) The moments of accidental torsion are determined with the relation:

$$M_{ai} = F_i e_{ai} \qquad (4.4)$$

where,

M_{ai} torsion moment applied at the level of i, with the vector oriented in a vertical direction;

e_{ai} accidental eccentricity of mass at level i;

F_i horizontal static seismic force applied at the level of i.

(297) Accidental torsional moments shall be determined separately for each main horizontal direction considered in the calculation of the building.

(298) Accidental eccentricity is calculated with the expression:

$$e_{ai} = \pm 0,05\, L_i \qquad (4.5)$$

where

e_{ai} accidental eccentricity of the mass at level i from the calculated position of the centre of mass, applied in the same direction at all levels;

L_i the maximum dimension of the convex polygonal envelope of the floor measured perpendicular to the direction of seismic action.

4.5.1.4 Modal response spectrum calculation method

(299) In the method of modal calculation with response spectra, the effects of seismic action in the horizontal direction are determined by linear static calculation considering the structure driven by:

(wwwww) horizontal forces, applied statically in the direction of calculation, positioned at the location of the masses oscillating in the horizontal direction;

(xxxxx) moments of accidental torsion, with the moment vector oriented vertically, positioned in line with the masses oscillating horizontally.

(300) In the method of modal calculation with response spectra, the effects of seismic action are determined separately by calculating the structure for each of its eigenmodes of vibration considered in the calculation.

(301) The calculation considers the natural modes of vibration with a significant contribution to the total seismic response of the structure. For each direction of calculation, this condition shall be deemed to be fulfilled if:

(yyyyy) the sum of the effective modal masses for the eigenmodes of vibration considered represents at least 90 % of the total mass of the structure,

(zzzzz) all eigenmodes of vibration with an effective modal mass greater than 5 % of the total mass are taken into account.

(302) Design value of the basic shear force, $F_{b,k}$ applied in the direction of action of the seismic motion in its eigenmode of vibration k is determined by the relation:

$$F_{b,k}=S_{r,h}\left(T_k\right)m_k \tag{4.1}$$

where

$S_{r,h}\left(T_k\right)$ ordinate of the design spectrum for the horizontal components of the field movement corresponding to the period T_k;

m_k effective modal mass associated with its eigenmode of vibration k which is established by the relation:

$$m_k=\frac{\left(\sum_{i=1}^{n} m_i\varphi_{i,k}\right)^2}{\sum_{i=1}^{n} m_i\varphi_{i,k}^2} \tag{4.2}$$

m_i mass i;

T_k the intrinsic period within the natural mode of vibration k;

$\varphi_{i,k}$ eigenvector component in vibration mode k in the direction of the degree of dynamic translational freedom, at the level of i.

(303) The maximum value of the effects of seismic action shall be determined by combining the values obtained for each specific mode of vibration considered.

(304) If all the proper modes of vibration are independent, the maximum value of the effect of seismic action is determined as the square root of the sum of the squares of the effects obtained for each proper mode of vibration considered, with the relation:

$$E_E = \sqrt{\sum E_{E,k}^2} \tag{4.3}$$

where:

E_E the effect of seismic action;

$E_{E,k}$ the effect of the seismic action corresponding to the eigenmode of vibration *k*.

(305) Modal responses for two consecutive eigenmodes of vibration, *k* and *k*+1, are considered independent if their own periods of vibration T_k and T_{k+1}, with $T_{k+1} \leq T_k$, meet the following condition:

$$\frac{T_k - T_{k+1}}{T_k + T_{k+1}} > \xi_k + \xi_{k+1} \tag{4.4}$$

where:

T_k period of vibration of the structure in its own mode k;

T_{k+1} period of vibration of the structure in its own mode $k+1$;

ξ_k fraction of critical damping corresponding to the mode k;

ξ_{k+1} fraction of critical damping corresponding to mode $k+1$.

(306) If there are non-independent eigenmodes of vibration, the maximum value of the effect of seismic action shall be determined by complete quadratic combination, with the relation:

$$E_E = \sqrt{\sum E_{E,i} r_{ij} E_{E,j}} \tag{4.5}$$

where:

E_E the effect of seismic action;

$E_{E,i}$ the effect of seismic action corresponding to the eigenmode of vibration *i*;

$E_{E,j}$ the effect of the seismic action corresponding to the eigenmode of vibration *j*;

r_{ij} the correction factor determined by the relationships:

$$r_{ij} = \frac{1}{1 + \left(\frac{\alpha_{ij}}{\xi}\right)^2} \text{ for } \xi = \xi_i = \xi_j; \tag{4.6}$$

$$r_{ij} = \frac{8\sqrt{\xi_i \xi_j}\left(\xi_i + \rho_{ij}\xi_j\right)\rho_{ij}^{3/2}}{\left(1-\rho_{ij}^2\right)^2 + 4\,\xi_i \xi_j \rho_{ij}\left(1+\rho_{ij}^2\right) + 4\left(\xi_i^2 + \xi_j^2\right)\rho_{ij}^2} \tag{4.7}$$

ξ_i fraction of critical damping corresponding to mode i;

ξ_j fraction of critical damping corresponding to mode j;

$$\alpha_{ij} = \frac{T_i - T_j}{T_i + T_j} \tag{4.8}$$

$$\rho_{ij}=\frac{T_i}{T_i} \quad (4.9)$$

T_i period of vibration of the structure in eigenmode *i;*

T_j period of vibration of the structure in eigenmode *j.*

(307) The effects of accidental torsion shall be assessed by static application of an accidental torsion moment. The effects of accidental torsion shall be combined with the effects of seismic action determined in accordance with (303)-(311).

(308) Accidental torsion moments shall be calculated using the relation:

$$M_{ai}=F_i e_{ai} \quad (4.10)$$

where:

M_{ai} torsion moment applied at the level of i, with the vector oriented in a vertical direction;

e_{ai} accidental eccentricity of mass at level i;

F_i static horizontal seismic force applied to the mass i.

(309) Accidental torsional moments shall be determined separately for each main horizontal direction considered in the calculation of the building.

(310) Accidental eccentricity shall be determined in accordance with 4.5.1.3, (298).

(311) If the basic shear force resulting from the modal combination of efforts immediately above the conventional fixation-clamping section, $F_{b,t}$, is less than the base shear force, F_b, calculated in accordance with 4.5.1.3, the effects of seismic action, efforts, and deformations resulting from the calculation of the structure shall be amplified by the factor $F_b / F_{b,t}$·.

4.5.1.5 Vertical component of the seismic action

(312) The main and secondary structural components which are sensitive to vertical seismic oscillations shall be checked taking into account the vertical component of the seismic action.

(313) The following types of main or secondary structural components shall be checked taking into account the vertical component of the seismic action:

(aaaaaa) horizontal or approximately horizontal elements with an opening of more than 20 m;

(bbbbbb) horizontal or approximately horizontal elements with a cantilever static scheme longer than 4.00 m;

(cccccc) horizontal or approximately horizontal pre-compressed structural components;

(dddddd) structural components that provide indirect support for vertical structural components.

4.5.1.6 Combination of the effects of seismic action components

(314) The effect of seismic action caused by the combination of two perpendicular horizontal components of seismic action shall be determined as the square root of the sum of the squares of the values of the same effect determined for each horizontal component.

(315) Alternatively to the provision of (314), the effect of seismic action caused by the combination of horizontal components of seismic action may be determined using the following combination rules:

$$E_{Edx} \text{ '+' } 0.30 E_{Edy} \tag{4.1}$$

$$0.30 E_{Edx} \text{ '+' } E_{Edy} \tag{4.2}$$

where

'+' means 'to be combined with';

E_{Edx} represents the effect of seismic action in the direction of the horizontal axis x chosen for the structure;

E_{Edy} represents the effect of seismic action in the direction of the horizontal axis y, perpendicular to the axis x .

(316) The provision regarding the combination of efforts given on (314) can be extended to all three components of seismic action. Alternatively, the effect of seismic action caused by the combination of horizontal and vertical components of seismic action may be determined using the following combination rules:

$$0.30 E_{Edx} \text{ '+' } 0.30 E_{Edy} \text{ '+' } E_{Edz} \tag{4.3}$$

$$E_{Edx} \text{ '+' } 0.30 E_{Edy} \text{ '+' } 0.30 E_{Edz} \tag{4.4}$$

$$0.30 E_{Edx} \text{ '+' } E_{Edy} \text{ '+' } 0.30 E_{Edz} \tag{4.5}$$

where

'+' means 'to be combined with';

E_{Edx} and E_{Edy} defined in accordance with (315);

E_{Edz} the effect of vertical seismic action.

(317) The sign of each component in the above combinations shall be chosen in such a way that the effect of the action considered is unfavourable.

(318) In the case of structures designed for ductility class DCH or DCM, when combining the horizontal components of seismic action, the equilibrium of the structure in the plastic mechanism phase shall be considered.

Note: In order to determine the efforts generated by the seismic action in the part of the structure involved in the overall plastic mechanism, it is recommended to combine the efforts that produce the flow in the plastic areas according to the rules set out in (314)-(317), and to determine the other types of efforts from the condition of equilibrium of the structure in the plastic mechanism phase.

4.5.2 Nonlinear static calculation

(319) By non-linear static calculation, the force-horizontal displacement response law shall be determined by loading the structure with monotonically increasing displacements in a horizontal direction, where:

(eeeeee) force is the sum of the projections in the direction of calculation of the shear forces developing in the structural components just above the conventional embedding rate;

(ffffff) displacement is the horizontal movement in the direction and sense of the force, measured at the displacement control point.

(320) The displacement control point coincides with the centre of mass of the highest rigid horizontal diaphragm in the building, except for horizontal diaphragms that border the top of the recessed technical levels.

(321) Modelling the non-linear response of structural elements shall be carried out in such a way that the plastic mechanism of the structure can be identified under seismic actions.

(322) Non-linear static calculation applies to buildings with rigid diaphragms. In this situation, the mass of the building is considered to be composed of concentrated masses arranged in the centres of mass of the diaphragms.

(323) The sum of the masses arranged in the centres of mass of the diaphragms meets the condition:

$$m=\sum_{1}^{n} m_i \tag{4.1}$$

where

m_i concentrated mass at the centre of mass of the diagram '*i*';

m the total mass of the building, above the theoretical fixation-clamping section.

(324) In a direction of seismic action, the loading of the structure with monotonically increasing displacements is performed for both directions, in distinct loading cases. Loading is performed considering distributions of normalized horizontal seismic forces acting at the diaphragm level.

(325) Seismic action shall be modelled using at least the following two modes of distribution of normalised horizontal seismic forces:

(gggggg) a mode of distribution in which lateral forces are proportional to the level masses;

$$F_i=\frac{m_i}{m} \tag{4.2}$$

where

F_i seismic force normalised at diaphragm level '*i*' ;

m_i concentrated mass at the centre of mass of the diagram '*i*';

m total mass of the building;

(hhhhhh) a distribution mode resulting from modal analysis for the fundamental vibration mode for horizontal oscillations in the direction of calculation:

$$F_i = \frac{m_i \varphi_i}{\sum_{j=1}^{n} m_j \varphi_j} \tag{4.3}$$

where

F_i normalised seismic force acting at diaphragm level '*i*';

m_i concentrated mass at the centre of mass of the diagram '*i*';

m_j concentrated mass at the centre of mass of the diagram '*j*';

n the total number of horizontal diaphragms;

φ_i the ordinate of the vibration mode in the fundamental form at the mass m_i.

(326) Alternatively to the provision of (325), (hhhhhh), for buildings with approximately equal floor masses in height, a simplified distribution of horizontal normalised seismic forces may be used considering a linear form of the fundamental vibration mode:

$$F_i = \frac{m_i z_i}{\sum_{j=1}^{n} m_j z_j} \tag{4.4}$$

F_i normalized seismic force at diaphragm level *i*

z_i vertically measured distance between the horizontal diagram *i* and the conventional embedding level;

z_j vertically measured distance between the horizontal diagram *j* and the conventional embedding level.

(327) Horizontal seismic forces shall be applied in the plane of rigid horizontal diaphragms at distances equal to the accidental eccentricities determined in accordance with 4.5.1.3, (298) relative to the centres of mass of the diaphragms, measured perpendicular to the direction of the forces.

(328) Loading with monotonically increasing displacements shall be carried out until the control displacement is reached, which is equal to the lower of the following values:

(iiiiii) horizontal displacement target, i.e. the horizontal displacement of the building caused by the design seismic action multiplied by 1.50;

(jjjjjj) capable horizontal displacement of the building, i.e. the minimum horizontal displacement corresponding to the failure of a main structural component as a result of exceeding the plastic deformation capacity, brittle fracture or loss of stability.

(329) The horizontal displacement of the building caused by the design seismic action shall be determined on the basis of site-specific values of the displacement

spectra for elasto-plastic response, considering the dynamic and resistance properties of the system with an equivalent degree of dynamic freedom.

(330) Mass of the system with an equivalent degree of freedom, m^{SDOF}, is determined by the relation:

$$m^{SDOF} = \sum_{i=1}^{n} m_i \varphi_i \tag{4.5}$$

where

m_i the mass arranged at the level of the horizontal diagram '*i*' of the system with several degrees of dynamic freedom;

φ_i the ordinate of the vibration mode in the fundamental form at the mass m_i, for the system with multiple degrees of dynamic freedom.

(331) The resistance and deformability properties of the system with an equivalent degree of dynamic freedom are established by transforming the force-displacement response law of the system with several degrees of dynamic freedom, determined by non-linear static calculation, with the relations:

$$F^{SDOF} = \frac{F_b}{\Gamma} \tag{4.6}$$

$$d^{SDOF} = \frac{d_n}{\Gamma} \tag{4.7}$$

where:

F^{SDOF} the horizontal force loading the system with an equivalent degree of dynamic freedom;

d^{SDOF} the horizontal movement of the system with an equivalent degree of freedom;

F_b the basic shear force determined by non-linear static calculation on the multi-degree dynamic freedom system according to (319) corresponding to the displacement d_n;

d_n the horizontal displacement of the displacement control point on the multi-degree-of-freedom dynamic system;

Γ the transformation factor established by the relationship:

$$\Gamma = \frac{\sum_{i=1}^{n} m_i \varphi_i}{\sum_{i=1}^{n} m_i \varphi_i^2} \tag{4.8}$$

m_i the mass arranged at the level of the horizontal diagram '*i*' of the system with several degrees of dynamic freedom;

φ_i the ordinate of the vibration mode in the fundamental form at the mass m_i of the system with several degrees of dynamic freedom.

(332) The horizontal force response law – horizontal displacement of the system with a single degree of dynamic freedom is transformed into a bilinear format. The transformation shall be carried out by identifying the conventional values of the

horizontal forces and horizontal displacements associated with the overall flow of the structure and the displacement requirement at the considered limit state. The force displacement response law consists of two segments, as follows:

- a segment starting at the origin passing through the point whose coordinates correspond to the first flow of the system with multiple degrees of dynamic freedom;
- a segment parallel to the axis of displacements positioned in such a way that the areas below the two response laws are equal.

The yield point corresponding to the bilinear response law corresponds to the intersection of the two segments.

The last point corresponding to the response law in bilinear format corresponds to the end of the segment parallel to the axis of displacement corresponding to the maximum displacement.

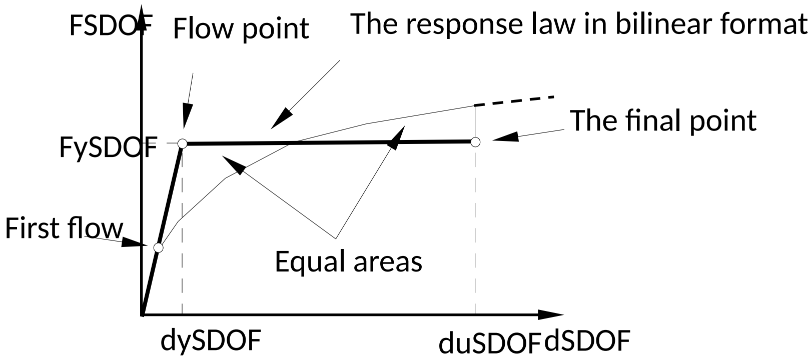


Figure 4.2 Schematic representation of the horizontal force response law – horizontal displacement

(333) System rigidity with an equivalent degree of dynamic freedom, k^{SDOF}, shall be determined by the relation:

$$k^{SDOF} = \frac{F_y^{SDOF}}{d_y^{SDOF}} \quad (4.1)$$

where

d_y^{SDOF} the horizontal flow displacement of the system with a single degree of dynamic freedom corresponding to the minimum horizontal movement associated with the flow entry of a main structural component of the system with multiple degrees of dynamic freedom;

F_y^{SDOF} horizontal flow force of the system with an equivalent degree of dynamic freedom corresponding to the displacement d_y^{SDOF}.

(334) The vibration period of the system with an equivalent degree of dynamic freedom, T^{SDOF}, shall be determined using the relation:

$$T^{SDOF} = 2\pi \sqrt{\frac{m^{SDOF}}{k^{SDOF}}} \quad (4.2)$$

where:

k^{SDOF} the rigidity of the system with an equivalent degree of dynamic freedom;

m^{SDOF} the mass of the system with an equivalent degree of freedom.

(335) Horizontal displacement of the system with an equivalent degree of dynamic freedom caused by design seismic action, d_t^{SDOF}, shall be determined using the relation:

$$d_t^{SDOF} = ¿\, S_{De}\left(T^{SDOF}\right) \text{ if } T^{SDOF} > T_C$$

$$d_t^{SDOF} = \frac{1+(u-1)\dfrac{T_c}{T_{SDOF}}}{u} S_{De}\left(T^{SDOF}\right) \leq 3\, S_{De}\left(T^{SDOF}\right) \text{if } T^{SDOF} \leq T_C \quad (4.3)$$

where:

$S_{De}\left(T^{SDOF}\right)$ the ordinate value of the elastic response spectrum of relative displacements for the horizontal components of the ground motion corresponding to the period T^{SDOF} ;

T^{SDOF} the vibration period of the system with an equivalent single degree of dynamic freedom;

T_C the corner period of the design acceleration spectrum;

u the ratio of the ordinate value of the horizontal acceleration spectrum of the elastic system, $S_e\left(T^{SDOF}\right)$, and the horizontal acceleration of the elasto-plastic response system with the same vibration period and damping;

$$u = S_e\left(T^{SDOF}\right) \frac{m^{SDOF}}{F_y^{SDOF}} \quad (4.4)$$

m^{SDOF} the mass of the system with an equivalent degree of dynamic freedom;

F_y^{SDOF} the horizontal flow force of the system with an equivalent degree of freedom, determined in accordance with (333) considering the average values of the resistances of the materials.

(336) The horizontal displacement of the system with several degrees of dynamic freedom caused by the design seismic action at the displacement control point shall be determined using the formula:

$$d_t = d_t^{SDOF}\, \Gamma \sqrt{1+\left(\frac{\Gamma'}{\Gamma}\right)^2} \quad (4.5)$$

where:

d_t^{SDOF} the horizontal displacement of the system with an equivalent degree of dynamic freedom caused by the design seismic action;

Γ the transformation factor established in accordance with (331);

Γ' the transformation factor for the vibration mode perpendicular to the vibration mode corresponding to the direction for which the non-linear static calculation is made, if it has non-zero modal mass.

Note: If the structure does not have vibration modes perpendicular to the vibration mode corresponding to the direction on which the nonlinear static calculation is made, the factor $\sqrt{1+\left(\frac{\Gamma'}{\Gamma}\right)^2}$ is equal to 1.0.

(337) Horizontal displacement of a specific point of the building, located at a distance x from the displacement control point, measured perpendicular to the direction of calculation, shall be determined by amplifying the displacement obtained through non-linear static calculation, corresponding to the target displacement d_t, with the factor:

$$c_p=\left(1+0,6\frac{x}{L_e}\right) \tag{4.6}$$

where:

L_e the maximum dimension of the building measured perpendicular to the direction of calculation;

x the distance between the point considered and the displacement control point, measured perpendicular to the direction of calculation.

(338) By way of exception from (337), in the case of systems with high torsional flexibility or where the distance between the centre of mass and the centre of rigidity is not less than 10 % of the maximum dimension of the building in each main orthogonal direction, at each level, the determination of the horizontal displacement caused by the design seismic action shall be made by non-linear dynamic calculation.

(339) Verification of the main structural components to fulfil the general verification condition given at 4.3.1.1(4.1) expressed in terms of deformations according to 4.3.1.2.3 shall be carried out using the design values of deformations in seismic groups, E_d, corresponding to displacements calculated in accordance with the provision (337).

4.5.3 Linear dynamic calculation

(340) By linear dynamic calculation, the variation over time of the effects of seismic action on the structure is determined.

(341) The variation over time of the effects of the seismic action of the structure is obtained by directly integrating the differential equations of motion.

(342) The design seismic action is represented by accelerograms containing discrete numerical values of the horizontal accelerations of the terrain during the earthquake. Accelerograms shall be selected in accordance with the provisions of 3.2.

(343) The fraction of the critical damping, to be chosen according to the material of which the structure is composed and the type of structure, as provided for in Chapters 5 to 9, shall be less than or equal to 5 % for all modes of vibration considered in the analysis.

(344) In the calculation of the structure, accidental torsion is considered by moving the centre of mass of the floor by a distance equal to 5 % of the maximum dimension of the building measured perpendicular to the direction of seismic action.

(345) The structure is operated simultaneously in two horizontal orthogonal directions.

(346) A minimum of seven analyses shall be performed using seven sets of accelerograms.

(347) For a set of accelerograms, the maximum value of a seismic effect shall be determined as the maximum value of this effect calculated at each time step of the analysis.

(348) The values of the effects of seismic action shall be determined, for each set of accelerograms, so as to ensure that the condition of 4.3.1.1, (199), is met and to quantify the foreseeable non-linear response as follows:

(kkkkkk) the scaling factors shall be calculated:

$$\eta_x = \frac{F_{b,x}}{V_{t,x}} \geq 1{,}0 \qquad (4.1)$$

$$\eta_y = \frac{F_{b,y}}{V_{t,y}} \geq 1{,}0 \qquad (4.2)$$

where:

$F_{b,x}$ design value of the basic shear force calculated in accordance with 4.5.1.3, (293), for the direction *ox* of the structure;

$F_{b,y}$ design value of the basic shear force calculated in accordance with 4.5.1.3, (293), for the direction *oy* of the structure;

$V_{t,x}$ basic shear force resulting from linear dynamic analysis without consideration of accidental torsion, in the direction *ox*;

$V_{t,x}$ basic shear force resulting from linear dynamic analysis without consideration of accidental torsion, in the direction *oy*;

(llllll) scaled values of the effects of seismic action shall be calculated:

- for seismic response in the direction *ox*:

$$E_{dx} = \eta_x \frac{(E¿¿dx')_x}{q_x} + \eta_y \frac{(E¿¿dx')_y}{q_y} ¿¿ \qquad (4.3)$$

where:

q_x behaviour factor of the structure in the direction *ox;*

q_y behaviour factor of the structure in the direction *oy;*

E_{dx} design value of the effect of seismic action in the direction *ox*;

$(E¿¿dx')_x ¿$ the value of the effect of seismic action in the direction *ox* resulting from linear dynamic calculation in the direction *ox*, with consideration of accidental torsion;

$(E¿¿dx')_y ¿$ the value of the effect of seismic action in the direction *ox* resulting from linear dynamic calculation in the direction *oy* without consideration of accidental torsion.

- for seismic response in the direction *oy*:

$$E_{dy}=\eta_x \frac{(E\text{¿¿}dy')_x}{q_x}+\eta_y \frac{(E\text{¿¿}dy')_y}{q_y}\text{¿¿} \tag{4.4}$$

where:

q_x behaviour factor of the structure in the direction *ox;*

q_y behaviour factor of the structure in the direction *oy;*

E_{dy} design value of the effect of seismic action in the direction *oy*;

$(E\text{¿¿}dy')_x\text{¿}$ the value of the effect of seismic action in the direction *oy* resulting from linear dynamic calculation in the direction *ox*, without consideration of accidental torsion.

$(E\text{¿¿}dy')_y\text{¿}$ the value of the effect of seismic action in the direction *oy* resulting from linear dynamic calculation in the direction *oy*, considering accidental torsion.

(349) The effect of the seismic action determined on the basis of the set of seven analyses shall be determined as the average of the maximum values determined for each set of accelerograms.

4.5.4 Non-linear dynamic calculation

(350) When performing the non-linear dynamic calculation, the provisions of 4.5.3 (340), (341), (342), (344), (345), (346), (347) and (349) shall apply, together with the additional provisions given in this paragraph.

(351) In order to determine the effects of seismic action, the design seismic action shall be represented by accelerograms which:

(mmmmmm) meet the criteria given at 3.2;

and

(nnnnnn) generate a maximum value of the basic shear force, determined by non-linear dynamic calculation immediately above the conventional fixation-clamping section, greater than or equal to the basic shear force determined in accordance with the provisions of 4.5.1.3.

(352) The damping shall be modelled in the calculation considering a fraction of the critical damping less than or equal to 2.5 % for all vibration modes considered in the analysis.

(353) In non-linear dynamic analysis, generalized force-generalized displacement response laws are considered for the main structural components that realistically describe rigidity, plastic deformation capacity and energy dissipation capacity.

Note: The chosen laws of behaviour take into account the degradation of rigidity, degradation of resistance and slippages that affect the element's response as a result of repeated cyclic incursions into the plastic response field.

4.5.5 Second-order effects

(354) The effects of seismic action shall be determined taking into account second-order effects.

(355) By way of exception from (354), second-order effects may be disregarded in the calculation if the condition is met for each:

$$\theta = \frac{P_{tot} d_{Ed,r}^{SLU}}{V_{tot} h_s} \leq 0{,}10 \tag{4.1}$$

where:

θ coefficient of sensitivity of relative level displacement

P_{all} total gravitational load from the level considered and from above, in the seismic grouping;

$d_{Ed,r}^{SLU}$ the relative level displacement corresponding to the maximum horizontal displacement of the structure under the design seismic action corresponding to the ultimate limit state;

V_{all} shear force at the level corresponding to the maximum horizontal displacement of the structure at the incidence of the design seismic action, corresponding to the ultimate limit state;

h_s the height of the floor.

(356) If $0{,}10 < \theta \leq 0{,}20$, where θ is defined in accordance with (355), second order effects can be considered approximately, by multiplying the values of seismic action effects resulting from the calculation of the structure by the factor $1/(1-\theta)$.

(357) If $0{,}20 < \theta \leq 0{,}30$, where θ is defined in accordance with (355), the effects of seismic action shall be determined by structural calculation that takes into account geometric non-linearity, considering the equilibrium on the deformed position of the structure.

(358) The building shall be designed in such a way that at all levels the condition is met:

$$\theta \leq 0{,}30 \tag{4.2}$$

(359) By way of exception from (355), particular provisions for the quantification of second-order effects may be laid down in Chapters 5-9, specific to structures made of different materials.

4.6 Design of buildings

(360) For the seismic design of buildings with the main structure made of concrete, the provisions given in Chapter 5 shall apply.

(361) For the seismic design of buildings with the main structure made of steel, the provisions given in Chapter 6 shall apply.

(362) For the seismic design of buildings with the main structure made of composite steel and concrete, the provisions given in chapter 7 shall apply.

(363) For the seismic design of buildings with the main structure made of masonry, the provisions given in chapter 8 shall apply.

(364) For the seismic design of buildings with the main structure made of wood, the provisions given in chapter 9 shall apply.

(365) For the seismic design of non-structural components, the provisions given in 10 shall apply

(366) For the seismic design of buildings equipped with seismic devices, the provisions given in Chapter 11 shall apply.

5 Structures made of reinforced concrete

5.1 General information

5.1.1 Purpose and scope

(367) This chapter contains provisions for the seismic design of buildings with the main structure of reinforced concrete, made of monolithic concrete, prefabricated or partially monolithic – partially prefabricated, without prestressing, hereinafter referred to as reinforced concrete structures.

(368) The provisions of this chapter also apply to main structural components made of prestressed concrete, without prestressing in critical areas.

(369) Specific technical regulations and standards SR EN 1992-1-1 and SR EN 1992-1-2 are used for the design of reinforced concrete structures for actions other than seismic ones.

5.1.2 Definitions

(370) The terms specific to this chapter are:

Frame: structural subassembly made up of beams and pillars rigidly connected in joints (joints which restrict the relative rotation of the beams and pillars in the sections adjacent to the joint). For the purposes of this definition, pillars shall have a vertical longitudinal axis or, if there are deviations, the angle formed by the axis of the pillar with the vertical is less than 0.10 rad.

Beam: structural component of reinforced concrete, primarily subjected to bending moment and shear force, where the average normalized axial effort is less than 0.10, with the ratio between the span and the cross-sectional height greater than 3.

Pillar: vertical or approximately vertical structural component supporting gravitational loads predominantly by axial compression, having a free-height to cross-sectional height ratio greater than 2, in both horizontal directions.

Wall: vertical structural component with the ratio of the dimensions of the cross-sectional sides $l_w/b_{w0} \geq 4$.

Ductile wall: wall with impeded rotation at the base, dimensioned and made to dissipate energy by plastic bending deformations in the critical area at its base.

Short wall: wall where the normalized shear force opening is less than 2.

Insulated wall: wall connected to the rest of the structure by horizontal elements, plates or beams, with rigidity and low bending strength.

Coupled wall: wall, part of an assembly of vertical elements to which it is connected by ductile beams, arranged regularly, so that the torque of the axial forces that develop in the vertical elements as a result of horizontal actions is greater than or equal to 30 % of the overturning moment of the assembly, in the phase of plastic mechanism, in the direction of calculation.

Structural wall system: structural system in which the horizontal actions are taken over by the walls, coupled or isolated, with the contribution of the walls to the absorption of the shear force at the level of the conventional fixation-clamping section being greater than 75 % of the basic shear force.

Structural wall system with insulated walls: structural system where horizontal actions are taken over by insulated walls, the contribution of the walls to the absorption of the shear force at the level of the conventional fixation-clamping section being greater than 75 % of the basic shear force.

Wall-type structural system with coupled walls: a structural system in which horizontal actions are absorbed by coupled walls, with the contribution of the walls to absorbing the shear force at the level of the conventional fixation-clamping section being greater than 75 % of the basic shear force.

Frame-type structural system: a structural system in which vertical and horizontal actions are mainly taken over by frames.

Dual structural system: structural system in which horizontal actions are partly taken over by frames and partly by structural walls, individual or coupled.

Dual structural system with predominant walls: dual structural system in which the contribution of the walls to the shear force at the base of the building is greater than 50 % of the basic shear force, and the frames take up at least 25 % of the overall overturning moment at the level of the conventional fixation-clamping section.

Dual structural system with predominant frames: dual system where the contribution of frames to take over the shear force at the base of the building exceeds 50 % of the basic shear force.

Flexible torsion structure: structure with insufficient rigidity and overall torsion resistance.

Inverted pendulum system: main structural system in which more than 50 % of the mass is concentrated in the upper third of the structure or where energy dissipation is mainly carried out at the base of a single building element.

Cantilevered structural system: main structural system consisting of pillars placed vertically, with the static cantilever scheme under horizontal actions and horizontal diaphragms.

Joint: The area of connection between the pillars and beams of the structures, encompassed between the cross-sections at the ends of these elements that delimit it.

End joint: joint into which a single beam enters in the calculation direction.

5.2 Design principles

5.2.1 Ductility classes

(371) Buildings with reinforced concrete structure shall be designed for one of the three ductility classes defined in 4.1.2.

(372) Reinforced concrete structures designed for ductility class DCH or DCM shall be made so that they have adequate and stable energy dissipation capacity under cyclic stress, without significantly reducing resistance to horizontal and vertical forces.

(373) Buildings located in areas of moderate or high seismicity shall be designed for ductility class DCH or DCM.

(374) By way of exception from (373), in areas of moderate or high seismicity, buildings may be designed for the ductility class DCL if their overall resistance

capacity to horizontal seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q=1$), irrespective of the location, where it is not possible to meet the design criteria specific to the ductility class DCH or DCM.

(375) The main structures shall be seismically designed for the ductility class DCL on the basis of the provisions of Chapters 1, 2, 3 and 4 of this technical regulation and the provisions of SR EN 1992-1-1, together with the provisions explicitly indicated for this class of ductility in this chapter.

5.2.2 Types of structures

(376) Buildings with reinforced concrete structures designed for seismic actions have the main structural system as follows:

(oooooo) structural frame-type system;

(pppppp) wall-type structural system with insulated walls;

(qqqqqq) wall-type structural system with coupled walls;

(rrrrrr) dual structural system with predominant walls;

(ssssss) dual structural system with predominant frames;

(tttttt) inverted pendulum structural system;

(uuuuuu) structural system with cantilevered pillars.

(377) Exceptionally, a building with a reinforced concrete structure that does not fall within the structural types indicated in (376) shall be designed for the ductility class DCL so that its overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q=1$).

(378) Buildings with reinforced concrete structures can have structural systems of different types in the two main horizontal orthogonal directions. In their design, the specific design rules for each type of structural system shall be used in the appropriate direction.

(379) If, along a horizontal main direction, the building has a main structural system consisting of two types, the lowest value of the behaviour factor corresponding to the two types of structures shall be used when designing the structure in this direction.

(380) By way of exception from (378), for reinforced concrete buildings with a structure of high torsion flexibility, according to 4.2.3 (147), the same type of structural system is used in the two horizontal orthogonal directions.

(381) In each main horizontal direction, the main structural system shall consist of a single type in the vertical direction. Exceptions may be made for one or two levels at the top of multi-storey buildings if the total height of these levels is less than 10 % of the height of the building measured above the conventional fixation-clamping section.

(382) When configuring main structures with reinforced concrete walls, the end areas of reinforced concrete walls can be considered to form flat frames together with beams oriented perpendicular to the plane of the walls only if:

(vvvvvv) the end zones meet the specific requirements for pillars laid down in this technical regulation;

and

(wwwwww) the junction areas between the beams and the end areas of the walls shall meet the specific provisions for beam-pillar joints given in this technical regulation.

(383) Ground floor structures, with cantilevered pillars and articulated beams, where the average normalised axial compression effort in the pillars is less than 0.20, where the top ends of the pillars are connected by means of a floor with a rigid horizontal diaphragm behaviour, shall be designed on the basis of specific provisions. Steel sheet covering of various types is not a main structural component and does not perform the role of a horizontal diaphragm.

(384) Structures defined in (376) shall be made only of main structural components of unprestressed reinforced concrete. By way of exception, the following may be used:

(xxxxxx) pre-stressed slabs, which serve as a horizontal diaphragm;

(yyyyyy) prestressed beams, in buildings with no more than two above-ground levels, if prestressing is applied outside their critical areas and the lever arm of the shear force in the design seismic action is greater than or equal to 5.00 m.

(385) Structures made up of slabs, pillars and walls that are located in an area of the building, without perimeter frames (structure with a core of walls and slabs), fall into the category of structures with high torsion flexibility.

(386) In buildings located in areas with moderate or high seismicity, it is not permitted to use structural systems consisting of slabs, pillars and walls situated in a part of the building without perimeter frames.

(387) In interpreting the provisions of (385) and (386), the walls shall be deemed to be locally positioned if, in the horizontal plane, for any main orthogonal direction, the area of the convex polygonal envelope of all the walls parallel to that direction is less than 50 % of the level area.

(388) Main structural systems made of pillars and slabs are not permitted.

(389) The main structural systems of buildings fall within the types indicated in 5.2.2, (376) only if the following conditions are met:

(zzzzzz) the structures are carried out in accordance with the definition of the type of structural system given in 5.1.2, (370);

(aaaaaaa) pillars, beams and joints of frame-type structural systems shall comply with the specific provisions of this technical regulation and technical regulation NP 007;

(bbbbbbb) the coupling walls and beams of structural systems of wall, dual or inverted pendulum type meet the specific provisions of this technical regulation and the provisions of technical regulation CR2-1-1.1;

(ccccccc) pillars, beams and joints of dual systems or inverted pendulum type shall comply with the specific provisions of this technical regulation;

(ddddddd) the pillars of structural systems with cantilevered pillars shall comply with the specific provisions of this technical regulation.

(390) Main structures that do not fulfil the provision (389) may be designed seismically in accordance with the provision 5.2.1, (377).

(391) All main structural components, regardless of the type of structural system, shall be designed for the same class of ductility.

5.2.3 Optimal plastic mechanism

(392) For structures designed for the ductility class DCH or DCM, the favourable seismic response to the design seismic action corresponding to the ultimate limit state shall be conditioned by the formation of an optimal plastic mechanism with adequate energy dissipation capacity induced by horizontal seismic action.

(393) The structure shall be formed in such a way that the plastic deformations of the main structural components occur due to the exceeding of the specific yield deformation of the stretched longitudinal reinforcements as a result of the bending of the structural components, with or without axial force.

(394) In frame-type structural systems, the optimal plastic mechanism shall be formed by the development of plastic zones at the ends of the beams and at the base of the pillars, just above the conventional fixation-clamping section.

(395) In wall-type structural systems, the optimal plastic mechanism is formed by the development of plastic zones at the base of the walls, just above the conventional fixation-clamping section, and in the coupling beams, if any.

(396) In dual structural systems, the optimal plastic mechanism shall be formed by the development of plastic zones at the base of walls and pillars, just above the conventional fixation-clamping section, in the coupling beams, if any, and at the ends of the frame beams.

(397) In inverted pendulum structural systems, the optimal plastic mechanism shall be formed by the development of plastic hinges at the base of vertical structural elements, just above the conventional fixation-clamping section.

(398) In structural systems with cantilevered pillars, the optimal plastic mechanism shall be formed by the development of plastic hinges at the base of the pillars, just above the conventional fixation-clamping section.

(399) For the control of the development of the plastic mechanism, the design shall be carried out in accordance with the principles of the hierarchy of resistance capacities method - the capacity design method.

(400) Infrastructures and foundations shall be designed to respond elastically to the action of seismic design corresponding to the ultimate limit state.

(401) By way of exception from (400), in design situations where an elastic response of all elements of the infrastructure cannot be ensured, plastic bending deformations of these elements shall be permitted if access for inspection and repair is provided after the earthquake. When determining the configuration of the plastic mechanism, account shall be taken of the plastic deformation of these elements.

Note: Infrastructure elements that are in direct contact with the land on one or more sides do not fulfil the condition of providing access for inspection and repair.

5.2.4 Behaviour factors

5.2.4.1 Ultimate limit state

(402) The behaviour factor shall be chosen according to the capacity of the structure to dissipate earthquake-induced energy. The maximum values of the behaviour factor for horizontal seismic actions for checks at the ultimate limit state shall be chosen according to the provisions of Table 5.1, except as provided for in 5.2.1, (4) and (2).

Table 5.1 Maximum behaviour factor values for horizontal seismic actions

Type of structural system	Maximum value of the behaviour factor		
	Ductility class		
	DCH	DCM	DCL
Frame-type structural system	$4.50\alpha_u/\alpha_1$	$3.00\alpha_u/\alpha_1$	1.50
Structural wall system with insulated walls	$3.50\alpha_u/\alpha_1$	$3.00\alpha_u/\alpha_1$	1.50
Wall-type structural system with coupled walls	$4.00\alpha_u/\alpha_1$	$3.00\alpha_u/\alpha_1$	1.50
Dual structural system with predominant walls	$4.50\alpha_u/\alpha_1$	$3.00\alpha_u/\alpha_1$	1.50
Dual structural system with predominant frames	$4.50\alpha_u/\alpha_1$	$3.00\alpha_u/\alpha_1$	1.50
Inverted pendulum structural system	2.00	1.50	1.00
Structural system with cantilevered pillars*	2.50	2.00	1.00
Structural system with cantilevered pillars and a rigid diaphragm roof, single-level, where the average normalised axial compression effort in the pillars is less than 0.200	3.50	3.00	1.00

* Note: This is the case for structures of ground floor halls with articulated beams where the floor is made without a system of bracings placed in a horizontal plane capable of preventing the relative displacements between the upper ends of the pillars, taking into account the asynchronous nature of the seismic excitation at their base.

(403) By way of exception from (402), in the case of structures falling within the types indicated in:

(eeeeeee) 5.2.2, (376), (oooooo) - (ssssss)

or

(ffffff) 5.2.2, (376), (uuuuuu), having a single level at which the average normalised axial compression effort in the pillars is less than 0.20, which have high torsion flexibility as specified in 4.5.1.1, (276) or 4.2.3, (147), the maximum values of the behaviour factor for horizontal seismic actions shall be chosen in accordance with the provisions of Table 5.2.

Table 5.2 Maximum behaviour factor values for horizontal seismic actions for structural systems with high torsion flexibility

Type of structural system	Maximum value of the behaviour factor		
	DCH	DCM	DCL
Structural system with high torsional flexibility	2.50	2.00	1.00

(404) The value of the ratio between the horizontal force resistance of the structure and the horizontal force corresponding to the occurrence of the first plastic deformation in a main structural component, α_u/α_1, is limited above and below according to the relation:

$$1{,}0 \leq \alpha_u / \alpha_1 \leq 1{,}35 \qquad (5.2)$$

(405) Ratio value $\alpha/_u\alpha_1$, for buildings in class III or IV of importance and exposure to earthquake shall be determined according to the provisions of Table 5.1 or by non-linear static calculation, with the limitation indicated at (404).

Table 5.1 Values of the ratio α_u/α_1 for use in class III or IV buildings of importance and exposure to earthquakes

Type of structural system	α_u/α_1
Frame-type or dual-frame structural system with predominant frames	
Single-level buildings	1.10
Multi-storey buildings with a single opening:	1.20
Multi-storey buildings with several openings	1.30
Wall-type structural system or predominantly walled dual structural system	
Structural system of insulated walls with a maximum of two walls arranged in the orthogonal direction considered	1.00
Structural system of insulated wall type with more than two walls arranged in the considered orthogonal direction	1.10
Wall-type structural system with coupled walls, dual structural system with predominant walls	1.20

(406) For class I or II buildings of importance and earthquake exposure, the value of the ratio α_u/α_1 shall be determined by non-linear static calculation, with the limitation indicated at (404). If the ratio $\alpha/_u\alpha_1$ is not determined by non-linear static calculation, it shall be considered equal to 1.00.

(407) In the case of structures designed for ductility class DCM or DCH with reinforced concrete walls, the maximum value of the behaviour factor shall be multiplied by the factor k_w expressing the effect of the proportions of the walls on their plastic deformation capacity:

$$k_w = 1 \qquad \text{if } \alpha_0 \geq 2$$
$$k_w = (1+\alpha_0)/3 \geq 0{,}50 \qquad \text{if } \alpha_0 < 2 \qquad (5.3)$$

where:

α_0 the predominant ratio between the height of the walls and the length of their core, for the structure as a whole.

If the ratio of sides h_{wi}/l_{wi} does not differ significantly from one wall to another, α_0 it can be calculated with the relation:

$$\alpha_0 = \sum h_{wi} / \sum l_{wi} \qquad (5.4)$$

where:

h_{wi} the height of each wall '*i*';

l_{wi} the length of the cross-section of each wall '*i*', measured in the direction of calculation.

If the ratio of sides h_{wi}/l_{wi} differs significantly from one wall to another, the minimum value of the factor α_0 established for each wall shall be used to determine the factor k_w in the considered direction of the seismic action.

For the application of this provision, only reinforced concrete walls parallel to the direction of calculation are considered.

(408) In the case of an irregular building, the maximum value of the behaviour factor shall be reduced as set out in 4.5.1.1.

(409) A single behaviour factor value shall be used to determine the values of the reduced spectrum for any horizontal direction of calculation. In the case of buildings with a reinforced concrete structure having structural systems of different types in two horizontal directions, the maximum value of the behaviour factor is equal to the minimum of the values corresponding to the two types of structural systems.

(410) Buildings with main structures of the type indicated in 5.2.2, (376), (oooooo) - (uuuuuu), which have interior and/or exterior non-structural walls made of masonry and/or concrete, shall be designed considering a maximum value of the behaviour factor reduced by 20 % compared to the maximum value resulting from the application of the provisions of this paragraph, for the type of structural system used.

(411) Main structures of the type indicated in 5.2.2, (376), (oooooo) - (tttttt), which are made, in whole or in part, of prefabricated beams, pillars or walls, shall be designed considering a maximum value of the behaviour factor reduced by 30 % from the maximum value resulting from the application of the provisions of this paragraph, for the type of structural system used.

(412) The maximum value of the behaviour factor resulting from the application of the provisions of this paragraph shall be limited lower at 1.00.

5.2.4.2 Service limit state

(413) The maximum behaviour factor value for horizontal seismic actions for service limit state checks shall be 1.50 for buildings designed for ductility class DCM or DCH and 1.00 for buildings designed for ductility class DCL.

5.2.5 Local effects caused by interaction with non-structural components

(414) The main structural components shall be designed to meet the seismic design criteria of 5.3 in the event of loading with the connecting forces to the non-structural components.

(415) By establishing the position and composition of non-structural components such as rigid and in-plane action-resistant panels, framed by the main structural components, such as framed masonry walls, the following are avoided:

(ggggggg) introduction of structural irregularities in the horizontal or vertical plane;

(hhhhhhh) formation of the effect of a short beam or short pillar in adjacent structural components.

(416) Framed masonry walls are included in the calculation model of the structure if they are made as full panels or panels with a door or window opening for which a system of compressed diagonals can be identified that transmits the forces to the perimeter structural components.

(417) In frame structures, the calculation of the structure considers the possible adverse effects of interaction with framed masonry panels regarding:

(iiiiiii) change in structural regularity in the horizontal and/or vertical plane;

(jjjjjjj) the change in the effects of seismic action due to the increase in the overall torque caused by the change in the position of the centre of rigidity in relation to the centre of mass;

(kkkkkkk) modification of the geometrical scheme for calculating the structure by modifying the lengths and/or the supporting conditions of the structural components;

(lllllll) modification of the dynamic properties of the building;

(mmmmmmm) the emergence of local efforts caused by the interaction between the frame and the panel, in particular at the joints of the frame and at the corners of the panel.

(418) In the design of the main structure, the possible favourable effects due to the presence of framed masonry panels are not taken into account.

(419) The design values of the forces acting in the plane of the framed masonry panels shall be calculated by considering the assembly formed by the frame and the masonry panels modelled as a triangulated system with diagonals articulated at the ends. The active width of the diagonal of the masonry panel, d_p, shall be established by the following relation:

$$z = \frac{z_{¿} + z_{inf}}{2} \quad d_p = 0{,}1 D_p \tag{5.1}$$

where:

D_p the length of the diagonal of the masonry panel.

(420) In the case of the cantilevered pillars structural system, with or without a rigid diaphragm to actions in its plane, there shall be no non-structural partition walls or enclosures made of concrete or masonry, in direct contact with the pillars in such a way as to restrict their oscillation. Where such non-structural components are used, they shall be insulated from the main structure by joints large enough to permit free oscillation of the pillars parallel to the plane of the walls and shall be designed to avoid loss of their own stability under the seismic design action corresponding to the ultimate limit state.

5.2.6 Foundations and infrastructures

(421) When designing infrastructures and foundations, the provisions of the specific technical regulations together with the additional provisions given in this chapter shall apply.

(422) The main structures of reinforced concrete are composed of foundation systems such as:

(nnnnnnn) surface foundations, as defined in technical regulation NP 112;

or

(ooooooo) piles foundations, as defined in technical regulation NP 123.

Note: The generic designation ‘pilots’ shall also be understood as ‘bars’ if they meet the conditions laid down in technical regulation NP 123.

(423) The components of foundations and/or infrastructures that balance the effort generated by seismic action shall be designed as main structural components.

(424) Notwithstanding the provisions of the technical regulation ‘Rules on the design of surface foundations’, reference number NP 112, paragraph I.6.1.1., paragraph 3.2, in the case of single-level buildings, the foundation shall be dimensioned in such a way that the compressed area of the foundation base in the seismic grouping is greater than or equal to 0.50 of the total area of the foundation base. In the case of applying this exception, the effects of the actions at the base of the foundation shall be determined by considering the soil-structure interaction.

(425) Additional provisions on the design of infrastructures and foundations for constructions with reinforced concrete walls are given in technical regulation CR 2-1-1.1.

(426) Further provisions on the design of building infrastructures and foundations in reinforced concrete frames are given in technical regulation NP 007.

5.2.7 Precast structures

(427) When designing prefabricated structures, the provisions of the specific technical regulations are used together with the additional provisions given in this technical regulation.

(428) Prefabricated structures shall be constructed in such a way as to meet the general seismic design requirements set out in Chapter 4.

(429) In the case of buildings designed for ductility classes DCH or DCM, continuity of prefabricated elements in critical areas shall be achieved only by wet reinforced

concrete joints or by joint processes with similar behaviour to wet joints, as demonstrated by technical approval meeting the provisions of 1.1, (14).

(430) The reinforcement arranged in the joining areas between the prefabricated elements shall be designed for an exclusively elastic response to the incidence of design seismic action corresponding to the ultimate limit state. The following may be an exception:

(ppppppp) vertical reinforcements of the pillars and walls that deform plastically as a result of their bending, according to the configuration of the optimal plastic mechanism, if the full transmission of the efforts under the seismic action of the design is ensured, without degradation of the joint;

(qqqqqqq) the horizontal reinforcements of the beams and the horizontal or inclined reinforcements of the coupling beams that deform plastically as a result of their bending, according to the configuration of the optimal plastic mechanism, if the full transmission of the efforts under the design seismic action is ensured, without degradation of the joint.

(431) In the case of joints between prefabricated elements where it is necessary to balance the tensile forces in the reinforcements, the composition measures of prefabricated elements and joints shall achieve a structural behaviour similar to that of monolithic structures.

(432) The floor consisting partially or entirely of prefabricated panels shall be constructed to ensure its function as a rigid and resistant diaphragm in its plane.

5.2.8 Modelling for calculation

(433) For buildings with reinforced concrete structure, when calculating the design value of seismic action, the fraction of the critical damping of the building, ξ, for all modes of vibration, shall be taken to be equal to 5 %:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad \xi=5\ \% \tag{5.1}$$

(434) By way of exception from (433), for non-linear dynamic calculation the fraction of critical damping shall be assumed to be equal to 2.5 % for all modes of vibration.

(435) Modelling the rigidity of structural components shall be based on the geometric characteristics of their rough concrete cross-sections, taking into account the effects of concrete cracking.

(436) Flexural rigidity for walls, long beams, coupling beams, pillars and slabs of reinforced concrete shall be considered equal to half of the value corresponding to the uncracked gross cross-section.

(437) The rigidity to axial force in their plane of the component slabs shall be considered equal to 0.70 of the value corresponding to the uncracked gross cross-section.

(438) By way of exception from (436) and (437), when calculating the stress in reinforced concrete structures, different values of the reduction factor for flexural rigidity and axial force due to concrete cracking may be chosen, if they are determined on the basis of the calculation models given in SR EN 1992-1-1, based on the actual composition of each element and the expected stress state.

(439) The values of the average modulus of elasticity of concrete, E_{cm}, for use in modelling the structure for checking horizontal displacements according to 4.3.1.2.2 and 4.3.2.1, shall be established in accordance with the provisions of SR EN 1992-1-1 and shall be limited to the upper values indicated in Table 5.4.

Table 5.1 The average modulus of elasticity of concrete for horizontal displacement checks

	Concrete strength class						
	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
$E_{cm}(N/mm^2)$	30000	31000	33000	34000	35000	36000	37000

5.3 Seismic performance criteria

5.3.1 General information

(440) The provisions of this paragraph shall apply to the main structure with a role in balancing seismic action.

(441) In the seismic design of reinforced concrete structures, the provisions given in this chapter shall apply together with the specific provisions of the other technical regulations for the design of reinforced concrete buildings, according to 5.1.1, (369).

5.3.2 Resistance

(442) Buildings with reinforced concrete structures shall be constructed in such a way as to meet the condition of resistance to horizontal actions laid down in 4.3.1.1. (199) and (201).

(443) The design value of the resistance capacity shall be greater than or equal to the design value of the effort in the section considered. This condition shall be fulfilled for all main structural components along their entire length.

(444) The main structural components subjected to bending, with or without axial force, and shear force shall be such that the conditions are met:

$$M_{Rd} \geq M_{Ed} \tag{5.1}$$

$$V_{Rd} \geq V_{Ed} \tag{5.2}$$

where

M_{Rd} design value of the bending strength;

M_{Ed} design value of the bending moment;

V_{Rd} design value of the strength capacity to the shear force;

V_{Ed} design value of the shear force.

(445) In the case of main structural components subjected to bending with axial force, the design value of the resistance capacity to the bending moment shall be determined taking into account the design value of the axial force. The assessment shall be made separately for each direction and sense of seismic action.

(446) The verification of reinforced concrete elements under oblique eccentric compression can be performed in a simplified manner in each main direction by

considering the value of the direct eccentric compression resistance capacity reduced by 30 %.

(447) Structural components shall be constructed in such a way that the failure of the bending sections, with or without axial force, does not occur by crushing the compressed concrete before the yielding of the stretched longitudinal reinforcement. This provision applies to buildings designed for the ductility class DCH, DCM or DCL.

(448) In the case of main structural components subjected to axial centric compressive force, the resistance condition shall be ensured by limiting the normalized axial stress in accordance with the provisions of this technical regulation.

(449) Stability, resistance and rigidity to horizontal seismic actions of structures shall not be ensured by the torsion response of structural components. The torsional strength and rigidity of structural components shall be neglected in seismic design. Exceptions may be made for some inverted pendulum structural systems where the torsion response of structural components is necessary to ensure stability, resistance and rigidity to horizontal seismic actions, and the torsion resistance capability must be explicitly verified.

(450) The main structures shall be made so that at each beam-pillar joint the following condition is met:

$$\sum M_{Rd,c} \geq \gamma_{Rd} \sum M_{Rd,b} \tag{5.3}$$

where:

$\sum M_{Rd,c}$ the sum of the design values of the axial bending resistance capacities of the pillars entering the joint, calculated in the considered direction, in the sections adjacent to the joint;

Note: The minimum values of the axial bending strength of the pillars corresponding to the possible variation of the axial forces in the seismic grouping shall be taken into account in this verification.

$\sum M_{Rd,b}$ the sum of the design values of the bending strengths of the beams entering the joint in the direction considered, in the sections adjoining the joint;

γ_{Rd} partial safety coefficient evaluating the uncertainties in the resistance capacity calculation model, mainly due to the post-elastic steel consolidation effect:

$$\begin{aligned} \gamma_{Rd} &= 1{,}35 \; for\ DCH \\ \gamma_{Rd} &= 1{,}25 \; for\ DCM \end{aligned} \tag{5.4}$$

Note: The pillars for which the provision of (450) is not fulfilled are considered secondary structural components in the design.

(451) By way of exception from (450), condition (5.3) may not be fulfilled in single-storey buildings and at the top end of the top-level pillars of multi-storey buildings.

(452) In the case of structures with beams arranged parallel to several horizontal directions, the condition (5.3) shall be verified separately for each of these directions and for each direction of seismic action in the direction considered, taking into account the composition of the bending moments in the beams at the joint.

(453) In the case of structures with beams arranged parallel to a single horizontal direction, the condition (5.3) shall be verified for that direction, for each of the two distinct directions of seismic action.

(454) Under the conditions referred to in (450), (452), and (453), condition (5.3) shall also be verified in the case of the intersection between the beams and the walls, when the beams are parallel to the plane of the core of the walls.

(455) By way of exception from (450), it is not mandatory to fulfil the condition (5.3) at the beam-pillar joints of the dual structures with predominant walls, in their direction, if it is verified by non-linear static calculation and/or non-linear dynamic calculation to avoid the formation of a plastic storey mechanism at a horizontal displacement value equal to the displacement caused by the design seismic action corresponding to the ultimate limit state multiplied by 1.50.

5.3.3 Ductility

(456) Buildings with reinforced concrete structure shall be constructed in such a way as to meet the conditions of ductility under horizontal seismic action given at 4.3.1.2.

(457) The design value of the relative displacement level allowed for checks at the ultimate limit state shall be determined as set out in 4.3.1.2.2, (212).

(458) By way of exception from (457), in the case of main structures where above the conventional embedding level there are slabs resting directly on the pillars, the design value of the relative displacement level allowed for checks at the ultimate limit state, $d_{Rd,r}^{SLU}$, is equal to $0.015h_s$, where h_s is the total story height.

(459) The displacement amplification factor for the ultimate limit state, c, shall be determined using the following relation:

$$\begin{cases} c = 1 + 2\left(1 - 1{,}1\dfrac{T_1}{T_C^{SLU}}\right)\left(\dfrac{\sqrt{q\,T_C^{SLU}}}{1{,}65} - 1\right) \\ c \geq 1 \\ c \leq \dfrac{\sqrt{q\,T_C^{SLU}}}{1{,}65} \end{cases} \tag{5.1}$$

where

T_1 the fundamental vibration period of the building for oscillations in the horizontal direction of calculation;

T_C^{SLU} corner period (control) of the horizontal acceleration spectrum for design checks at SLU.

q behaviour factor of the structure.

5.3.4 Stability

(460) Reinforced concrete buildings shall be constructed in such a way as to meet the conditions of stability under seismic action given at 4.3.1.3.

5.3.5 Rigidity

(461) Reinforced concrete buildings shall be constructed in such a way as to meet the conditions of rigidity under horizontal seismic action given at 4.3.2.1.

(462) The design value of the permissible level relative displacement shall be determined in accordance with the provisions of 4.3.2.1, (238).

5.4 Stresses design values

(1) This chapter contains provisions on the determination of the design values of the forces that develop in the main structural components for strength checks.

5.4.1 Buildings designed for DCH or DCM ductility class

(463) The design value of an effort caused by seismic action is the maximum value of that effort that develops as a result of the incidence of seismic design action. Exceptions are the areas where plastic deformations develop from bending, according to the configuration of the optimal plastic mechanism, where the design value of the bending moment is the value corresponding to the load of the structure with the design seismic action.
(464) The design values of the efforts likely to cause brittle fractures of structural elements shall be determined taking into account the uncertainty of the assessment by multiplying them by a partial safety coefficient greater than one.

(465) When establishing the design effort values, the maximum bending moments that develop in the plastic zones shall be determined by multiplying the design values of the bending strength capabilities by a partial safety coefficient.

(466) The design effort values shall be determined by:

(rrrrrr) transforming the efforts resulting from the calculation of the structure by a linear static calculation method, in order to quantify the non-linearity of the structural response caused by the design seismic action, in accordance with the principles of the resistance capacity hierarchy method;

or

(sssssss) directly, by non-linear calculation.

(467) The determination of the design values of the efforts of the main structural components, based on the efforts resulting from the calculation of the structure by a linear static calculation method, shall be carried out according to the provisions of 5.4.1.1, 5.4.1.2, 5.4.1.3, 5.4.1.4, 5.4.1.5 and 5.4.1.6.

(468) In the case of buildings for which the calculation of the structure is carried out by a linear static calculation method, when determining the design values, redistributions of effort between the main structural elements are allowed according to the provisions of 5.4.1.7.

(469) The determination of the design effort values based on the effort resulting from the calculation of the structure by the non-linear static calculation method shall be carried out in accordance with the provisions of 5.4.1.8.

5.4.1.1 Beams

5.4.1.1.1 Bending moments

(470) The design values of the bending moments in the plastic zones of the beams are equal to the values obtained from the calculation of the structure in the seismic combination.

(471) The design values of the bending moments in the elastic response zone of the beams shall be established from the beam balance in the situation of the formation of the optimal plastic mechanism, also considering the loads acting transversely on the axis of the beam in the seismic grouping.

5.4.1.1.2 Shear forces

(472) The design values of the shear forces in the beams shall be established from the equilibrium of the beam in the event of the formation of the optimal plastic mechanism, also considering the loads acting transversely on the axis of the beam in the seismic grouping.

(473) The calculation of the design values of the shear forces shall be made separately for each beam span and for each direction of seismic action.

(474) The values of the maximum bending moments that load the beam at the ends in the event of the formation of the plastic mechanism, $M^i_{d,b}$, shall be calculated with the relation:

$$M^i_{d,b} = \gamma_{Rd} M^i_{Rd,b} \min\left(1, \frac{\sum M_{Rd,c}}{\sum M_{Rd,b}}\right) \quad (5.1)$$

where:

$M^i_{Rd,b}$ design value of the bending resistance capacity of the beam at end *i*, for the direction of rotation corresponding to the direction of action of the horizontal forces;

$\sum M_{Rd,b}$ sum of the design values of the bending strength capacities of the beams entering the end joint *i* of the beam, in the direction of seismic action, for the direction of rotation corresponding to the direction of action of the horizontal forces;

$\sum M_{Rd,c}$ sum of the design values of the bending resistance capacities with axial force of the pillars entering the joint adjacent to the end *i* of the beam, for the direction of rotation corresponding to the direction of action of the horizontal forces, calculated considering the design values of the axial force in the pillar for the considered direction of seismic action;

γ_{Rd} partial safety coefficient established in accordance with the relation 5.3.2.1(5.4).

(475) The design values of the shear forces shall be limited to not less than the values corresponding to those resulting from the calculation of the structure in the seismic combination, where the part caused by the seismic action is multiplied by γ_{Rd}.

5.4.1.2 Pillars.

5.4.1.2.1 Axial forces

(476) The design value of the axial force in the pillars shall be determined from the equilibrium of the pillar in the situation of forming the plastic mechanism considering:

(ttttttt) the shear forces in the beams associated with their loading with the maximum bending moments caused by horizontal seismic action;

(uuuuuuu) shear forces in beams or plates from gravitational actions in the seismic combination of actions;

(vvvvvvv) the weight of the pillar;

(wwwwwww) other forces from the seismic group directly loading the pillar.

(477) If, according to the configuration of the plastic mechanism, plastic zones are developed at both ends of the beams and the design values of the capable moments in the beams do not exceed by more than 10 % the values of the bending moments resulting from the calculation of the structure, in each plastic zone, the design value of the axial force may be considered equal to the value of the axial force resulting from the calculation of the structure in the seismic grouping.

(478) Further provisions on the determination of design values of axial forces in pillars are given in technical regulation NP 007.

5.4.1.2.2 Bending moments

(479) The design values of the bending moments in the plastic zones of the pillars, according to the configuration of the optimal plastic mechanism, are equal to the values of the bending moments resulting from the calculation of the structure in the seismic combination.

(480) The design values of the bending moments in the pillars, in the elastic response area, shall be established from the balance of the pillars in the situation of the formation of the plastic mechanism, considering also the loads acting transversely on the axis of the pillar in the seismic grouping, if any.

5.4.1.2.3 Shear forces

(481) The design values of the shear forces in the pillars shall be established from the equilibrium of the pillar in the event of the formation of the plastic mechanism, also considering the loads acting transversely on the axis of the pillar in the seismic grouping, if any.

(482) The calculation of the design values of the shear forces in the pillars shall be made at each level, distinct for each direction of seismic action and for each direction of calculation.

(483) The values of the maximum bending moments that load the pillar at the ends in the event of the formation of the plastic mechanism, $M^i_{d,c}$, shall be calculated with the relation:

$$M^i_{d,c} = \gamma_{Rd} M^i_{Rd,c} \min\left(1, \frac{\sum M_{Rd,b}}{\sum M_{Rd,c}}\right) \quad (5.1)$$

where:

$M^{i}_{Rd,c}$ the design value of the bending resistance capacity of the pillar at end i, for the direction of rotation corresponding to the direction of action of the horizontal forces;

$\sum M_{Rd,c}$ sum of the design values of the axial bending resistance capacities of the pillars entering the joint adjacent to the end *i* of the considered pillar, for the direction of rotation corresponding to the direction of action of the horizontal forces, calculated considering the design values of the axial force in the pillar;

$\sum M_{Rd,b}$ sum of the design values of the bending capacities of the beams entering the end joint *i* of the pillar, for the direction of rotation corresponding to the direction of action of the horizontal forces.

γ_{Rd} partial safety coefficient established in accordance with the relation 5.3.2.1(5.4).

(484) The design values of the shear forces shall be limited to the lower values resulting from the calculation of the structure in the seismic grouping, where the part caused by the seismic action is multiplied by γ_{Rd}.

(485) In the case of pillars in direct contact with rigid and resistant non-structural components, if the height of the panel is lower than the free height of the level, the design value of the shear force in the pillar shall be determined by considering a calculation model with plastic zones developed at the two ends of the gap.

Note: Such panels are, for example, masonry parapets.

5.4.1.3 Joints

(486) This paragraph contains provisions regarding the calculation of the design values of the shear forces loading the beam-to-pillar joints.

(487) The design value of the shear force at the joint shall be determined from its equilibrium in the event of forming the plastic mechanism, separately for each direction of seismic action and for each calculation direction.

(488) The design value of the shear force in the joint can be determined using the following simplified formulas:

(xxxxxxx) for all joints except the end joints:

$$V_{Ed,j} = \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} - V_{Ed,c} \tag{5.1}$$

(yyyyyyy) for end-joints:

$$V_{Ed,j} = \gamma_{Rd} A_{s1} f_{yd} - V_{Ed,c} \tag{5.2}$$

where:

A_{s1}, A_{s2} the areas of the stretched reinforcements at the top and bottom, respectively, of the beams entering the joint in the considered direction of the seismic action, determined according to the direction of the seismic action;

$V_{Ed,c}$ the design value of the shear force in the pillar above the joint for the direction and sense considered of the seismic action;

γ_{Rd} partial safety coefficient established in accordance with the relation 5.3.2.1(5.4).

5.4.1.4 Coupling walls and beams

(2) The design effort values for walls and coupling beams shall be determined in accordance with CR 2-1-1.1.

5.4.1.5 Diaphragms

(489) The provisions of this paragraph shall apply to the design effort in the diaphragms constituted by the floors subjected to stress by loads in their median plane.

(490) The design values of the efforts in the diaphragms are equal to the efforts associated with the mobilization of the overall plastic mechanism of the structure multiplied by a partial safety coefficient $\gamma_{Rd}=1{,}20$.

(491) The efforts in a diaphragm shall be determined by considering its equilibrium under the action of horizontal forces and the design values of the shear forces in the vertical structural elements that load the diaphragm in the horizontal direction.

5.4.1.6 Infrastructures and foundations

(492) The design values of the forces and deformations in the elements of the infrastructure and foundations shall be determined by considering their equilibrium under the forces connecting with the superstructure and the forces bearing on the ground.

(493) When designing the infrastructure and foundations and when checking the foundation ground, the maximum values of the efforts related to the superstructure, corresponding to the situation of the formation of the optimal plastic mechanism under the seismic design action, and the loads acting directly on them, shall be considered.

(494) The design values of the efforts and deformations in the infrastructure elements and foundations shall be obtained considering the soil-structure interaction.

(495) For the calculation of the efforts in infrastructure and foundations, in the case of buildings with surface foundations, the ground support shall be minimally modelled by unidirectional springs required only at compression. The equivalent rigidity of springs shall be established in accordance with the characteristics of the foundation ground for values of its specific deformations corresponding to the response of the structure under the design seismic action, in the seismic grouping, for the dynamic loading regime.

(496) In the case of buildings with piles foundations, the design values of the forces connecting each pile to the rest of the structure, corresponding to the overall response of the building under design seismic action, in the seismic grouping, shall be taken into account when checking each pile.

(497) For the structural calculation taking into account the terrain-structure interaction, the piles shall be modelled minimally by unidirectional springs oriented in a vertical direction. The equivalent springs rigidity shall be determined in accordance with the characteristics of the pile and the terrain for values of vertical displacements corresponding to the response of the structure under seismic design action, in the seismic grouping, for dynamic loading.

(498) In the case of elements with isolated foundations, the design values of the efforts at the base of the plastic zone of the vertical structural components, E_{Fd}, can be determined by transforming the effort values resulting from the linear static calculation with the equation:

$$E_{Fd} = E_{F,G} + \gamma_{Rd} \Omega E_{F,E} \qquad (5.1)$$

where:

$E_{F,G}$ the sectional effort produced by actions other than seismic action that are included in the seismic grouping;

$E_{F,E}$ sectional effort resulting from the calculation at seismic design action;

Ω bending overstrength factor of the wall;

γ_{Rd} partial safety coefficient taking into account the uncertainty contained in the resistance capacity assessment, to be chosen according to the relation 5.3.2.1(5.4).

(499) Further provisions on the establishment of design effort values for buildings with structures in reinforced concrete frames are given in technical regulation NP 007.

(500) Further provisions on the establishment of design effort values for buildings with reinforced concrete walls are given in technical regulation CR 2-1-1.1.

5.4.1.7 Efforts redistribution

(501) The redistribution of the effort resulting from the calculation of the structure by a linear static calculation method shall only be applied if, in the modelling of the structure for calculation, the values of the rigidity reduction factors in accordance with 5.2.8, (436), and (437) have been taken into account.

(502) Redistribution shall be carried out only for structural elements with a ductile response to seismic action. Redistribution applies only to the part of the structure in which the overall plastic mechanism is formed.

(503) Effort redistribution shall be carried out for:

(zzzzzzz) considering the impact of changes in rigidity properties of structural concrete elements after cracking on the state of effort in the structure;

(aaaaaaaa) uniformization of composition and reinforcement solutions;

(bbbbbbbb) achieving a state of effort associated with the mobilization of the plastic mechanism that favours the ductile response of the structure and the avoidance of fragile fractures.

(504) Redistribution shall be carried out in compliance with the conditions of overall and local equilibrium. The overall turning moment of the structure at horizontal actions shall remain unchanged after redistribution.

(505) Provisions on the redistribution of effort in reinforced concrete frame structures are given in technical regulation NP 007.

(506) Provisions regarding the redistribution of efforts for reinforced concrete wall constructions are given in the technical regulation CR 2-1-1.1.

5.4.1.8 Static non-linear calculation method

(507) The effort design values shall be established on the basis of the corresponding efforts resulting from the static nonlinear calculation of the structure as a whole, for the two modes of distribution of seismic forces given in 4.5.2.

(508) Verification of the strength capacity of the elements according to 5.3.2, (443), shall be carried out separately for each mode of seismic force distribution.

(509) The effort design values that may cause ductile failures are the effort values corresponding to the movement requirement to the ultimate limit state.

> Note: This is the general case of the flow of longitudinal reinforcement due to the action of bending moment (except for longitudinally sub-reinforced or over-reinforced elements which may have fragile yields from the bending moment) and of the flow of diagonal reinforcements of the coupling beams.

(510) The effort design values that may cause fragile type failures are the effort values corresponding to the movement requirement to the ultimate limit state multiplied by 1.50.

> Note: In general, ductile-type yields can be obtained by the action of the bending moment, under the conditions of ensuring the rotational ductility of the structural element, or by the flow of the diagonal reinforcements of the coupling beams. All other ways of breaking must be considered fragile in design.

(511) The design values of the infrastructure and foundations efforts, for all component elements, are the values of the effort corresponding to the requirement to move the structure to the ultimate limit state multiplied by 1.50.

> Note: This provision applies to all structural elements located below the conventional fixation-clamping section.

(512) By way of exception from (508), verification of the bending strength of the main structural components in areas where plastic deformations occur in accordance with the configuration of the optimal plastic mechanism can be done by checking the strength of the structure to horizontal actions according to 5.3.2, (442).

> Note: In the case of non-linear static calculation, this verification shall be carried out by comparing the force F_y of the bi-linear response law with the basic shear force determined in accordance with the provisions of chapter. 4. For this verification, the force F_y shall be determined using the material resistance design values.

5.4.2 Buildings designed for DCL ductility class

(513) The design effort values shall be established by transforming the efforts resulting from the calculation of the structure by a linear calculation method.

(514) The design values of bending moments and shear forces in the beams are equal to those resulting from the calculation of the structure by a linear calculation method in the seismic combination.

(515) The design values of bending moments and axial forces in the pillars are equal to those resulting from the calculation of the structure by a linear calculation method in the seismic combination.

(516) The design values of the shear forces in the pillars and joints are equal to the shear forces in the calculation of the structure by a linear calculation method in the seismic combination, multiplied by 1.20:

$$V_{Ed}=1{,}2\,V'_{Ed} \qquad (5.1)$$

where:

V_{Ed} design value of the shear force;

V'_{Ed} the value of the shear force resulting from the calculation of the structure in the seismic grouping.

(517) The design values of the efforts in the diaphragms, constituted by the floors subjected to stress by loads parallel to their median plane, are equal to the efforts resulting from the linear static calculation of the structure, multiplied by 1.20.

(518) The design values of the efforts in infrastructure and foundations are equal to the efforts resulting from the linear static calculation, multiplied by 1.20.

(519) Provisions regarding the determination of the design values of the efforts that develop in walls, coupling beams, horizontal diaphragms, foundations and infrastructures, as the case may be, in structures with reinforced concrete walls are given in the technical regulation CR 2-1-1.1.

5.5 Resistance capacity

5.5.1 Beams

(3) The bending moment resistance and shear force capacity of the beams shall be determined on the basis of the provisions of SR EN 1992-1-1, together with the additional provisions given in this paragraph.

5.5.1.1 Bending moment

(520) When calculating the bending moment resistance capacity of the monolithically cast beams together with the slab, all reinforcements in the beam and reinforcements in the slab parallel to the beam, arranged in the active area of the slab, shall be considered. The unit tensile effort in the reinforcements shall be determined according to their anchorage length in relation to the calculation section.

(521) The active plate width of the beams working together with the plate, to the left and right of the core, b_{eff}, shall be determined with the relation:

$$b_{eff}=b_w+min¿¿, d_0, 0{,}125\,l_{cl}¿ \qquad (5.1)$$

where:

b_w width of the beam core;

b_{eff} the width of the active plate;

h_f plate thickness;

d_0 the corresponding width of the slab for the beam in question, resulting from the geometry of the floor, taking into account the parallel beams and the actual edge of the slab;

l_{cl} free opening of the beam.

(522) The active plate width of the beams working together with the plate, on one side of the core, b_{eff}, shall be determined with the relation:

$$b_{eff}=b_w+min¿¿, d_0, 0{,}08\,l_{cl}¿ \quad (5.2)$$

where:

b_w width of the beam core;

b_{eff} the width of the active plate;

h_f plate thickness;

d_0 the corresponding width of the slab for the beam in question, resulting from the geometry of the floor, taking into account the parallel beams and the actual edge of the slab;

l_{cl} free opening of the beam.

(523) When calculating the bending resistance capacity of beams working with fully or partially prefabricated plates, the contribution of the reinforcements in the plate placed within the active zone shall be determined by considering the method of connecting the plates with the beam.

(524) When calculating the bending resistance capacity of beams working with prestressed plates, account shall be taken of the compressive effort transmitted to the beams by prestressing the plates.

5.5.1.2 Shear force

(525) In critical areas of the beams, the inclination of the compressed diagonals in the lattice beam pattern is considered equal to 45°.

(526) In accordance with the provision of (525), the shear force resistance capacity of the beams is determined by the relation:

$$V_{Rd}=minim\left(V_{Rd,max},V_{Rd,s}\right) \quad (5.1)$$

where

$V_{Rd,max}$ the shear force in the beam corresponding to the failure of the compressed diagonal of the concrete, in N;

$$V_{Rd,max}=0{,}75\,b_w d\sqrt{f_{cd}} \quad (5.2)$$

b_w the width of the core of the cross-section of the beam;

d the useful height of the cross-section of the beam;

f_{cd} design value of concrete compressive strength, in N/mm^2;

$V_{Rd,s}$ the shear force in the beam which can be suspended by means of the transverse reinforcement;

$$V_{Rd,s}=0{,}9\,A_{sw}^{tot} f_{yd} \quad (5.3)$$

A_{sw}^{tot} the total area of the vertical ramifications of the transverse reinforcement of the beam, intersected by a crack inclined at 45^0;

f_{yd} design value of the flow limit of the steel from which the transverse reinforcements are made.

5.5.2 Pillars

5.5.2.1 Bending moment and axial force

(4) The simultaneous action of axial force and bending moment shall be considered for each seismic design combination.

5.5.2.2 Shear force

(527) In the critical areas of the pillars, the inclination of the compressed diagonals in the truss beam pattern is considered equal to 45°.

(528) In accordance with the provision of (527), the shear force resistance capacity of the pillars shall be determined with the relation:

$$V_{Rd} = minim\left(V_{Rd,max}, V_{Rd,s}\right) \quad (5.1)$$

where

$V_{Rd,max}$ the shear force in the pillar corresponding to the failure of the compressed diagonal of the concrete, in N;

$$V_{Rd,max} = 0{,}83\, b_w d \sqrt{f_{cd}} \quad (5.2)$$

b_c the width of the cross-section of the pillar;

d the effective height of the cross-section of the pillar;

f_{cd} design value of concrete compressive strength, in N/mm^2;

$V_{Rd,s}$ the shear force in the pillar that can be suspended by means of the transverse reinforcement;

$$V_{Rd,s} = 0{,}9\, A_{sw}^{tot} f_{yd} \quad (5.3)$$

A_{sw}^{tot} the total area of the ramifications of the transverse reinforcement of the pillar parallel to the direction of calculation, intersected by a crack inclined at 45°;

f_{yd} design value of the flow limit of the steel from which the transverse reinforcements are made.

5.5.3 Beam-pillar joints

(529) A beam-pillar joint is considered confined by the effect of transverse beams entering the joint in a direction perpendicular to the direction of seismic action if all the following conditions are met:

(cccccccc) from both sides of the joint, the transverse beams extend over a length greater than the height of their cross-section;

(dddddddd) the width of the beams is greater than ¾ of the cross-sectional dimension of the pillar perpendicular to the direction of seismic action;

(eeeeeeee) the beams meet the minimum reinforcement requirements for the critical area of the main seismic components given in this chapter.

The transverse beams with the static cantilever scheme also participate in the confinement of the joint.

(530) A beam-pillar joint is considered confined by the effect of the pillars entering the joint if the following conditions are cumulatively met:

(ffffffff) from the upper face of the joint, the pillar extends over a length greater than the height of its cross-section;

(gggggggg) the top pillar meets the minimum conditions for the critical area of the main seismic components given in this chapter;

(hhhhhhhh) the cross-section of the pillar above the joint meets the requirements of this technical regulation regarding the reduction of sections from one level to another.

The joint confinement also involves the extensions of the pillars above the joints.

(531) A beam-pillar joint is considered confined by the effect of longitudinal beams entering the joint in the direction of seismic action if the following cumulative conditions are met:

(iiiiiiii) from both sides of the joint, the longitudinal beams extend over a length greater than the height of their cross-section;

(jjjjjjjj) the beams meet the minimum reinforcement requirements for the critical area of the main seismic components given in this chapter.

In the confinement of the joint, the longitudinal beams with the static cantilever scheme also participate.

(532) The design value of the shear force resistance capacity corresponding to the crushing of concrete in the compressed diagonal, in N, shall be determined using the following equations:

(kkkkkkkk) for joints confined by the effect of pillars, longitudinal and transverse beams entering the joint:

$$V_{Rd} = 1{,}75\, b_j h_j \sqrt{f_{cd}} \tag{5.1}$$

(llllllll) for joints confined by the effect of longitudinal or transverse pillars and beams entering the joint or for joints confined only by the effect of longitudinal and transverse beams entering the joint:

$$V_{Rd} = 1{,}25\, b_j h_j \sqrt{f_{cd}} \tag{5.2}$$

(mmmmmmmm) for joints confined only by the effect of pillars, longitudinal beams or transverse beams entering the joint:

$$V_{Rd} = 1{,}00\, b_j h_j \sqrt{f_{cd}} \tag{5.3}$$

(nnnnnnnn) for joints unconfined by the effect of pillars and beams entering the joint:

$$V_{Rd} = 0{,}75\, b_j h_j \sqrt{f_{cd}} \tag{5.4}$$

where:

h_j the dimension of the horizontal section through the joint measured in the direction of seismic action is to be taken as equal to the height of the cross-section of the pillar;

b_j design width of the joint;

$$b_j = min(b_c; b_w + 0,5 h_j; b_w + 2e) \quad (5.5)$$

b_w the minimum width of the core of the beams aligned with the direction of seismic action;

b_c the dimension of the cross-section of the pillar measured perpendicular to the axis of the beam;

e the minimum horizontal distance between the lateral face of the beam and the lateral face of the pillar on the same side of the beam's core, measured perpendicularly to the axis of the beam;

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

5.5.4 Coupling walls and beams

(533) The bending moment resistance, with or without axial force, and the shear force resistance of the coupling walls and beams shall be determined in accordance with technical regulation CR 2-1-1.1.

5.5.5 Slab floors

(534) Slab floors without transverse reinforcement shall be designed so that the unit penetration effort along the control perimeter, generated by loads perpendicular to the plane of the slab in the seismic grouping, is less than or equal to 0.40 of the value $V_{Rd,c}$ established in accordance with SR EN 1992-1-1.

(535) In the case of slab floors with transverse reinforcement, the penetration shearing resistance capacity and the calculation perimeter, beyond which no penetration reinforcements are required, shall be determined in accordance with SR EN 1992-1-1, considering 0.40 of the value $V_{Rd,c}$ established in accordance with the provisions of this standard.

(536) When checking the penetration of transversely reinforced slabs, only the transverse reinforcements anchored to the potential position of the penetration surface are considered.

5.6 Distortion capacity

(537) The permissible values of the rotations, θ_{Rd}^{SLU}, shall be determined by calculation based on the composition and reinforcement characteristics, according to the provisions of SR EN 1998-3, for the significant degradation limit state, with the method based on the confinement model from SR EN 1992-1-1.

(538) By way of exception from (537), for main structural components made up in accordance with this technical regulation, where rotations caused by design seismic action are determined by the linear static calculation method, the permissible values of rotations, θ_{Rd}^{SLU}, for checks at the ultimate limit state, may be established in accordance with the provisions of Table 5.1.

Table 5.1 Allowable rotation values, θ_{Rd}^{SLU}

Main structural component	Allowable rotation values θ_{Rd}^{SLU} (rad)	
	Ductility class	
	DCH	DCM
Beam	3.00 %	2.50 %
Coupling beam reinforced with orthogonal bars	1.50 %	1.50 %
Coupling beam reinforced with diagonal casings	4.00 %	4.00 %
Pillar	2.50 %	2.00 %
Wall	1.00 %	0.75 %

5.7 Composition and reinforcement

(5) The main structural components shall be constructed in such a way as to meet the composition and reinforcement requirements given in this paragraph.

(6) The geometry of the concrete section, the quantity of longitudinal reinforcement and its arrangement shall be determined in conjunction so that the failure of the bending sections, with or without axial force, does not occur by crushing the compressed concrete before the flow of the stretched longitudinal reinforcement. This condition applies to beams, pillars and walls, for buildings designed for the ductility class DCH, DCM, or DCL.

(7) The provisions on the quality of materials, the composition and reinforcement of the main structural components are established differently for:

(oooooooo) critical areas;

(pppppppp) current areas.

(539) The length and position of the critical areas of the main structural components shall be differentiated according to the type of element, the stress state and the ductility class, in accordance with the provisions of this paragraph. The part of the element that is located outside critical areas shall be considered a current area.

(540) By way of exception from (5), in the case of prestressed beams used under the conditions laid down in 5.2.2, (384), (yyyyyy), the provisions of paragraph 5.7 shall apply only in their critical areas.

5.7.1 Quality of materials

5.7.1.1 Concrete

(541) The characteristic value of the compressive strength of concrete in the main structural components meets the condition:

$$25\,N/mm^2 \leq f_{ck} \leq 50\,N/mm^2 \text{ for DCH} \quad (5.1)$$

$$20\,N/mm^2 \leq f_{ck} \leq 60\,N/mm^2 \text{ for DCM} \quad (5.2)$$

$$20\,N/mm^2 \leq f_{ck} \text{ for DCL} \quad (5.3)$$

(542) When choosing the quality of concrete, the specific durability requirements given in the specific technical regulations shall also be taken into account.

(543) Reinforced concrete elements shall be carried out in compliance with the provisions of technical regulations NE 012/1 and NE 012/2.

5.7.1.2 Steel

(544) Structural components shall be reinforced with steel bars of ductility class B or C, according to the classification given in SR EN 1992-1-1, as follows:

(qqqqqqqq) class C steels shall be used in critical areas of main structural components designed for ductility class DCH or DCM;

(rrrrrrrr) class B or C steel shall be used in current areas of main structural components designed for ductility class DCH or DCM and in main structural components designed for ductility class DCL.

(545) The characteristic value of the flow limit of steel from the main structural components shall meet the condition:

$$400\,N/mm^2 \leq f_{yk} \leq 500\,N/mm^2 \text{ for DCH and DCM} \quad (5.1)$$

$$400\,N/mm^2 \leq f_{yk} \leq 600\,N/mm^2 \text{ for DCL} \quad (5.2)$$

(546) For main structural components of buildings designed for ductility class DCH or DCM, the ratio of the effective characteristic value of the flow limit of steel, $f_{yk,act}$, and the specified characteristic value of the flow limit, f_{yk}, shall meet the condition:

$$1{,}00 \leq f_{yk,act}/f_{yk} \leq 1.15 \quad (5.3)$$

where the effective characteristic value of the flow limit of the steel is determined by tests on the casting batches.

(547) The main structural components are reinforced only with profiled steel bars.

(548) The provisions given in this paragraph do not refer to prestressing fittings.

5.7.2 Concrete section

5.7.2.1 Beams

(549) Height of cross-section of beams, h_w, meets the conditions:

$$l_{cl}/16 \leq h_w \leq l_{cl}/4 \text{ for DCH} \quad (5.1)$$

$$l_{cl}/16 \leq h_w \leq l_{cl}/3 \text{ for DCM} \quad (5.2)$$

where l_{cl} is the free opening of the beam.

(550) The width of the cross-section of the beam meets the conditions:

$$b_w \geq h_w/3 \text{ and } b_w \geq 250\,mm \text{ for DCH} \quad (5.3)$$

$$b_w \geq h_w/4 \text{ and } b_w \geq 200\,mm \text{ for DCM} \quad (5.4)$$

where h_w is the height of the cross-section of the beam.

(551) In the case of pillar-beam joints, the ratio of the distance between the longitudinal axis of the beam and the axis of the cross-section of the pillar, in the direction of the axis of the beam, to the width of the cross-section of the pillar measured perpendicularly to the axis of the beam shall be less than or equal to:

(ssssssss) 1/4 for DCH;

(tttttttt) 1/3 for DCM.

5.7.2.2 Pillars

(552) In the case of buildings designed for the ductility class DCH, DCM or DCL, the dimensions of the cross-sectional sides of the pillar shall be greater than or equal to 300 mm.

(553) The pillar shall be so constructed that in any horizontal direction the following condition is met:

$$\frac{l_{cl}}{h_c} \geq 2,5 \tag{5.1}$$

where:

l_{cl} the free height of the pillar;

h_c the height of the cross-section of the pillar, which is the dimension of the cross-section measured in the direction of calculation.

(554) In the case of buildings designed for the ductility class DCH, the pillars shall be constructed with a rectangular, circular or regular polygonal cross-section with the number of sides greater than or equal to 4. This provision shall not apply to buildings with a wall-type structural system, insulated walls or coupled walls.

(555) Ratio between the largest cross-sectional dimension of the pillar and the dimension measured in perpendicular direction less than or equal to:

(uuuuuuuu) 2.5 for DCH;

(vvvvvvvv) 4.0 for DCM and DCL

The largest cross-sectional dimension of the pillar is the maximum value a measured along any line passing through the centre of gravity of the section.

(556) By way of exception from (555), in the case of buildings with a wall-type structural system, insulated walls or coupled walls, designed for the ductility class DCH, DCM or DCL, the ratio between the largest cross-sectional dimension of the pillar and the dimension measured in a perpendicular direction at the centre of gravity of the section is equal to or less than 4.0.

(557) For each side of the pillar cross-section, the ratio between the total cross-sectional dimension measured perpendicular to that side and the length of the side shall be equal to or less than 4.0.

(558) Average normalized axial effort in any seismic design combination, v_d, fulfils the condition:

$$v_d \leq 0.45 \text{ for DCH} \tag{5.2}$$

$$\nu_d \leq 0.50 \text{ for DCM} \tag{5.3}$$

$$\nu_d \leq 0.55 \text{ for DCL} \tag{5.4}$$

(559) By way of exception from (558), in the case of dual buildings with predominant walls, the average normalised axial effort may be limited upwards to values 10 % higher than those indicated in (558) if the capable rotation of the pillar, determined using the behaviour pattern of the bent reinforced concrete elements, is greater than the requirement established in accordance with 4.3.1.2.3.

(560) In the case of the structural system of the frame type, the structural system of the dual type with predominant frames or the structural system with cantilevered pillars, if the cross-section of a pillar varies from one floor to another, the horizontal projection of the cross-section of the top-level pillar is internal, at the tangent limit, to the cross-section of the bottom-level pillar.

5.7.2.3 Beam-pillar joints

(561) The projections in the horizontal plane of the cross sections of the pillars leading in a joint are internal or, at the limit, tangent to the perimeter of the horizontal section of the joint, throughout its height.

5.7.2.4 Coupling walls and beams

(562) Coupling walls and beams shall comply with the provisions of technical regulation CR2-1-1.1.

5.7.2.5 Diaphragms

(563) The thickness of reinforced concrete slabs made monolithically that can perform the role of rigid diaphragm is greater than or equal to 100 mm.

(564) The floors can also be made as mixed elements, from prefabricated slabs with over-concreting, provided that the two layers of concrete are properly connected. In this case, the floor can act as a rigid horizontal diaphragm only if the thickness of the over-concreting is greater than or equal to 60 mm.

(565) Floors made of prefabricated plates, without over-concreting, can perform the role of rigid horizontal diaphragm only if they have a thickness greater than or equal to 100 mm and if the joints between the prefabricated plates are made in a wet system, by monolithization, and by the arrangement of the elements the visual inspection of the quality of monolithization can be made.

(566) If the floor just below the conventional embedding elevation is made of prefabricated plates with over-concreting, the over-concreting shall be carried out with a minimum thickness of 100 mm.

(567) The calculation of the efforts in diaphragms shall be made on the basis of the provisions given in the regulations specific to the different types of structures and their resistance capacity shall be established on the basis of the provisions of SR EN 1992-1-1.

5.7.2.6 Infrastructures and foundations

(568) Surface foundations shall be made in accordance with the provisions of technical regulation NP 112 and the additional provisions given in this paragraph.

(569) Piles foundations shall be carried out in accordance with the provisions of technical regulation NP 123 and the additional provisions given in this paragraph.

(570) The components of the foundations that ensure the support of the main structure on the ground are joined horizontally by a diaphragm or cross-ties.

(571) In the case of the piles foundation with general foundation frame, the latter shall be designed to be able to provide the role of horizontal diaphragm.

(572) The foundation beams and balancing beams that ensure the balance of the bending moment that develops at the bottom of a vertical structural component under seismic design action shall be constructed in compliance with the conditions given for beams in this chapter.

(573) It is recommended that the foundations of the pillars of type block and bearing box or sole be connected to each other by balancing beams that work together with the plate that borders at the bottom the level located immediately above the backing section on the ground. Balancing beams and/or foundation beams shall be arranged in such a way as to ensure the connection of the bottom of the pillars and/or walls in two horizontal orthogonal directions.

(574) By way of exception from (570), in the case of ground floor halls with articulated beams, insulated foundations may be made, without connection through a rigid diaphragm or cross-ties, if the deformations of the building caused by the relative displacements in the horizontal direction between the foundations, caused by the seismic design action, are not such as to prevent compliance with the basic requirements of the seismic design given in Chapter 2.

(575) In the case of prefabricated reinforced concrete pillars foundations made as insulated reinforced soles or glass foundations, when establishing the connecting forces on the base of the foundation, the balancing of the shearing moment or force in the pillar shall not be taken into account by means of the efforts that develop in the base floor as a result of its friction on the support layer.

(576) The components of the structure placed in a horizontal plane from the bottom of the pillars, situated below the conventional fixation-clamping section, shall be placed in such a way as to avoid the formation of short pillars, which do not comply with the condition given on 5.7.2.2, (553).

(577) In the case of buildings with underground levels, the transfer floor plate, located just below the conventional fixation-clamping section, shall be made with a thickness greater than or equal to 150 mm.

(578) In the case of main structures with pile foundations, piles constituting main structural components shall be made with a diameter greater than or equal to 400 mm.

(579) In the case of underground levels, the perimeter basement walls of reinforced concrete shall have a core thickness greater than or equal to 200 mm.

5.7.3 Reinforcement

5.7.3.1 Beams

(580) The reinforcement of the beams shall meet the conditions given at 5.7.3.1.1 and 5.7.3.1.2 for critical and current areas.

(581) Areas at the ends of beams with $l_{cr}=1{,}50\,h_w$ length, measured from the front of the pillars, as well as the areas of this length, located on either side of a section of the beam field, where flow in the seismic grouping may occur, are critical areas.

5.7.3.1.1 Longitudinal reinforcement

(582) Longitudinal reinforcement in the beams ensures the fulfilment of the condition of resistance to the bending moment and shear force, according to 5.3.2.

(583) The longitudinal reinforcement determined in accordance with the provisions of this paragraph shall be placed in the core of the beam.

(584) Longitudinal reinforcement coefficient of the stretched area, ρ, throughout the beam opening, fulfils the condition:

$$0{,}5¿ \qquad (5.1)$$

where:

f_{ctm} average value for tensile strength of the concrete;

f_{yk} characteristic value of flow limit of the steel;

$$\rho=\frac{A_s}{b_w d} \qquad (5.2)$$

A_s the extended longitudinal reinforcement area of the beam;

b_w width of the beam core;

d the useful height of the cross-section of the beam.

(585) Longitudinal stretched and compressed reinforcements are sized so that the height of the compressed area, x_u, at the final stage fulfils the condition.

$$x_u \le 0{,}25\,d \qquad (5.3)$$

In its calculation x_u, the contribution of reinforcements from the compressed area may also be taken into account.

(586) The beams shall be reinforced longitudinally continuously, throughout the opening, as follows:

(wwwwwwwww) at the top and bottom of the beams, at least two bars with a diameter of 14 mm or more shall be provided;

(xxxxxxxx) at least one quarter of the reinforcement in the stretched area of the beams in the maximum bending moment section shall be disposed over the entire length of the beam.

(587) Along the entire length of the beam, at least half of the reinforcement area of the stretched area shall be disposed in the compressed area.

5.7.3.1.2 Transverse reinforcement

(588) The transverse reinforcement in the beams shall ensure that the condition of resistance to shear force is met, according to 5.3.2.

(589) For transverse reinforcement, closed clamps are used, made of steel bars with a diameter of 8 mm or more. This applies to the ductility class DCH, DCM or DCL.

(590) Cross reinforcements in critical areas of the beams shall be provided with hooks bent at an angle of 135°, the straight length of which is equal to or greater than $10d_{bw}$, where d_{bw} is the diameter of the steel bar from which the reinforcement is made.

(591) The distance between the end of the beam and the first clamp is less than or equal to 50 mm.

(592) The distance between clamps in the critical area for buildings designed for ductility class DCH or DCM shall meet the condition:

$$s \leq min\{h_w/4 \,;150mm\,;6\,d_{bL}\} \text{ for DCH} \tag{5.1}$$

$$s \leq min\{h_w/4 \,;150mm\,;7\,d_{bL}\} \text{ for DCM} \tag{5.2}$$

where d_{bl} is the minimum diameter of the longitudinal reinforcements.

(593) At the end of a beam, there are inclined reinforcements in two directions, which make an angle of 45° with the longitudinal axis of the beam, in situations where the cumulative conditions are met:

$$\zeta = \frac{V_{Ed,min}}{V_{Ed,max}} \leq -0{,}50 \tag{5.3}$$

and

$$\frac{max\left(\left|V_{Ed,min}\right|,\left|V_{Ed,max}\right|\right)}{b_w d\, f_{ctd}} \geq 2+\zeta \tag{5.4}$$

where

$V_{Ed,min}$ the minimum design value of the shear force acting at the end of the beam;

$V_{Ed,max}$ the maximum design value of the shear force acting at the end of the beam;

Note: if the shear forces $V_{Ed\,min}$ and $V_{Ed\,max}$ have opposite signs, when calculating the ratio ζ, the minus sign shall be attributed to the lowest of the absolute values of the two forces, while the plus sign shall be attributed to the highest. The ratio ζ shall be situated between -1 and 1. ζ=-1 represents the worst case stress scenario, when the two shear forces have equal absolute values and opposite signs, ζ=1 represents the situation in which the shear force caused by the horizontal seismic action is insignificant.

(594) Inclined fittings arranged according to (593) shall meet the condition:

$$2\,A_{si} f_{yd} \sin\alpha \geq max\left(\left|V_{Ed,min}\right|,\left|V_{Ed,max}\right|\right) \tag{5.5}$$

where

A_{si} the area of the inclined reinforcement arranged in one of the two directions, i.e. the one that crosses the potential slip plane;

α the angle of inclination of the reinforcement A_{si};

f_{yd} the design value of the flow limit of the steel from which the inclined fittings are made.

(595) Inclined fittings set according to (593) shall be fitted in addition to the transverse fittings established in accordance with 5.3.2.

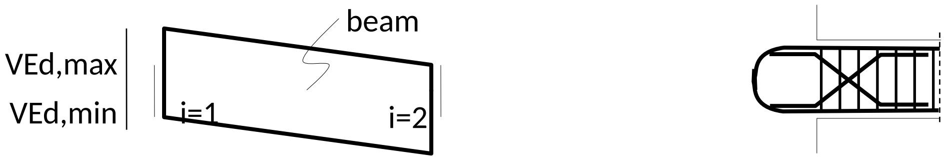


Figure 5.2 Informative representation on the significance of sizes $V_{Ed,max}$ and $V_{Ed,min}$ and the arrangement of the inclined reinforcement in the critical zone of the beams

(596) In critical areas of the beams, at the top and bottom faces, the fittings shall be arranged in such a way that the horizontal distance measured between the consecutive longitudinal beam bars at the corner of a clamp or pinned is less than 200 mm for DCH and 250 mm for DCM.

(597) In the current areas, a number of clamps at least equal to half that of the critical area shall be arranged.

(598) In the case of beams of structures designed for ductility class DCL, the maximum distance between clamps shall be 300 mm.

5.7.3.2 Pillars

(599) The reinforcement of the pillars shall meet the conditions given at 5.7.3.2.1 and 5.7.3.2.2 for critical and current areas.

(600) Areas at the ends of the pillars at each level are considered critical areas. The part of the element that is not considered a critical area shall be considered a current area.

(601) The length of each critical area, l_{cr}, shall meet the conditions:

(yyyyyyyy) for the critical areas of the pillars at the bottom of the pillars at each level:

$$l_{cr} \geq maxim\left(1{,}5\,h_c;\, l_{cl}/6;\, 600\,mm\right) \text{ for DCH} \tag{5.1}$$

$$l_{cr} \geq maxim\left(h_c;\, l_{cl}/6;\, 450\,mm\right) \text{ for DCM} \tag{5.2}$$

(zzzzzzzz) for the critical areas of the pillars at the top of the pillars at each level:

$$l_{cr} \geq max\{h_c;\, l_{cl}/6;\, 600\,mm\} \text{ for DCH} \tag{5.3}$$

$$l_{cr} \geq max\{h_c;\, l_{cl}/6;\, 450\,mm\} \text{ for DCM} \tag{5.4}$$

where

h_c the largest cross-sectional dimension of the pillar;

l_{cl} the free height of the pillar, at the level considered.

(602) If at a certain level $l_{cl}/h_c \leq 3$, the entire length of the pillar is considered as a critical area.

(603) In addition to the critical areas established in accordance with the provisions of (600), (601) and (602), in the case of pillars bordering non-structural components such as masonry walls, the entire length of the pillars shall be considered a critical area if:

(aaaaaaaaa) the masonry wall is provided with a gap that is adjacent to the pillar;

or

(bbbbbbbbb) masonry walls are adjacent only on one or two adjacent sides of the pillar.

(604) In the case of pillars in direct contact with rigid and resistant non-structural components, such as masonry parapets, the area of the pillar immediately above and below the upper limit of the parapet for a length equal to l_{cr} is considered a critical area.

(605) Within critical areas, clamps and clips shall be provided to ensure the necessary ductility and prevent local buckling of longitudinal bars. The transverse reinforcement shall be distributed in such a way as to achieve an efficient triaxial stress state. The minimum conditions to meet these requirements are those given at 5.7.3.2.1 and 5.7.3.2.2.

5.7.3.2.1 Longitudinal reinforcement

(606) Longitudinal reinforcement of pillars ensures the fulfilment of the condition of resistance to the bending moment, with or without axial force, and shear force, according to 5.3.2.

(607) Total longitudinal reinforcement coefficient, ❑$_t$, throughout the length of the pillar, fulfils the condition:

$$0{,}01 \le \rho_t \le 0{,}04 \text{ for ductility class DCH or DCM} \tag{5.1}$$

$$0{,}008 \le \rho_t \le 0{,}04 \text{ for ductility class DCL} \tag{5.2}$$

where

$$\rho_t = \frac{A_{st}}{A_c} \tag{5.3}$$

A_{st} total longitudinal reinforcement area of the cross-section of the pillar;

A_c the cross-sectional area of the pillar.

(608) At least one longitudinal intermediate bar shall be provided between the longitudinal fittings at the corners of the cross-section on each side.

(609) In the case of cross-reinforced pillars with a hooped reinforcement or circular clamps, there shall be at least six longitudinal bars on the perimeter.

(610) In the case of frame-type structural system, dual-type structural system with predominant frames or cantilevered structural system designed for ductility class DCH, the diameter of the reinforcement bars shall be set such that the overlap length of the longitudinal reinforcements is less than $h_s/2$.

5.7.3.2.2 Transverse reinforcement

(611) The transverse reinforcement of the pillars shall ensure that the condition of resistance to shear force is met, according to 5.3.2.

(612) For transverse reinforcement, closed clamps shall be used, made of steel bars with a diameter of 8 mm or more. This applies to pillars of structures designed for the ductility class DCH, DCM or DCL.

(613) Transverse reinforcements in critical areas of the pillars shall be provided with hooks bent at an angle of 135°, whose straight length is equal to or greater than $10d_{bw}$, where d_{bw} is the diameter of the steel bar from which the reinforcement is made.

(614) In the critical areas at the base of the pillars, immediately above the conventional fixation-clamping section, in both main horizontal directions, the transverse reinforcement shall meet the conditions:

$$\rho_w \geq 0{,}005 \text{ for DCH} \tag{5.1}$$

$$\rho_w \geq 0{,}0035 \text{ for DCM} \tag{5.2}$$

$$\omega_{wd} \geq 0{,}12 \text{ for DCH} \tag{5.3}$$

$$\omega_{wd} \geq 0{,}08 \text{ for DCM} \tag{5.4}$$

where:

ρ_w transverse reinforcement coefficient;

$$\rho_w = \frac{A_{sw}}{b_w s} \tag{5.5}$$

A_{sw} transverse reinforcement area in the direction considered;

b_w the dimension of the cross-section of the pillar perpendicular to the direction in question;

s distance between clamps;

ω_{wd} mechanical transverse reinforcement coefficient:

$$\omega_{wd} = \frac{volu\,me\;of\;the\;confinement\;clamps}{volum\,e\;of\;the\;confined\;concrete\;core} \frac{f_{yd}}{f_{cd}} \tag{5.6}$$

f_{yd} design value of steel flow limit;

f_{cd} design value of concrete compressive strength.

(615) In all critical areas except those referred to in (611), in both main horizontal directions, the transverse reinforcement shall meet the conditions:

$$\rho_w \geq 0{,}0035 \text{ for DCH} \tag{5.7}$$

$$\rho_w \geq 0{,}0025 \text{ for DCM} \tag{5.8}$$

$$\omega_{wd} \geq 0{,}08 \text{ for DCH} \tag{5.9}$$

$$\omega_{wd} \geq 0{,}06 \text{ for DCM} \tag{5.10}$$

(616) The distance between the clamps in the critical areas of the pillars shall fulfil the conditions:

$$s \leq min¿ \text{for DCH} \tag{5.11}$$

$$s \leq min¿ \text{for DCM} \tag{5.12}$$

where

b_0 the minimum size of the useful section, located inside the perimeter clamp;

d_{bL} minimum diameter of the longitudinal bars.

(617) The distance between the clamps in the critical zone immediately above the conventional fixation-clamping section shall meet the condition:

$$s \leq min¿ \tag{5.13}$$

(618) In the case of pillars of structures designed for the ductility class DCL, the distance between the callipers shall meet the condition:

$$s \leq min¿ \tag{5.14}$$

(619) The transverse reinforcement shall be arranged in such a way that the distance measured along the perimeter of the cross section between the longitudinal bars of the consecutive pillar at the corner of a clamp or pinned is less than 150 mm for DCH and 200 mm for DCM.

(620) If, in order to meet the conditions of (611), (614), (615), (616) and (617), several types of transverse reinforcements are used, such as clamps or cramps with different geometric configurations, conditions from (616) and (617) shall be applied separately for each type of transverse reinforcement.

(621) On the first two levels of buildings with more than 5 levels and on the first level in the case of lower buildings, above the critical area immediately above the conventional fixation-clamping section, large frequent clamps shall be provided for a distance equal to half of its length.

(622) In pillars of structures designed for the ductility class DCL, the transverse reinforcement coefficient shall be equal to or greater than 0.003 in each direction over a length equal to the maximum cross-sectional dimension of the pillar, h_c above the conventional fixation-clamping section. At the other levels, the transverse reinforcement coefficient at the bottom of the pillars shall be greater than or equal to 0.0025.

(623) A number of transverse reinforcement at least equal to half that of the critical area shall be provided in the current area.

(624) In the case of pillars which are in contact with non-structural components such as masonry walls which are less than the free height of the pillar, if the length over which the pillar is not in contact with the masonry wall is less than $1.5h_c$, the shear force caused by seismic action acting parallel to the plane of the wall is taken over by inclined reinforcements. They are disposed in addition to the transverse fittings disposed according to the provisions (612)..(623).

5.7.3.3 Beam-pillar joints

(625) Vertical and horizontal reinforcement in the beam-pillar joints of the main seismic components shall comply with the provisions of this paragraph.

(626) The vertical reinforcement in the beam-pillar joints shall be at least equal to the longitudinal reinforcement arranged in the pillars, in the critical areas adjacent to the joint.

(627) The horizontal reinforcement in joints shall be arranged in the form of closed clamps or cramps placed in a horizontal plane.

(628) The horizontal reinforcement in the beam-pillar joints shall be greater than or equal to the transverse reinforcement arranged in the adjacent critical areas of the pillars entering the joint.

(629) Total area of horizontal reinforcement in the joint, A_{sh}, shall meet the conditions:

(cccccccc) at all joints except the end joints:

$$A_{sh} \geq 0{,}8 (A_{s1} + A_{s2})(1 - 0{,}8 \nu_d) \tag{5.1}$$

where

A_{s1} and A_{s2} the areas of the stretched reinforcements at the top and bottom, respectively, of the beams entering the joint in the considered direction of the seismic action, determined according to the direction of the seismic action;

ν_d the design value of the average normalised axial stress in the lower pillar of the joint;

(dddddddd) at the end joints:

$$A_{sh} \geq 0{,}8 A_{s2} (1 - 0{,}8 \nu_d) \tag{5.2}$$

where

A_{s2} the area of the stretched reinforcements of the beam entering the joint in the considered direction of the seismic action, determined according to the direction of the seismic action.

(630) If steels of different quality are used for horizontal and vertical reinforcement of the joint, the quantity of reinforcement determined according to (5.1) or (5.2) shall be multiplied by the ratio f_{yd} / f_{ywd} where f_{yd} is the flow limit of the steel from which the longitudinal beam reinforcements are made and f_{ywd} is the flow limit of the steel from which the horizontal reinforcements in the joint are made.

(631) Total area of horizontal reinforcement in the joint, A_{sh}, resulting from the application of the relation (5.1) or (5.2) shall be evenly distributed over the height of the joint.

(632) In the case of external joints, the reinforcement, A_{sh}, resulting from the application of the relation (5.1) or (5.2), shall be increased by 20 %.

(633) In the case of external joints, the longitudinal reinforcements in the beam are turned inside the joint clamps, in the vicinity of their opposite side to the end section of the beam.

(634) Longitudinal reinforcements of beams that stop in notes by bending have the beak facing the longitudinal axis of the beam. The longitudinal reinforcements of the pillars that stop in joints by bending have the beak oriented towards the longitudinal axis of the pillar.

5.7.3.4 Coupling walls and beams

(635) Coupling walls and beams shall comply with the provisions of technical regulation CR2-1-1.1.

5.7.3.5 Slab floors

(636) This paragraph contains additional provisions for the construction of floors made of slabs leaning directly on vertical structural components.

(637) The design of the floors resting directly on the vertical structural components shall be made according to the provisions of SR EN 1992-1-1. The provisions of this paragraph contain minimum composition conditions that apply in conjunction with the provisions of SR EN 1992-1-1.

(638) The slab shall made with thickness greater than or equal to 150 mm.

(639) For all longitudinal reinforcements at the bottom of the slab located inside the supporting strips, continuity shall be ensured along the entire length of the slab. The jointing of the reinforcements shall be designed considering a tensile effort equal to the design value of the flow limit of the steel. For each main direction, the width of the supporting strip shall be calculated as the sum of the cross-sectional dimension of the vertical structural component and 25 % of the free apertures of the slab, to the left and right of the pillar, perpendicular to the direction of the strip.

(640) At the joints of the vertical structural component-slab there are longitudinal integrity reinforcements in both main horizontal directions at the bottom of the slab. In each direction, the integrity reinforcement shall be made from at least two bars of 14 mm diameter. Integrity reinforcements uninterruptedly cross the joint, inside the reinforcement housing of the vertical structural component. Through the detailing mode, the integrity reinforcement continuity shall be ensured along the entire length of the slab considering a tensile effort equal to the design value of the flow limit of steel. The reinforcement at the bottom of the slab may be considered as integrity reinforcement if it meets the requirements for integrity reinforcement given in this paragraph.

(641) Longitudinal integrity reinforcements shall be arranged in such a way that the area of all integrity reinforcements intersecting the vertical perimeter surface of the joint meets the condition:

$$\sum A_s \geq \frac{V_{Ed}}{2{,}00\sqrt{f_{yd} f_{cd}}} \tag{5.1}$$

where:

$\sum A_s$ the area of all integrity reinforcements intersecting the vertical perimeter surface of the joint;

V_{Ed} design value of the shear force in the slab that equilibrates itself through the slab-pillar joint;

f_{yd} design value of the flow limit of reinforcements;

f_{cd} design value of the compressive strength of the concrete.

(642) In case of direct support of the slab on the pillars, inside the critical penetration perimeter u_1, established according to the provisions of SR EN 1992-1-1, the slab shall be made continuous, without gaps.

(643) By way of exception from (642), a single void may be made within, tangential to or intersecting the critical penetration if the conditions are met:

$$l_g \leq 0{,}25\,d$$
$$l_g \leq 0{,}10\,c \qquad (5.2)$$

where

l_g the maximum dimension of the void in the horizontal plane;

d the minimum useful height of the cross-section of the slab on the perimeter of the void;

c the minimum cross-sectional dimension of the vertical structural component constituting the support for the slab.

(644) The edges of the slabs shall be reinforced according to the detail of Figure 5.2.

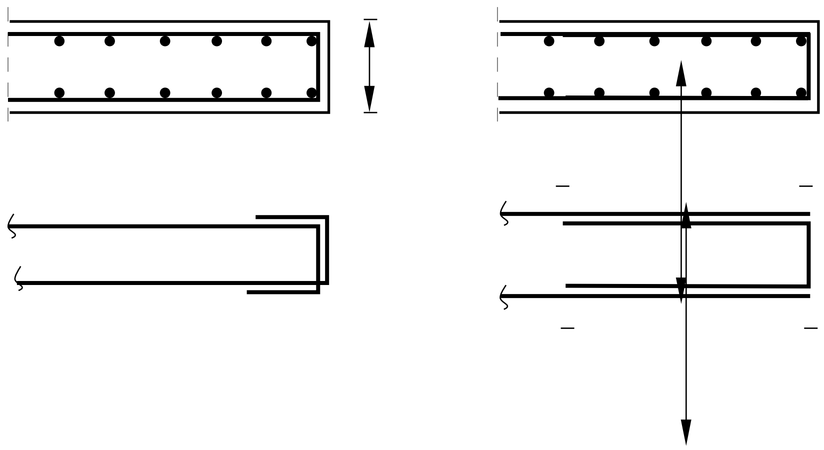


Figure 5.2 Reinforcement detail of the edges of the slab

(645) Transverse reinforcement of the slab in the area of resting on vertical structural components shall be carried out only with vertical reinforcements.

(646) Slab areas located in the vicinity of pillars or end areas of walls, up to a distance equal to 4,00 d measured in relation to their perimeter, where d is the useful height of the cross-section of the slab, shall be cross-reinforced. The transverse reinforcement shall be set in such a way that the transverse reinforcement coefficient, ρ_w, fulfils the condition:

$$\rho_w \geq 0{,}08\frac{\sqrt{f_{ck}}}{f_{yk}} \qquad (5.1)$$

where:

f_{yk} the characteristic value of the flow limit of the steel from which the transverse reinforcements are made;

f_{ck} characteristic value of concrete compressive strength.

Note: Reinforcement established in accordance with the relation (5.1) constitute a minimum quantity of transverse reinforcement to be supplemented, if necessary, so that the penetration resistance condition according to SR EN 1992-1-1 is met.

5.7.3.6 Reinforcements anchoring and jointing

(647) When designing reinforcements anchorages and jointing, the provisions of SR EN 1992-1-1 shall apply together with the additional provisions given in this paragraph.

(648) The lengths of anchoring or jointing shall be determined according to the value of the effort that develops in the bar when forming the overall plastic mechanism of the structure.

(649) The anchoring or jointing lengths shall be determined considering that the design value of the tensile effort developing in the longitudinal reinforcements over the entire length of the critical areas is equal to $1{,}20 f_{yd}$.

(650) Reinforcements shall be anchored outside critical areas.

(651) Reinforcement jointing is recommended to be carried out outside critical areas.

(652) Overlapping the longitudinal reinforcements of the beams shall carried out outside the joints, critical areas of the beam and at a distance of more than $1.5h_w$ from the end sections of the beam.

(653) In the case of the structural system of the frame type, the structural system of the dual type with predominant frames or the structural system with cantilevered pillars designed for the ductility class DCH, the overlapping of the longitudinal bars of the pillars shall be carried out in their middle area, from each level.

(654) The anchoring length determined in accordance with 5.7.3.6 (648) shall be limited according to the relationship:

$$l_{bd} \geq 40\varphi \tag{5.1}$$

where φ is the diameter of the anchoring bar.

(655) The jointing length determined in accordance with 5.7.3.6, (2), shall be lower limited to the values indicated in Table 5.6, where φ is the diameter of the jointing bar. For intermediate values of the ratio between the area of reinforcements installed in section and the area of all reinforcements, linear interpolation shall be performed.

Table 5.1 Minimum values of the overlapping jointing length

Ratio between the area of jointed reinforcements in section and the area of all the reinforcements	<0.25	33 %	50 %	>50 %
Minimum length of jointing by overlapping	40φ	45φ	55φ	60φ

Note: In the case of bent elements, the area of all reinforcements means the area of the stretched or compressed reinforcements of a section, where appropriate, including the bar that is being jointed.

(656) If in a seismic design combination, the design value of the axial force in a pillar is tensile, the length of the anchorage or jointing of the longitudinal

reinforcements determined in accordance with 5.7.3.6, (648), shall be increased by 50 %.

(657) The shape of a longitudinal reinforcement bar anchored in a beam-pillar joint shall be determined considering the anchorage length measured from distance $5d_{bL}$ from the face of the anchoring element, inside it, where d_{bL} is the diameter of the anchoring bar.

(658) Longitudinal beam bars that stop in the beam-pillar joints are bent inside the pillar housing, on the opposite side to the section of the beam from which the anchoring is made. The straight part that develops at the end of the bar, parallel to the direction of the longitudinal reinforcements in the pillars, shall be made with a length greater than or equal to $12d_{bl}$, where d_{bL} is the diameter of the anchoring bar.

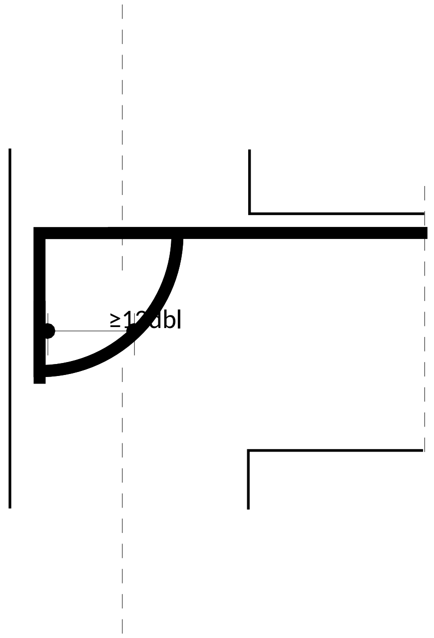


Figure 5.2 Minimum beak length for longitudinal reinforcements in beams

(659) The anchoring length of the reinforcements that stop in the beam-pillars joints shall be ensured by a maximum of bending them inside the joint.

(660) Longitudinal reinforcements in the critical area of the pillars shall not be hardened by overlapping in the beam-pillars joints.

(661) The diameter of the longitudinal reinforcements of the beams passing through the beam-pillar joints shall be determined in such a way that the conditions are met:

(eeeeeeeee) in the case of end joints:

$$d_{bL} \leq 10\left(1+0,8\,v_d\right)\frac{f_{ctm}}{f_{yd}}h_c \tag{5.1}$$

(fffffffff) in other cases:

$$d_{bL} \leq 10\frac{1+0,8\,v_d}{1+0,75\,A_{s2}/A_{s1}}\frac{f_{ctm}}{f_{yd}}h_c \tag{5.2}$$

where

h_c the size of the pillar side parallel to the bars;

A_{s2}, A_{s1} area of compressed reinforcement and respectively, stretched from beams passing through the joint;

f_{ctm} average value for concrete tensile strength

f_{ym} average value of the steel flow limit

v_d the design value of the average axial stress normalised in pillars in the seismic design situation.

(662) In critical areas of pillars where significant plastic deformations are expected, according to the configuration of the optimal plastic mechanism, no jointing by overlapping shall be carried out. Jointing by overlapping is recommended be avoided in all other critical zones.

(663) The reinforcements of the pillars, beams and walls shall not be welded along the length of the critical areas of these elements.

(664) Welding jointing shall be projected at an average unit effort value in the steel bar equal to or greater than $1{,}25 f_{yd}$.

(665) Only products with technical approval explicitly specifying the suitability for use under seismic stress conditions, with dynamic actions applied cyclically, compatible with the ductility class used in the design of the building, shall be used for end-to-end bracing of reinforcements with coupling devices. For buildings designed for the ductility class DCH or DCM, only coupling devices may be used for which the technical approval specifies that the yield of the bars is guaranteed until their deformation capability at alternating cyclic stresses is exhausted, without failure of the joint. Failure of the joint is not allowed.

(666) In multi-storey buildings, where at the reinforcement of the pillars and edge elements of the walls the longitudinal reinforcement bars are joint by overlapping in the critical area at the bottom of a level, the length of jointing l_0 shall be determined with the relation:

$$l_0 = 2\sqrt{A_s^{'}/A_s}\, l_{bd} \le 1{,}5\, l_{bd} \tag{5.3}$$

where

$A_s^{'}/A_s$ the ratio between the area of the longitudinal reinforcements that join in section and the total area of the longitudinal reinforcement;

l_{bd} the basic anchorage length calculated in accordance with SR EN 1992-1-1.

(667) The distance between the transverse reinforcements of beams, pillars or bulbs of concrete walls in the overlapping areas of longitudinal reinforcements shall meet the condition:

$$s \le minim\left(\frac{h}{4}, 100mm\right) \tag{5.4}$$

where

h the height of their cross-section.

(668) Area A_{sw1} of the section of a branch of the transverse reinforcement in the joining area shall meet the condition:

$$A_{sw1} \geq s \frac{d_{bL}}{50} \frac{f_{yd}}{f_{ywd}} \tag{5.5}$$

where

f_{yd} and f_{ywd} are the design values of the flow limit of the longitudinal and transverse reinforcements;

d_{bL} diameter of the longitudinal jointing reinforcement.

(669) Additional provisions on anchorages and jointing of reinforcements of beams and pillars to structures in reinforced concrete frames are given in technical regulation NP 007.

(670) Additional provisions regarding anchorages and jointing of reinforcements to walls and coupling beams for reinforced concrete walls constructions are given in technical regulation CR 2-1-1.1.

5.7.3.7 Infrastructures and foundations

5.7.3.7.1 Foundations

(671) Continuous reinforcements shall be provided at the top and bottom of the balancing, foundation beams and connecting soles between the foundations along their entire length.

(672) The areas of intersection between the vertical structural components and the foundation or balancing beams shall be made as beam-pillars joints.

(673) The foundation plate shall be reinforced with at least one net of steel reinforcements at the top and at the bottom. The coefficient of reinforcement for each of these two nets, in each direction, shall be equal to or greater than 0.002.

(674) For buildings designed for the ductility class DCH or DCM, the transverse reinforcement of piles in their critical areas shall meet the requirements for piles of the ductility class DCM.

(675) The length of the critical area of the piles shall meet the condition:

$$l_{cr} \geq 2d \tag{5.1}$$

where:

d diameter of the pile.

At least one critical area at the top of the pile shall be considered for the design.

If the pile crosses the interface between two terrain layers with very different shear rigidity, for which the shear deflection module ratio is equal to or greater than 6.00, the areas of l_{cr} above and below the interface shall be considered as critical areas.

5.7.3.7.2 Basement walls

(676) The perimeter and interior walls of the underground levels shall be reinforced in a horizontal and vertical direction. The way of realization and arrangement ensures the continuity of horizontal and vertical reinforcements over the entire surface of the walls.

(677) The total horizontal reinforcement distributed in the heart of the walls of the infrastructure corresponds to a minimum reinforcement percentage greater than or equal to 0.30 %.

(678) The distance between horizontal bars distributed in the heart of the walls is less than 250 mm.

(679) The total vertical reinforcement distributed in the heart of the walls of the infrastructure corresponds to a minimum reinforcement percentage greater than or equal to 0.30 %.

(680) The distance between vertical bars distributed in the core of the walls is less than or equal to 300 mm.

5.7.3.7.3 Infrastructure floors

(681) In multi-storey buildings having one or more underground levels, the slabs over the basements shall be longitudinally reinforced at both sides with continuous nets.

(682) The amount of reinforcement in each net, in each of the two horizontal directions, corresponds to a reinforcement rate greater than 0.25 % and is greater than 300 mm^2/m.

5.7.3.8 Other provisions

(8) In determining the distance between the transverse reinforcements according to the minimum diameter of the longitudinal reinforcements, no account shall be taken of the diameter of the surface reinforcement arranged to prevent the separation of the coating according to the provisions of SR EN 1992-1-1.

6 Steel structures

6.1 General information

6.1.1 Subject matter and scope

(683) This chapter contains provisions for the seismic design of buildings with the main structure made of steel.

(684) When designing the main steel structures, the provisions of the Romanian standards of the SR EN 1993-1 series shall also be used, as indicated in this chapter.

(685) Specific technical regulations and standards of the SR EN 1993-1 series shall be used for the design of buildings with steel structure for actions other than seismic ones.

6.1.2 Definitions

(686) The terms specific to this chapter are:

Dissipative bar: structural component of the eccentric braced frame, bounded at least at one end by a diagonal of the bracing, conformed to have a high plastic deformation capacity under the action of the shear force and/or bending moment.

Frame: structural sub-assembly consisting of pillars and beams;

Frame with moveable joints: frame which under the action of external horizontal loads is deformed by horizontal displacements and rotations of joints; unbraced frames are frames with shiftable joints.

Fixed joints frame: frame that under the action of external forces deforms only by rotating the joints. Frames which, under the action of horizontal external forces, have limited horizontal displacements are accepted in this category. Structures with braced frames or dual frames may be considered to have fixed joints if reinforced concrete structural walls, shearing walls or vertical braced systems reduce horizontal movement by at least 80 %.

Bracing with restrained buckling: bracing components of which the compressive strength is at least equal to that of the tensile strength.

Beam: steel structural component, preponderantly under stress at the moment bending and shear force.

Shear panels: panels consisting of a core made of sheet metal panels, whether or not stiffened, attached to the whole contour by screws or welded in relief by vertical and horizontal bordering elements.

Structural system of unbraced frame type: structural system in which horizontal forces are taken over by bending structural components;

Structural system of centric braced frame type: structural system consisting of frames with diagonals arranged in such a way that the axes of the structural components intersect at the same point. Horizontal forces in the plane of the frame are mainly taken over by diagonal axial efforts. Bracing can be designed as:

- bracings with active stretched diagonals, where the horizontal forces are taken over only by the stretched diagonals, the compressed diagonals being negligible;

- bracings with diagonals in V, at which the horizontal forces are taken over by both stretched and compressed diagonals, the intersection point of these diagonals is located on the beam, which must be continuous.

Eccentric braced frame structural system: frame-type structural system with diagonals arranged in such a way that their axes do not intersect at least at one end at the same point on the beam. Horizontal forces are mainly taken over by shearing and/or bending the dissipative bars.

Inverted pendulum structural system: structural system where more than 50 % by mass is concentrated in the upper third of the structure or where energy dissipation occurs mainly at the base of a single structural component. Unbraced ground-floor structures, with the upper extremities of the pillars connected by a rigid diaphragm behaviour system, do not fall into this category if the axial forces in the pillars meet the equation 6.2.5.1(6.1).

Dual type structural system: structural system consisting of non-braced frames and braced frames, connected to each other by rigid horizontal diaphragms, at which the non-braced frames take over at least 25 % of the horizontal force.

Frame structural system with obstructed buckling bracing: structural system consisting of braced frames with bars with obstructed buckling. The horizontal forces are balanced by stretching or compressing the bars with hindered buckling.

Pillar: vertical or slightly inclined structural component supporting gravitational loads predominantly by axial compression.

Non-dissipative structure: structure that responds only elastically to seismic design action, corresponding to the ultimate limit state.

Potentially plastic area: specially conformed area of a structural element in which deformations can develop in the inelastic domain.

Dissipative area: potential plastic area where cyclic plastic deformations dissipate the energy induced by the earthquake.

6.2 Design principles

6.2.1 Ductility classes

(687) Main seismic structures shall be seismically designed for:

(ggggggggg) high or medium dissipative behaviour;

(hhhhhhhhh) poorly dissipative behaviour;

(iiiiiiiii) non-dissipative behaviour.

(688) Structures with high or medium dissipative behaviour are designed to respond elasto-plastic to design seismic action, plastic deformations being directed to dissipative areas. In this approach, buildings shall be designed for the ductility class DCM or DCH, subject to the provisions specific to those ductility classes given in this chapter.

(689) Structures with poorly dissipative behaviour are designed to respond elastically to the seismic action of the design, without producing significant incursions of steel

into the plastic field. These structures shall be designed for the ductility class DCL, subject to the provisions specific to this ductility class given in this chapter.

(690) Steel-structure buildings shall be designed for seismic actions for the ductility class DCH, DCM or DCL.

(691) Buildings located in areas of moderate or high seismicity shall be designed for ductility class DCH or DCM.

(692) By way of exception from (691), in areas of moderate or high seismicity, where it is not possible to meet the design criteria specific to the ductility class DCH or DCM, buildings may be designed for the ductility class DCL if their overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q=1$).

(693) Structures not falling within the types indicated 6.2.4, (376), shall be designed for seismic actions for the ductility class DCL so that their overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than or equal to the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q=1$).

(694) When choosing the design concept for the calculation of the construction at seismic actions, the provisions of chapter 4.1.2 shall also be complied with.

(695) Frame structures where the bracing intersects on the free height of the pillars shall be designed for the ductility class DCL.

(696) The main structures shall be seismically designed for the ductility class DCL on the basis of the provisions of Chapters 1, 2, 3 and 4 of this technical regulation and the provisions of SR EN 1993-1-1, together with the provisions explicitly indicated for this class of ductility in this chapter.

(697) When checking structural components, partial safety coefficients shall be used for seismic design of steel structural components and their joints γ_M laid down in SR EN 1993-1-1, SR EN 1993-1-3, SR EN 1993-1-5 , SR EN 1993-1-8.

6.2.2 Section class

(698) The main structural components shall meet the specific requirements of each ductility class in terms of sections class and plastic deformation capacity.

(699) For structures designed for the ductility class DCH, the dissipative zones of the main structural components shall be carried out with class 1 sections.

(700) For structures designed for the DCM ductility class, the dissipation zones of the main structural components shall be made with class 1 or 2 sections.

(701) For structures designed for ductility class DCL, the main structural components shall be made with sections of class 1, 2, 3 or 4.

(702) For structures designed for ductility class DCL made of structural components with sections of class 1, 2 or 3, the maximum value of the factor q shall be limited to the upper value of 1.50.

(703) For structures designed for ductility class DCL made of elements with sections of class 4, the maximum value of the behaviour factor q shall be limited to the upper

value of 1.00. Different values of the behaviour factor can be established on the basis of the provisions of specific technical regulations, with their upper limit at 1.50.

(704) The main structural components shall meet the requirements of the section class given in Table 6.1, depending on the ductility class of the structure and the maximum value of the behaviour factor.

Table 6.1 Section class of dissipative elements

Ductility class of the structure	Maximum value of the behaviour factor q	Section class	Remarks
DCH	According to 6.2.6	class 1	Structure with high or medium dissipative behaviour
DCM	According to 6.2.6	class 1 or 2	
DCL	$1,0 \leq q \leq 1,5$	class 1, 2 or 3	Structure with poorly dissipative or non-dissipative behaviour
	$q = 1,0$	class 1, 2, 3 or 4	

(705) Structural components made of class 4 sections shall be designed in accordance with SR EN 1993-1-1, SR EN 1993-1-3 and SR EN 1993-1-5.

6.2.3 Conditions on materials

(706) Steel in the main structural components shall meet the requirements SR EN 1993 and SR EN 10025 and the additional provisions given in this paragraph specific to seismic design.

(707) Steel from structural components that deform plastic shall meet the following conditions:

(jjjjjjjj) the ratio of tensile strength, f_u, and the flow limit, f_y, greater than or equal to 1.20;

(kkkkkkkkk) the specific deformation at breakage is greater than 17 %;

(llllllll) it has a distinct plateau of yield; the steel grades provided for in Table 6.1. shall be used

(708) Structural components or parts thereof made of sheet metal with a thickness of more than 16 mm, required by unit tensile efforts perpendicular to the plane of the sheet, shall be ultrasonically controlled throughout the area so requested, in accordance with SRN EN 1993-1-10 and SR EN 10164.

Note: Such components are, for example, the end plates of the beams or flanges.

(709) For parts forming rigid beam-pillars joints required to be extended perpendicular to their plane, tensile tests shall be carried out in a direction perpendicular to their surface in accordance with SR EN ISO 6892-1. Construction steels with improved deformation characteristics in the direction perpendicular to the surface of the product in accordance with SR EN 10164 or tensile tests in the direction perpendicular to the surface of the product in accordance with SR EN ISO 6892-1 shall be used for parts forming rigid beam-pillar tension joints perpendicular to their plane in order to avoid failure by lamellar detachment.

(710) When making dissipative structural components or dissipative areas, steel with an effective flow limit of less than or equal to $\omega_{rm} f_y$ shall be used, which shall be explicitly indicated in the draft.

(711) Screw joints of main structures shall be designed with screws of mechanical characteristics groups 8.8 and 10.9.

(712) The bolts and anchor rods of the pillars in the foundations are made of steel of quality groups 4.6, 5.6 or steel of the grades S235, S275, S355, S420 or S460. Screws or rods with physico-mechanical characteristics of quality group 8.8 can also be used if they are made of low-alloy steel with normalizing heat treatment.

(713) The toughness of steel and welds shall meet the requirements for seismic action at quasi-permanent operating temperature according to the provisions of SR EN 1993-1-10, for a design value of unit effort $\sigma_{Ed}=0{,}75 f_{y(t)}$.

(714) The maximum wall thickness of stretched, bent or bent and stretched sections of structural elements shall be determined in accordance with SR EN 1993-1-10, depending on the steel grade, the value of the breaking energy KV (in J) and the minimum reference temperature, T_{Ed}.

(715) The breaking energy KV of steel and welded joints shall be equal to or greater than 27J at the operating temperature considered in the load grouping including seismic action. These values shall be explicitly indicated in the draft.

(716) For the main structural components, the minimum quality class of steel shall be correlated with the execution class of the elements set out in SR EN 1090-2, as follows:

(mmmmmmmmm) for execution class EXC2, steel of grade JR or above shall be used;

(nnnnnnnnn) for execution class EXC3, steel of grade J0 or above shall be used;

(ooooooooo) for execution class EXC4, steel of grade J2 and above shall be used.

(717) In the checks on the hierarchy of resistance referred to in the paragraphs 6.6-6.11, account shall be taken of the possibility that the effective flow limit of steel may be higher than the nominal flow limit, f_y, by using the yield overstrength factor of steel ω_{rm}, determined in accordance with the provisions of Table 6.2.

Note: The value of the flow limit of steel may vary from the nominal flow limit.

Table 6.1 Yield overstrength factor of steel, ω_{rm}

Steel grade	Overstrength factor, ω_{rm}
S235	1.45
S275	1.35
S355, S420	1.25
S460	1.20

6.2.4 Types of structures

(718) Steel structure buildings designed for seismic actions have the main structural system of the type:

(ppppppppp) unbraced frame-type structural system;

(qqqqqqqqq) centric braced frame type structural system;

(rrrrrrrr) structural system of eccentric braced frame type;

(sssssssss) inverted pendulum structural system;

(tttttttt) dual-type structural system;

(uuuuuuuuu) structural frame-type system with obstructed buckling bracing;

(vvvvvvvvv) structural frame-type system with steel shear panels.

(719) Steel-structured buildings may have different structural systems on the two main horizontal orthogonal directions. In the design of these structures, design rules specific to each structural system shall be used, in the appropriate direction.

(720) By way of exception from (719), for steel structures with high torsion flexibility the same type of structural system shall be used on the two horizontal orthogonal directions.

(721) The main structural systems of buildings fall within the types indicated in 6.2.4, (718), only if the following conditions are met:

(wwwwwwwww) the structures are constructed in accordance with the definition of the type of structural system given in 6.1.2, (686);

(xxxxxxxxx) all the main structural components, their joints and foundation fastenings fulfil the specific provisions of this technical regulation.

(722) Main structures that do not fulfil the provision (721) may be designed seismically in accordance with the provision 6.2.1, (693).

(723) All main structural components, regardless of the type of structural system, shall be designed for the same class of ductility.

6.2.5 Plastic mechanisms

(724) Steel structures designed for the ductility class DCH or DCM shall be designed to form the following types of optimal plastic mechanisms, at the incidence of design seismic action, corresponding to the ultimate limit state:

(yyyyyyyyy) unbraced frame structures: the optimal plastic mechanism is formed by the development of plastic deformations at the ends of the beams, in the vicinity of the beam-pillar joint, as a result of bending; for the formation of the optimal plastic mechanism, plastic deformations from bending can also be formed:

- at the bottom of the pillars, immediately above the conventional fixation-clamping section and at the top of the pillars at the top level of the multi-storey buildings;
- at the top and bottom of the pillars of single-storey buildings, if the condition is met:

$$N_{Ed}/N_{pl,Rd} \le 0{,}30 \qquad (6.1)$$

where

N_{Ed} the design value of the axial force in the pillar in the seismic grouping;

$N_{pl,Rd}$ the design value of the plastic resistance capacity of the section to axial force.

(zzzzzzzzz) centric braced frame structures: optimal plastic mechanism is formed by the development of plastic deformations in diagonal bracings that are required when stretching or, where appropriate, when stretched and compressed;

(aaaaaaaaaa) eccentric braced frame structures: the optimal plastic mechanism is formed by the development of plastic deformations in dissipative bars, created by the eccentric grip of diagonals on the beam, as a result of bending or cyclic shear force.

(bbbbbbbbbb) inverted pendulum structures: the optimal plastic mechanism is formed by the development of plastic deformations predominantly at the bottom of a single vertical structural component, immediately above the conventional fixation-clamping section;

(cccccccccc) dual structures consisting of non-braced frames and braced frames, the plastic mechanism is formed separately, corresponding to each type of frame;

(dddddddddd) frame structures with obstructed buckling bracing: the optimal plastic mechanism is formed by the development of plastic deformations in bracings with hindered buckling, by stretching or compression efforts.

(725) unbraced frames with steel shear panels: the plastic mechanism is formed mainly by post-elastic deformation of diagonal tension fields formed in the core of the shear panels, and then by the formation of plastic joints at the ends of the plating beams of the shear panels; plastic joints are allowed at the bottom of the pillars, immediately above the conventional fixation-clamping section.

6.2.6 Behaviour factors

(726) The behaviour factor shall be chosen according to the capacity of the structure to dissipate earthquake-induced energy. The maximum values of the behaviour factor for different types of structures and classes of ductility are indicated in Table 6.3. The use of the maximum values shall be subject to the conditions relating to the regularity of the building set out in Chapter 4 and the provisions of this Chapter of 6.3 ÷ 6.10.

(727) If the building is irregular in the horizontal plane or in the vertical plane, the values of the behaviour factor shall be reduced according to the provisions of 4.5.1.1 in relation to the values determined in accordance with the provisions of (726).

(728) For structures with high torsion flexibility as defined in 4.2.3, the values of the behaviour factor shall be reduced by 20 % compared to the values established according to the provisions of (726) and (727).

(729) The value of the ratio between the capable horizontal force of the structure and the horizontal force corresponding to the plastic input of the first structural element, α_u/α_1, shall be determined by non-linear static calculation and shall be limited to the upper limit of 1.30.

For buildings in classes of importance and exposure to earthquakes I or II, where the value α_u/α_1 is not be determined by non-linear static calculation, it shall be taken to be equal to 1.0.

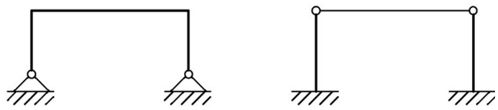
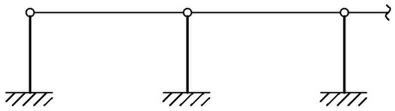
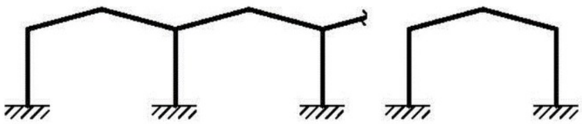
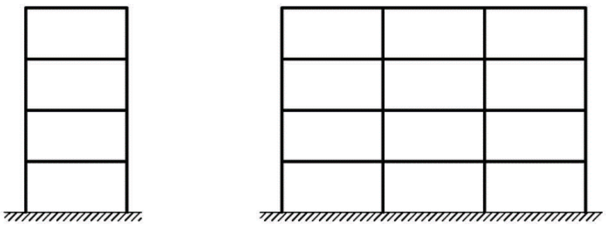
For buildings in classes of importance and exposure to earthquakes III or IV, values α_u/α_1 specified in Table 6.3 may be used, without their determination by non-linear static calculation.

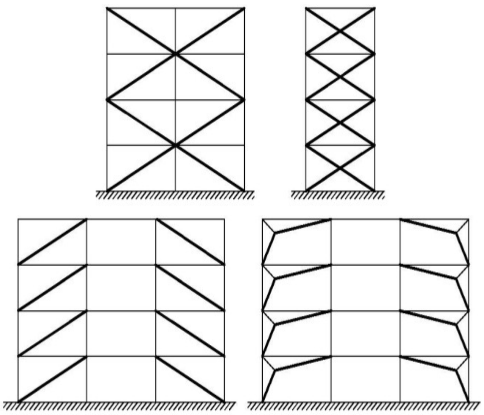
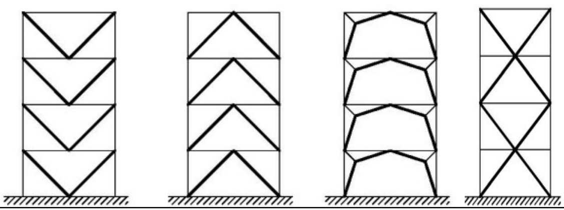
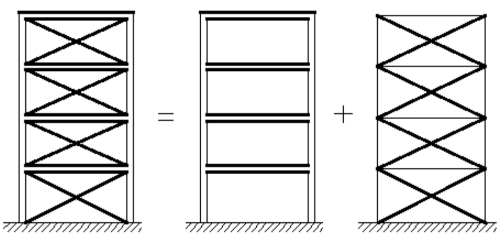
Note: The capable horizontal force of the structure is the force corresponding to the formation of a sufficient number of plastic zones that bring the structure to the threshold of the kinematic mechanism situation. The horizontal force corresponding to yielding of the first structural element is the horizontal force corresponding to the appearance of the first plastic hinge when the resistance capacity is met in the first element of the structure.

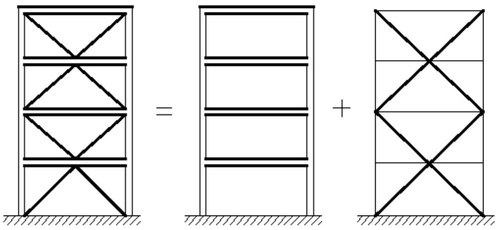
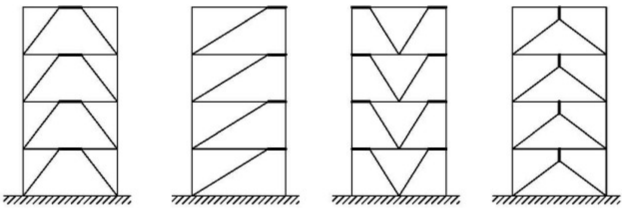
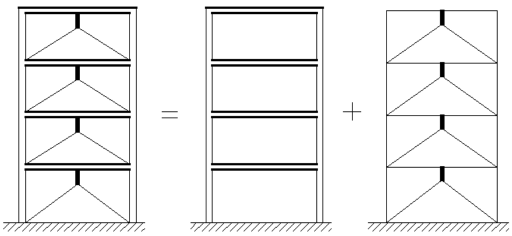
(730) The structure shall conform in such a way as to have the deformation capacity in the inelastic domain as close as possible in both directions.

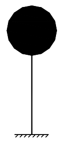
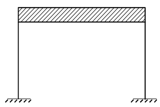
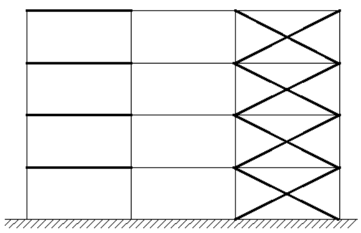
(731) If the building has structural systems of a different type on the two horizontal orthogonal directions, the behaviour factor shall be determined separately in each direction, in relation to the type of structural system used.

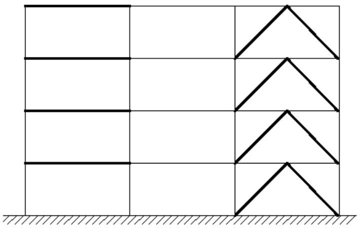
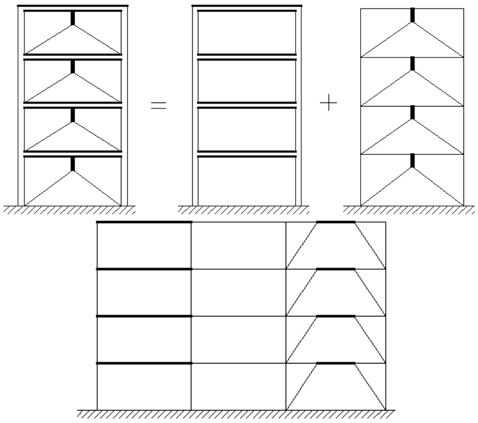
Table 6.1 Maximum value of the behaviour factor, q

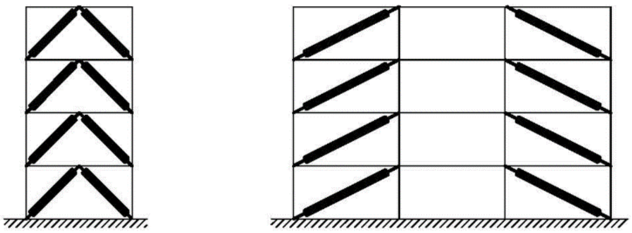
Types of structures	Ductility class of the structure		
	DCH	DCM	DCL
(eeeeeeeeee) Structural system of unbraced frame type			
(i) Single-level buildings			
	2.50	2.00	1.00
	3.00	2.50	1.50
 $\frac{\alpha_u}{\alpha_1}=1,10$ $\frac{\alpha_u}{\alpha_1}=1,00$	$5\frac{\alpha_u}{\alpha_1}$	3.00	1.50
(ii) Multi-level buildings			
 $\frac{\alpha_u}{\alpha_1}=1,20$ $\frac{\alpha_u}{\alpha_1}=1,30$	$5\frac{\alpha_u}{\alpha_1}$	3.50	1.50

Types of structures	Ductility class of the structure		
	DCH	DCM	DCL
(ffffffffff) Centric braced frame structural system (iii) Bracings with diagonals where only the stretched diagonals are formed 	4.00	2.50	1.50
(iv) Bracings with diagonals in V or X on two levels at which the plastic areas are formed in the stretched and compressed diagonals 	2.50	2.00	1.50
(v) Bracings with diagonals in X on one level or with alternating diagonals where the plastic areas are formed in the extended diagonals and at the ends of the beams  $\frac{\alpha_u}{\alpha_1}=1,10$	$4\frac{\alpha_u}{\alpha_1}$	3.00	1.50

Types of structures	Ductility class of the structure		
	DCH	DCM	DCL
(vi) Bracings with V-diagonal where the plastic areas are formed in the stretched and compressed diagonals and at the ends of the beams  $\frac{\alpha_u}{\alpha_1} \leq 1,10$	$2,5\frac{\alpha_u}{\alpha_1}$	2.00	1.50
(gggggggggg) Eccentric braced frame structural system (vii) Eccentric braced frames where plastic areas form in bent or sheared dissipative bars  $\frac{\alpha_u}{\alpha_1} = 1,20$	$5\frac{\alpha_u}{\alpha_1}$	3.0	1.50
(viii) Eccentric braced frames where the plastic areas are formed in bent or sheared dissipative bars and at the ends of the beams  $\frac{\alpha_u}{\alpha_1} = 1,20$	$5\frac{\alpha_u}{\alpha_1}$	3.50	1.50

Types of structures	Ductility class of the structure		
	DCH	DCM	DCL
(hhhhhhhhhh) Inverted pendulum structures (ix) Structures where plastic areas are formed at the base of the pillars  $\frac{\alpha_u}{\alpha_1}=1{,}00$	$2\frac{\alpha_u}{\alpha_1}$	2.00	1.00
(x) Structures where the plastic zones are formed at both ends of the pillars and the axial force in the pillars meets the condition $\frac{N_{Ed}}{N_{pl,Rd}}>0{,}3$  $\frac{\alpha_u}{\alpha_1}=1{,}10$	$2\frac{\alpha_u}{\alpha_1}$	2.00	1.00
(iiiiiiiii) Dual type structural system (xi) Dual frames consisting of unbraced frames connected by rigid diaphragms with X-braced and alternating frames where plastic areas are formed in unbraced frames and stretched diagonals  $\frac{\alpha_u}{\alpha_1}=1{,}20$	$4\frac{\alpha_u}{\alpha_1}$	3.00	1.50

Types of structures	Ductility class of the structure		
	DCH	DCM	DCL
(xii) Dual frames consisting of unbraced frames connected by rigid diaphragms with V-braced frames where the plastic zones are formed in the unbraced frames and diagonally  $\frac{\alpha_u}{\alpha_1}=1{,}10$	$2{,}50\,\frac{\alpha_u}{\alpha_1}$	2.00	1.50
(xiii) Dual frames consisting of unbraced frames connected by rigid diaphragms with eccentrically braced frames where plastic areas form in unbraced frames and bent or sheared dissipative bars  $\frac{\alpha_u}{\alpha_1}=1{,}20$	$5\,\frac{\alpha_u}{\alpha_1}$	3.50	1.50
(xiv) Dual frames consisting of unbraced frames and braced frames with restrained buckling bracing $\frac{\alpha_u}{\alpha_1}=1{,}20$	$5\,\frac{\alpha_u}{\alpha_1}$	3.50	1.50

Types of structures	Ductility class of the structure		
	DCH	DCM	DCL
(jjjjjjjjjj) Frame structural system with obstructed buckling bracing  $\frac{\alpha_u}{\alpha_1}=1,20$	$5\frac{\alpha_u}{\alpha_1}$	-	-
(kkkkkkkkkk) Frame-type structural system with shear panels	4.00	3.00	1.50

(732) Following the penalty of the value of the behaviour factor for irregularities in the horizontal plane, in the vertical plane or for high torsion sensitivity, the value of the behaviour factor q shall be lower limited to 1.50 for structures designed for ductility class DCH or DCM and 1.00 for structures designed for ductility class DCL.

6.3 Structural calculation

(733) The calculation of the structure shall be carried out under the assumption that all the main structural components are active.

(734) By way of exception from (733), for the calculation of structures in centrically braced frames with 'X' diagonals or alternating diagonals by a linear calculation method, the compressed diagonal shall be considered not to take part in seismic action.

(735) For steel-structured buildings, when calculating the design value of seismic action, the fraction of the critical damping of the building, ξ, shall be considered to be equal to 3 %.

(736) The design value of the permissible relative level displacement for service limit state checks shall be determined in accordance with the provisions of 4.3.2.1, (238). Exceptionally, in the case of buildings that do not have non-structural components that may be degraded as a result of horizontal displacements of the structure, the design value of the relative level displacement allowed for checks at the service limit state shall be established with the relation:

$$d_{Rd,r}^{SLS}=0,01\,h_s \qquad (6.1)$$

(737) Movement amplification factor for ultimate limit state checks for steel structures, c, shall be established in relation to:

$$c=\begin{cases}\frac{\Omega_T}{q}+\left(1-\frac{\Omega_T}{q}\right)\frac{T_C}{T_1}\leq 3\, if\, T_1<T_C \\ 1\, if\, T_1\geq T_C\end{cases} \quad (6.2)$$

where

T_1 the fundamental natural period of vibration of the building

T_C the control period of the response spectrum

q behaviour factor;

Ω_T the overstrength factor determined according to the type of structural system.

(738) By way of exception from 4.3.1.2.2, (212), the design value of the permissible relative level displacement at the last limit state for:

(llllllllll) unbraced frame structures;

(mmmmmmmmmm) dual frame structures;

(nnnnnnnnnn) frame structures with obstructed buckling bracing;

(oooooooooo) frame-type structures with shear panels;

shall be established with the relationship:

$$d_{Rd,r}^{SLU}=0{,}020\, h_s \quad (6.3)$$

where

$d_{Rd,r}^{ULS}$ the design value of the relative displacement level allowed for checks at the ultimate limit state;

h_s the total height of the level.

(739) By way of exception from 4.3.1.2.2, (212), the design value of the permissible relative level displacement at the last limit state for:

(pppppppppp) centric braced frame structures;

(qqqqqqqqqq) eccentric braced frame structures;

(rrrrrrrrrr) inverted pendulum structures;

shall be established with the relationship:

$$d_{Rd,r}^{SLU}=0{,}015\, h_s \quad (6.4)$$

where

$d_{Rd,r}^{ULS}$ the design value of the relative displacement level allowed for checks at the ultimate limit state;

h_s the total height of the level.

(740) The calculation of the structure shall be made taking into account the effects of order II according to the provisions of 4.5.5.

(741) Floors shall be designed as horizontal diaphragms as specified in 4.2.6.

(742) For ground floor hall type buildings, which have plane symmetry in both directions and are not equipped with means of lifting or transport, the calculation on flat models can be used to determine the state of stresses and deformations. The calculation shall be made taking into account the additional eccentricity given by the position of the centre of masses in relation to the centre of rigidity of the structure.

6.4 Design of structures for ductility class DCL

(743) Buildings shall be designed for the ductility class DCL so as to meet the basic requirements of seismic design given in Chapter 2.

(744) The resistance capacity of structural components and their joints shall be determined in accordance with SR EN 1993-1-1, SR EN 1993-1-3 and SR EN 1993-1-8.

6.5 Design of structures for ductility class DCM or DCH

6.5.1 General information

(745) The design criteria stipulated in 6.5.2 shall apply to the zones or bars of structures designed in accordance with the concept of dissipative structural behaviour under seismic actions.

(746) Design criteria given at 6.5.2 shall be deemed satisfied if the rules given on 6.5.3 ÷ 6.5.5. are complied with.

6.5.2 Design criteria

(747) Structures shall be designed in such a way that plastic deformation, loss of local stability or other phenomena due to hysteretic behaviour do not result in loss of overall stability of the structure.

(748) Plastic zones shall be directed into structural components specially designed for this purpose, in accordance with the configuration of the optimal plastic mechanism defined in 6.2.5.

(749) The main structural components that deform plastic at seismic action, corresponding to the ultimate limit state, according to the configuration of the optimal mechanism, shall be made in such a way as to meet the criteria of ductility and resistance.

(750) The main structural components that respond elastically to the design seismic action, corresponding to the ultimate limit state, shall be made in such a way as to have sufficient resistance to limit the development of plastic deformations in plastic areas established according to the optimal plastic mechanism.

(751) All joints between the main structural components shall be designed to respond elastically to the design seismic action corresponding to the ultimate limit state.

(752) By way of exception from (751), when constructing structures with an elasto-plastic response to design seismic action, corresponding to the ultimate limit state, joints with dissipative components may be used. They shall be selected for use in the project on the basis of a technical approval containing provisions for their use under seismic stress conditions. In this situation, the main structural components shall be designed in such a way that their resistance favours the development of cyclic plastic deformations in the respective joints.

6.5.3 Design rules for dissipative elements subjected to compressive and/or bending actions

(753) The elements that deform plastically at the design seismic action, corresponding to the ultimate limit state, due to compression and/or bending, shall be designed in such a way that they have sufficient ductility by limiting the suppleness of the section walls, according to the cross section classes defined in SR EN 1993-1-1.

6.5.4 Design rules for tensile elements

(754) The ductility requirements for stretched elements are given in SR EN 1993-1-1.

(755) The provisions of SR EN 1993-1-1 shall be used to check the elements subject to stretching.

6.5.5 Design rules for joints in dissipative zones

(756) The construction of joints in the vicinity of plastic areas shall be carried out in such a way as to limit the occurrence of high residual stresses, defects of execution and to direct the development of plastic deformations in specially conformed areas for this purpose.

(757) The joints of the dissipative elements made with weld or rods shall be designed so as to meet the condition:

$$F_{Rd,j} \geq \omega_{rm} \omega_{sh} F_{Rd,pl} \qquad (6.1)$$

where,

$F_{Rd,j}$ the design value of the joint strength corresponding to the stress to which it is subjected, determined in accordance with the provisions of SR EN 1993-1-8;

$F_{Rd,pl}$ the design value of the plastic strength capacity of the joining element, corresponding to the effort to which it is subjected, determined in accordance with the provisions of 6.6.2, 6.7.3 and 6.8.2 using the nominal value of the flow limit of the steel;

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, determined in accordance with the provisions of Table 6.2;

ω_{sh} the overstrength factor taking into account the strengthening effect of the steel, the value of which shall be determined in accordance with the provisions of Table 6.4.

Table 6.1 Overstrength factor values ω_{sh}

Types of structural system	Plastic areas	Effort that causes plastic deformation	ω_{sh}
Unbraced frame type	beams	bending	$\frac{(f_y+f_u)}{2f_y} \leq 1,2$
	the base of the pillars		
centric braced frame	diagonal bars	axial force	1.10

type or dual type with centric braced frames	other structural components	bending	1.10
Eccentric braced frame type or dual type with eccentric braced frames	short dissipative bar	shear force $e \leq M_{p,link}/V_{p,link}$ (very short dissipative bar)	1.80
		shear force $M_{p,link}/V_{p,link} < e$ $e \leq 1{,}60\, M_{p,link}/V_{p,link}$ (short dissipative bar)	1.50
	intermediate dissipative bar	bending and shear force $e \leq 2{,}60\, M_{p,link}/V_{p,link}$	1.50
		bending and shear force $2{,}60\, M_{p,link}/V_{p,link} < e$ $e \leq 3{,}00\, M_{p,link}/V_{p,link}$	1.35
	long dissipative bar	bending $3{,}00\, M_{p,link}/V_{p,link} < e$ $e \leq 5{,}00\, M_{p,link}/V_{p,link}$	1.25
		bending $e > 5{,}00\, M_{p,link}/V_{p,link}$	$\omega_{sh} = \frac{(f_y + f_u)}{2 f_y} \leq 1,$
	beam-pillar joint	bending	1.10
Buckling-restrained braced frames or dual-frame structures with restrained buckling bracing	restrained buckling bracing	axial	$\omega_{sh} \leq 1{,}50$ According to the provisions of 6.10
	beam-pillar joint	bending	1.20
Frames with shear panels	the core of the shear panels	diagonal field of unit tensile effort	1.10

where:

$M_{p,link}$ design value of the bending strength of the dissipative bar;

$V_{p,link}$ design value of the cutter shearing resistance capacity of the dissipative bar;

e length of the dissipative bar;

f_y the nominal value of the flow limit of steel.

(758) The overstrength of the joints of the dissipative elements made with deep welding with full penetration, acceptance level B, according to the technical

regulations in force regarding the quality of the steel welded joints of the constructions, shall be established in accordance with the provisions of the technical approval of the welding process.

(759) Screw joints required in the plane of the joint, perpendicular to the rods, shall be made with screws of quality classes 8.8 or 10.9 as:

(ssssssssss) sliding joints of category B according to SR EN 1993-1-8, or

(tttttttttt) joints working by contact between parts and rods, by shearing the rod and pressure on the walls of the hole, without processing the surfaces in contact, the pre-tensioning force of the rods being equal to at least 50 % of the force capable of pre-tensioning the screws. The effect of pre-tensioning shall not be taken into account when calculating the strength of the joint.

(760) Screw joints required in the joint plane shall be made in such a way that the shear strength of each screw is at least 20 % higher than the pressure resistance on the walls of the hole.

(761) Screw joints required at tension perpendicular to the plane of the joint shall be made with screws of quality classes 8.8 or 10.9 pre-tensioned so that the pre-tensioning force is greater than or equal to 50 % of the force capable of pre-tensioning the screws. Category E joints shall be used in accordance with the provisions of SR EN 1993-1-8. No processing of contact surfaces is required.

(762) Screw joints subjected to complex loads, in the plane of the joint and perpendicular to its plane, shall be made with screws of quality classes 8.8 or 10.9. Joints of categories B and C according to the provisions of SR EN 1993-1-8 with full pre-tensioning or joints without processing of the surfaces in contact but at which the rods are pre-tensioned so that the pre-tensioning force is greater than or equal to 50 % of the force capable of pre-tensioning the screws are allowed. The effect of pre-tensioning shall not be taken into account when calculating the strength of the joint.

(763) Welding beads shall not be used in screw joints to balance efforts.

6.5.6 Design rules for joints in non-dissipative areas

(764) Joints in non-dissipative areas shall be constructed in such a way as to prevent the development of plastic deformations and the occurrence of high residual stresses.

(765) Joints of non-dissipative elements made with welding or rods shall be made in such a way as to comply with the provisions of SR EN 1993-1-8.

(766) Joints of non-dissipative elements shall be made in such a way as to meet the requirement:

$$F_{Rd,j} \geq \omega_{rm} F_{Rd,pl} \tag{6.1}$$

where,

$F_{Rd,j}$ the design value of the joint strength corresponding to the stress to which it is subjected, determined in accordance with the provisions of SR EN 1993-1-8;

$F_{Rd,pl}$ the design value of the plastic strength capabilities of the joining element, corresponding to the effort to which it is subjected, determined using the nominal value of the flow limit of steel;

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, determined in accordance with the provisions of Table 6.2.

6.5.7 Design rules for fastening pillars in foundations

(767) Anchor bolts shall be so designed that the design value of their tensile strength capability is greater than or equal to the maximum tensile stress that may develop in the screw at the incidence of design seismic action, corresponding to the ultimate limit state.

(768) The design values of the effects of the actions at the base of the pole are determined with the relation:

$$E_{Fd} = E_{Fd,G} + \Omega_T \cdot E_{Fd,E} \qquad (6.1)$$

where

E_{Fd} the design value of the effect of actions in the seismic grouping of actions;

$E_{Fd,G}$ the design value of the effect of non-seismic actions in the seismic grouping of actions;

$E_{Fd,E}$ design value of the seismic action effect in the seismic grouping of actions, corresponding to the ultimate limit state

Ω_T the value of the structure's overstrength to horizontal actions:

$$\Omega_T = \omega_{rm} \cdot \omega_{sh} \cdot \Omega_d \qquad (6.2)$$

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, according to the provisions of Table 6.2;

ω_{sh} the overstrength factor taking into account the strengthening effect of the steel, the value of which shall be determined in accordance with the provisions of Table 6.4.

Ω_d the minimum value of the ratio between the design value of the plastic area resistance capacity and the design value of the effect of actions in the seismic grouping, for the relevant stress; Ω_d shall be calculated according to the type of structural system for each main direction of the structure.

(769) Value of structural system overstrength Ω_T may be limited in such a way that the condition is met; $\Omega_T \leq q$, where q is the behaviour factor of the structure used to determine the design values of the effects of seismic action.

(770) When designing buildings in the class of importance and exposure to earthquakes III or IV, for determining the values Ω_T, the provisions of Table 6.5 can be used.

Table 6.1 Values of structural system overstrength, Ω_T

Structural system of the following type:	Ω_T		
	DCH	DCM	DCL
Unbraced, single-level frame	2.50	1.70	1.50
Unbraced, multi-level frame	3.50	2.00	1.50

Centre braced frame, with ‘X’ or diagonal bracings	2.50	1.50	1.50
Centre braced frame with inverted ‘V’ bracelets	2.00	1.50	1.50
Eccentric braced frame	3.00	2.00	1.50
Inverted pendulum	2.20	1.70	1.50
Frame with restrained buckling bracing	3.00 γ_{CT}	-	-
Dual, consisting of:			
- unbraced frames and centrically-chained frames with an "X" diagonal	3.00	1.70	1.50
- unbraced frames and centrically-chained frames with diagonals in ‘V’	2.50	1.70	1.50
- unbraced frames and eccentric braced frames	3.00	2.00	1.50
- unchained frames and braced frames with restrained buckling	3.50	-	-
Frame with shear panels	2.50	2.00	1.50
where γ_{CT} is the correction factor for the compression strength of the bracing.			

(771) When the base of the pillar is not embedded in the foundation system or infrastructure, in order to avoid brittle fracture, the fastening detail of the pillars shall be made in such a way as to ensure a free deformation area of the anchor bolts of minimum $5d$, where d is the diameter of the screw rod.

(772) It is recommended that the transmission of shear forces between pillars and foundations or infrastructure is not carried out by means of anchor bolts. For this, one of the following constructive solutions can be used:

(uuuuuuuuuu) embedding the base of the pillar in a reinforced over-concreting on a height equal to at least 40 cm or 0,50 of the height of the cross-section of the pillar;

(vvvvvvvvvv) the provision of welded elements under the base plate of the pillar, which will be embedded in gaps specially made in foundations or infrastructure, along with the under-concreting of the base; these elements shall be dimensioned in such a way that they can transmit the shear force from the base of the pillar to the foundation;

(wwwwwwwwww) embedding the pillar in foundations or infrastructure on a height that ensures its direct anchorage, without the need for anchor bolts.

(773) In the case of embedded mounting of the pillar in foundations, when checking the base of the pillar, the condition is fulfilled:

$$M_{Rd,j} \geq \omega_{rm} \cdot \omega_{sh} \cdot M_{N,Rd,c} \tag{6.3}$$

where

$M_{Rd,j}$ the design value of the bending strength at the moment of the clamping of the foundation pillar;

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, according to the provisions of Table 6.2;

ω_{sh} the overstrength factor taking into account the strengthening effect of the steel, the value of which shall be determined in accordance with the provisions of Table 6.4;

$M_{N,Rd,c}$ the design value of the resistance capacity at the bending moment, in the presence of the axial force, of the cross section of the pillar.

6.5.8 Continuity joints for pillars

(774) Continuity joints shall be made in such a way as to ensure the continuity of the rigidity and strength of the pillars.

(775) Continuity joints of pillars shall be positioned in such a way that all of the following conditions are met:

(xxxxxxxxxx) the distance from the infill joint to the bottom of the pillar at that level is approximately 1/3 of the floor height;

(yyyyyyyyyy) the distance from the continuity joint to the nearest beam-pillar joint is greater than or equal to 1.20 m.

(776) Continuity joints of pillars shall be carried out with welding or screws.

(777) The design values of the joint effort shall be taken to be greater than or equal to the design values of the load capacities of the joining pillar sections.

(778) The design values of the strength capacities of the continuity joints shall be determined in accordance with the provisions of SR EN 1993-1-8.

(779) Continuity joints of the pillars shall be made in such a way that the following condition is fulfilled, on both main horizontal directions of the pillar section:

$$V_{Rd,c} \geq \frac{2M_{N,Rd,c}}{h_s} \quad (6.1)$$

where:

$V_{Rd,c}$ the design value of the shear force strength capability of the pillar continuity joint;

$M_{N,Rd,c}$ design value of the tensile strength at the moment of bending of the pillar section in the presence of axial force;

h_s level height.

6.5.9 Buckling lengths for the pillars of multi-storey structures

(780) The provisions of this paragraph shall apply to the determination of the buckling lengths of pillars of multi-stage structures made with bars of constant cross-section along their length.

(781) The provisions of this paragraph shall apply if specific technical regulations concerning metal structures of different types for buildings do not include specific provisions for determining the buckling lengths of pillars.

(782) For the purposes of this paragraph, frames of braced, dual or shear-walled structures may be considered as having fixed joints if the vertical bracing systems or shearing walls reduce the horizontal movements of the frame by at least 80 %. For

other situations, framing in frames with shiftable joints or fixed joints shall be carried out in accordance with the provisions of SR EN 1993-1-1.

(783) For the purposes of this paragraph, the following shall be considered:

(zzzzzzzzzz) in the case of pillars, the length of the bar, L, is equal to the level height;

(aaaaaaaaaaa) in the case of beams, the length of the bar, L, is considered equal to the aperture of the beam.

(784) Ratio of buckling length, L_{cr}, and the length of the bar, L, for a pillar in a fixed joint frame at a given level, shall be determined using the diagram given in Figure 6.1.

(785) Ratio of buckling length, L_{cr}, and the length of the bar, L, for a pillar in a frame with shiftable joints, from a given level, shall be determined using the diagram given in Figure 6.2.

(786) In application of the provisions of (5) and (6), the rigidity distribution factors shall be determined with the relations:

$$\eta_1 = \frac{K_C + K_1}{K_C + K_1 + K_{11} + K_{12}} \tag{6.1}$$

$$\eta_2 = \frac{K_C + K_2}{K_C + K_2 + K_{21} + K_{22}} \tag{6.2}$$

where

η_1, η_2 the factors of distribution of the rigidity of the joints at the top end and the bottom end of the pillar;

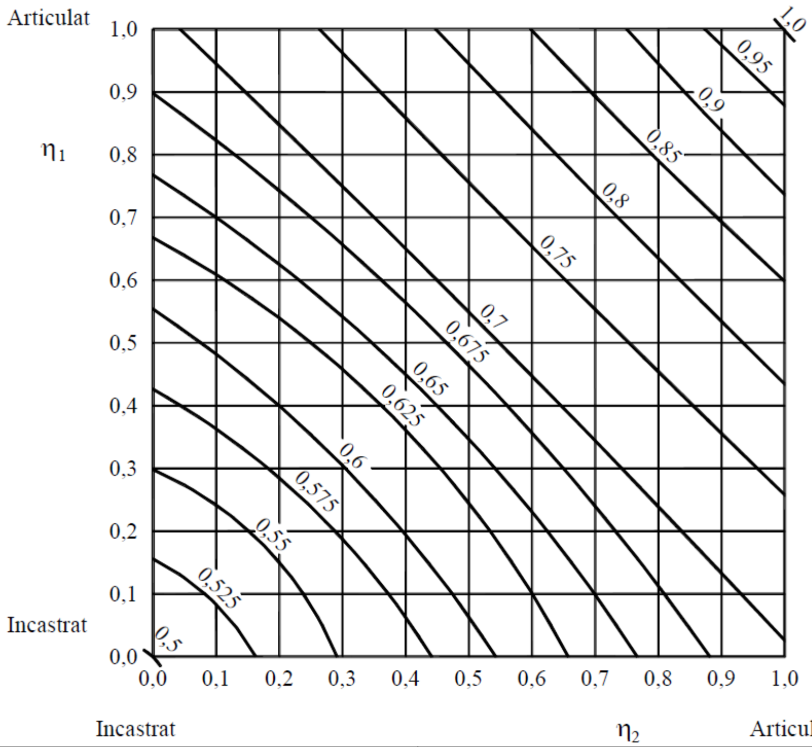
K_C rigidity factor of the pillar at the level considered, determined in accordance with (15);

K_1 rigidity factor of the pillar above the level considered, determined in accordance with (15);

K_2 rigidity factor of the pillar at the level below the level considered, determined in accordance with (15);

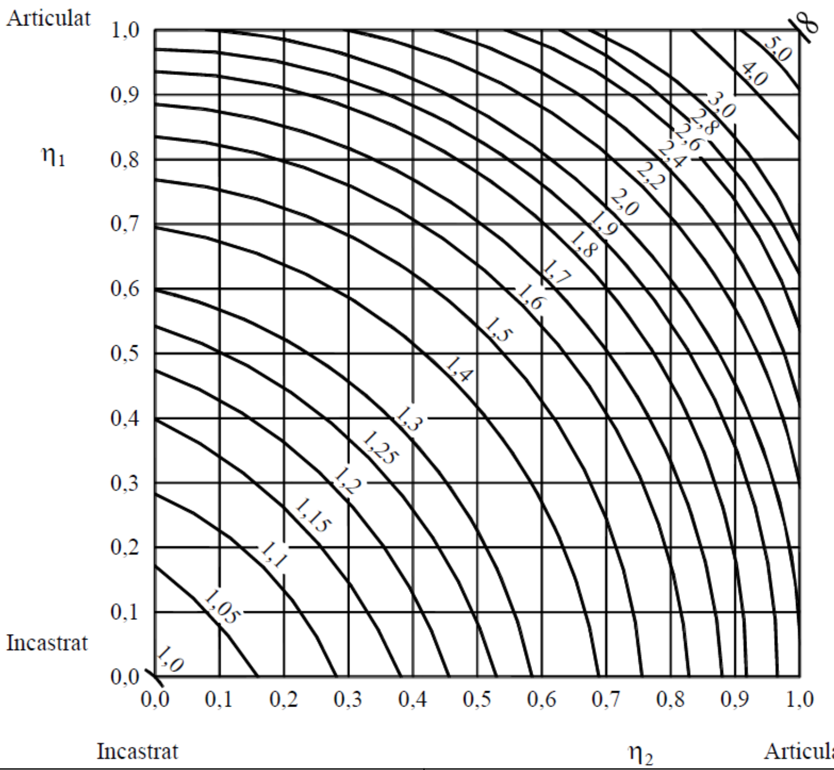
K_{11}, K_{12} the rigidity factors of the beams intersecting the pillar at the top of it, in the load plane, established according to the provisions of (9), (10), (11) and (12);

K_{21}, K_{22} the rigidity factors of the beams intersecting the pillar at its bottom, in the load plane, established according to the provisions of (9), (10), (11) and (12);



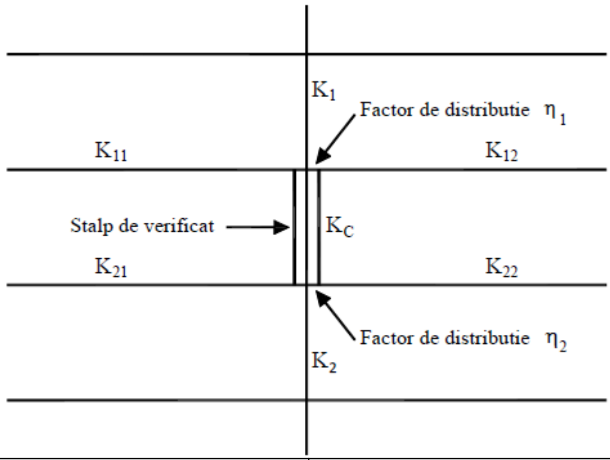
Articulat	Hinged
Incastrat	Embedded

Figure 6.2 Ratio of buckling length to bar length, L_{cr}/L, for a pillar in a frame with fixed joints



Articulat	Hinged
Incastrat	Embedded

Figure 6.3 Ratio of buckling length to bar length, L_{cr}/L, for a pillar in a frame with movable joints



Factor de distribuție	Distribution factor
Stâlp de verificat	Check pillar

Figure 6.4 Distribution factors for continuous pillars

(787) Alternatively to the provision of (784) or (785), the ratio of the buckling length, $L_{cr}$and the length of the bar,L, for a pillar can be determined with the relation:

(bbbbbbbbbbb) for fixed joint frames:

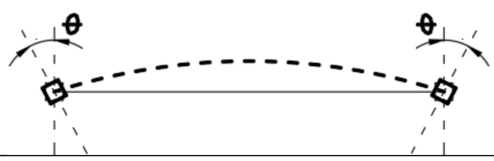
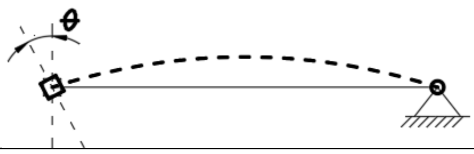
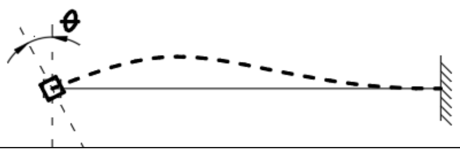
$$\frac{L_{cr}}{L}=\left[\frac{1+0{,}145\left(\eta_1+\eta_2\right)-0{,}265\,\eta_1\,\eta_2}{2-0{,}364\left(\eta_1+\eta_2\right)-0{,}247\,\eta_1\,\eta_2}\right] \tag{6.1}$$

(ccccccccccc) for frames with movable joints:

$$\frac{L_{cr}}{L}=\left[\frac{1-0{,}2\left(\eta_1+\eta_2\right)-0{,}12\,\eta_1\,\eta_2}{2-0{,}8\left(\eta_1+\eta_2\right)-0{,}60\,\eta_1\eta_2}\right]^{0{,}5} \tag{6.2}$$

(788) The rigidity factors of elastic response beams, part of fixed joint frames, which are not loaded by axial force, shall be determined according to the provisions of Table 6.6.

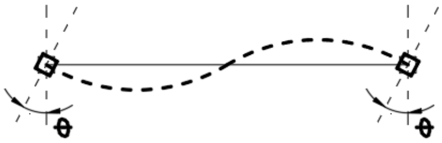
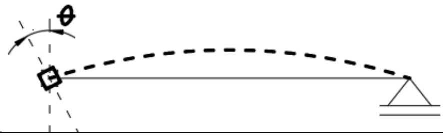
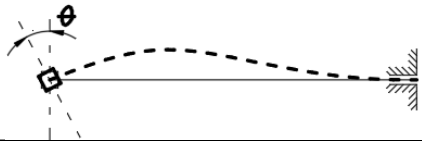
Table 6.1 Rigidity factor of frame beams with fixed joints

Case	Rigidity factor K of beams in the case of fixed joint frames	
1		$K=0{,}50\frac{I}{L}$
2		$K=0{,}75\frac{I}{L}$
3		$K=1{,}00\frac{I}{L}$
where:		

I	the moment of inertia of the section of the beam in relation to the axis to which the bending occurs;
L	the length over which the buckling of the bar manifests itself.

(789) The rigidity factors of elastic response beams, part of fixed joint frames, which are not loaded by axial force, shall be determined according to the provisions of Table 6.7.

Table 6.2 Rigidity factor of beams in case of frames with movable joints

Case	Rigidity factor K of beams in the case of frames with movable joints	
1		$K=1{,}5\frac{I}{L}$
2		$K=0{,}75\frac{I}{L}$
3		$K=1{,}0\frac{I}{L}$

where:

I	the moment of inertia of the section of the beam in relation to the axis to which the bending occurs;
L	the length over which the buckling of the bar manifests itself.

(790) In the case of buildings in rectangular frames with concrete floors, with the topology of the regular structure and uniform loading, the rigidity factors of the beams can be determined according to the provisions of Table 6.8.

Table 6.3 Rigidity factor of a beam in a structure with reinforced concrete floors

Rigidity factor K of a beam in a structure with reinforced concrete floors		
Beam loading requirements	Structure with fixed joints	Structure with movable joints
Beams which provide direct support for the reinforced concrete floor slab	$K=1{,}0\frac{I}{L}$	$K=1{,}0\frac{I}{L}$
Other directly loaded beams	$K=0{,}75\frac{I}{L}$	$K=1{,}0\frac{I}{L}$
Beams that only bear the action of moments occurring at the extremities	$K=0{,}5\frac{I}{L}$	$K=1{,}5\frac{I}{L}$
where:		

I	the moment of inertia of the section of the beam in relation to the axis to which the bending occurs;
L	the length over which the buckling of the bar manifests itself.

(791) If the design value of the bending moment in a section is greater than the elastic strength capacity at the bending moment, $W_{el} \ f_y$, in application of the provisions (9) and (10), the beam can be considered articulated in that section.

(792) In the case of beams required by axial force, the value of the rigidity factor shall be corrected using stability functions.

(793) Alternatively to the provision of (13), the value of the rigidity factor for beams required by axial force may be determined according to the provisions of Table 6.9, for fixed joint frames, and Table 6.10, for frames with movable joints.

Table 6.4 Rigidity factor of frame beams with fixed joints

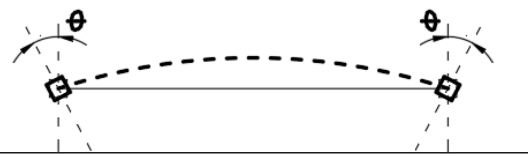
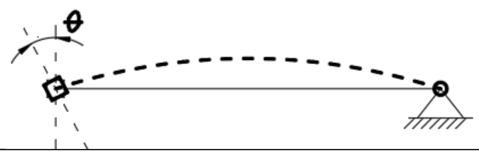
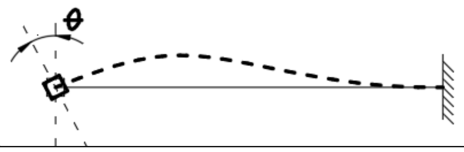
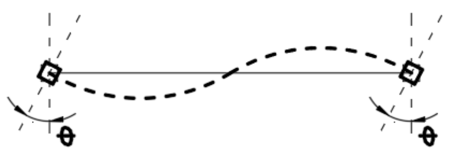
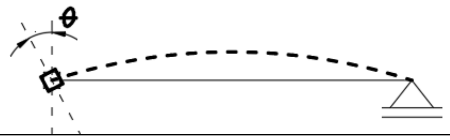
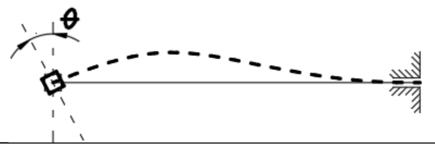
Case	Rigidity factor K of beams in the case of frames with fixed joints	
1		$K=0{,}5\frac{I}{L}\left(1-1{,}0\frac{N}{N_E}\right)$
2		$K=0{,}75\frac{I}{L}\left(1-1{,}0\frac{N}{N_E}\right)$
3		$K=1{,}0\frac{I}{L}\left(1-1{,}0\frac{N}{N_E}\right)$
where $N_E=\pi^2 EI/L^2$ and N is the axial beam force in the seismic load grouping used for checking pillars		

Table 6.5 Rigidity factor of frame beams with movable joints

Case	Rigidity factor K of beams in the case of frames with movable joints	
1		$K=1{,}5\frac{I}{L}\left(1-0{,}2\frac{N}{N_E}\right)$
2		$K=0{,}75\frac{I}{L}\left(1-1{,}0\frac{N}{N_E}\right)$
3		$K=1{,}0\frac{I}{L}\left(1-0{,}4\frac{N}{N_E}\right)$
where $N_E=\pi^2 EI/L^2$ and N is the axial beam force in the seismic load grouping		

used for checking pillars

(794) Rigidity factor of pillars K shall be determined with the relations given for the beams in Table 6.9 and Table 6.10, considering case 1 as end-to-end links.

(795) At the first level of the building, the rigidity factor of the embedded pillars at the base shall be determined considering that the distribution factor of the rigidity of the joint at the bottom end of the pillar η_2 is equal to 0.

6.6 Frames without wind bracing

6.6.1 Design criteria

(796) Unbraced frames must be designed in such a way that plastic joints form in beams. The formation of plastic joints and in pillars according to 6.2.5, (1), (a).

(797) Depending on the chosen dissipative areas, the provisions of 6.5.2 (4), i.e. 6.5.2 (4) shall apply.

(798) Formation of plastic joints in specially conformed areas in the structure can be achieved by observing 6.6.2. and 6.6.3.

6.6.2 Beams

(799) For checking and conforming the beams to general stability, the provisions of SR EN 1993-1-1 shall be used considering the hypothesis that only at one end a plastic joint was formed, and at the other end the bending moment generated by the loads in the design seismic group develops. For checking the overall stability of the beam, at the less required end the design value of the bending moment caused by the loads in the seismic group shall be considered, and at the other end a bending moment equal to the plastic resistance capacity of the beam section shall be considered.

(800) In the plastic areas of the beams, according to the configuration of the optimal plastic mechanism provided in 6.2.5, the core of the beams shall be made without gaps. The same shall apply to the areas at the ends of the beam measured at a distance equal to $3h_w$, in relation to the face of the supporting pillars, where h_w is the height of the core of the beam. In the case of use in the plastic zone of a beam of a reduced section, the distance shall be measured from the middle of the length of this zone.

(801) The loss of the overall stability of the beam may be prevented by providing lateral connections to the compressed sole placed at distances complying with the provisions for $L_{stabilă}$ of SR EN 1993 1-1.

(802) The beam shall be made in such a way as to meet the following conditions in potentially plastic areas:

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1{,}00 \tag{6.1}$$

$$\frac{N_{Ed}}{N_{pl,Rd}} \leq 0{,}15 \tag{6.2}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0{,}50 \tag{6.3}$$

where

N_{Ed}, M_{Ed}, V_{Ed} design values of axial force, bending moment and shear force, in the load grouping including seismic action;

$$V_{Ed} = V_{Ed,G} + V_{Ed,M} \qquad (6.4)$$

$V_{Ed,G}$ shear force from non-seismic actions contained in the load grouping including seismic action;

$V_{Ed,M}$ shear force corresponding to the loading of the beam in the plastic zones ‘A’ and ‘B’ at the ends, with bending moments corresponding to the design values of the bending strength capabilities, with the opposite sign:

$$V_{Ed,M} = (M^{A}_{pl,Rd} + M^{B}_{pl,Rd}) / L_{AB} \qquad (6.5)$$

L_{AB} the distance between the plastic areas developing in the same beam aperture, but not more than 0.90 of the free beam aperture measured between the side faces of the pillars;

$N_{pl,Rd}$, $M_{pl,Rd}$, $V_{pl,Ed}$ design values of axial force strength, bending moment and shear force.

(803) If any of the relations (6.1), (6.2) or (6.3) are not met, the value of the moment $M_{pl,Rd}$ from the relation (6.1) shall be replaced by a reduced value to take account of the influence of axial force and/or shear force as provided for in SR EN 1993 1-1.

(804) For Class 3 sections, in relation to (6.1) is replaced $M_{pl,Rd}$with $M_{el,Rd}$and the verification relations in SR EN 1993-1-1 apply.

(805) At the ends of potentially plastic areas and in areas where concentrated loads are applied, lateral supports shall be made for both beam soles

(806) In the case of floors with beams that work together with the reinforced concrete slab, no connectors shall be placed along the length of the considered potential plastic area to ensure the collaboration.

(807) Side supports determined in accordance with the (805) in the plastic areas of the beams shall be made in such a way that they can balance a horizontal force greater than or equal to $0{,}06\,\omega_{rm} f_y t_f b$. The other lateral supports shall be such that they can balance a horizontal force equal to or greater than$0{,}02\,\omega_{rm} f_y t_f b$.

(808) For directing the plastic joints in the beam, the width of the soles can be reduced in the vicinity of the beam-post joint according to 6.6.2. (809). The reduced section shall be checked at the final limit state considering the design efforts in the load grouping including seismic action.

(809) The low-section beam-pillar joint shall be obtained by cutting the soles in the area adjacent to the pillar, to direct the formation of the plastic joint in the low-section area of the beam, subject to the following conditions:

$$\begin{aligned} &0{,}50\, b_{bf} \le a \le 0{,}75\, b_{bf} \\ &0{,}65\, d \le b \le 0{,}85\, d \\ &0{,}10 \le c \le 0{,}25\, b_{bf} \\ &R = (b^2 + 4c^2)/8c. \end{aligned} \qquad (6.6)$$

where:

a distance from the front of the pillar to the reduced area;

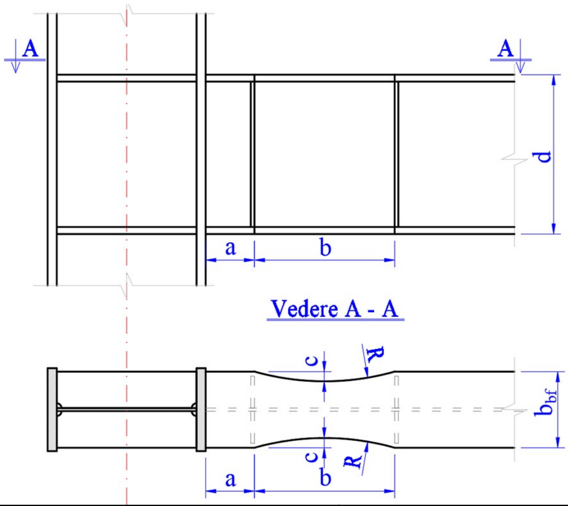
b length of the reduced area;

c the maximum reduction of the beam sole;

d the height of the section of the beam;

b_{bf} the width of the unreduced sole of the section of the beam;

R cutting radius.



Vedere A - A	View A - A

Figure 6.2 Jointing of a low-section beam

(810) The cutting of the soles shall be carried out by a technological process which ensures a smooth cut so that no indentations or other defects appear which are numb to cracks. The cut shall be such that the roughness of the cut surface is less than or equal to 13 µm and the connection between the cut and the unaltered sole shall be rounded. The edges of the soles in the cut-out areas grind.

6.6.3 Pillars

(811) The strength of the pillars shall be checked in each seismic combination of loads. Strength and stability checks shall be carried out on the basis of the provisions of SR EN1993-1-1. The design values of the efforts shall be determined with the relations:

$$N_{Ed}=N_{Ed,G}+\Omega_T N_{Ed,E} \tag{6.1}$$

$$M_{Ed}=M_{Ed,G}+\Omega_T M_{Ed,E} \tag{6.2}$$

$$V_{Ed}=V_{Ed,G}+\Omega_T V_{Ed,E} \tag{6.3}$$

where:

N_{Ed}, M_{Ed}, V_{Ed} design values of axial force, bending moment and shear force;

$N_{Ed,G}$, $M_{Ed,G}$, $V_{Ed,G}$ axial force, bending moment and shear force, from non-seismic actions in the load grouping including seismic action;

$N_{Ed,E}$, $M_{Ed,E}$, $V_{Ed,E}$ axial force, bending momentum and shear force, from design seismic action;

Ω_T the value of the structural system overstrength, which shall be determined with respect to the relation:

$$\Omega_T = \omega_{rm} \cdot \omega_{sh} \cdot \Omega_d \qquad (6.4)$$

Ω_d minimum ratio value $\Omega_{M,i} = M_{pl,Rd,i} / M'_{Ed,i}$ calculated for all dimensioned beams in the load grouping comprising seismic action;

$M_{pl,Rd,i}$ design value of the plastic resistance capacity at the bending moment of the beam "i";

$M_{Ed,i}$ the value of the bending moment in the beam "i", resulting from the calculation of the structure in the grouping of actions that includes seismic action.

For each beam of the structure, a single ratio value shall be calculated $\Omega_{M.i}$, at the end of the beam where the absolute value of the bending moment $¿M_{Ed,i} \vee ¿$ has the maximum value.

Its value $\Omega_d$shall be calculated for each seismic design combination.

Note: Values of efforts N_{Ed}, M_{Ed}, V_{Ed} can be obtained from seismic groups, where the one-way seismic action is multiplied by Ω_T.

(812) Value of overstrength Ω_T may be limited upwards so that the condition is met $\Omega_T \leq q$, where q is the behaviour factor of the structure. When designing buildings classified as class III or IV of importance and exposure to earthquakes, overstrength values Ω_T provided for in Table 6.5 may be used.

(813) For each horizontal orthogonal direction of the structure, the beams shall be such that the difference between the maximum value and the minimum value of the ratio $\Omega_{M.i}$, determined in accordance with (1), be less than or equal to 25 % of the maximum value.

(814) By way of exception from (3), where a difference between the maximum value and the minimum value of the ratio cannot be ensured $\Omega_{M.i}$, determined in accordance with (1), less than or equal to 25 % of the maximum value, the plastic mechanism of the structure shall be checked by non-linear, static or dynamic calculation.

(815) In plastic areas established in accordance with the provisions of 6.2.5, the pillars shall be made with the section class established in accordance with the provisions of Table 6.1. In the elastic response zones, the pillars shall be made with section class 1, 2 or 3.

(816) In the case of the design of the structure in the ductility class DCH, the pillars and beams shall be such that the condition is met:

$$\sum M_{N,pl,Rd,c} \geq \sum \left[\omega_{rm} \omega_{sh} \left(M_{pl,Rd,b} + s_{h,c} V_{Ed,M} \right) + s_{h,c} V_{Ed,G} \right] \qquad (6.5)$$

where:

$\sum M_{N,pl,Rd,c}$ sum of the bending strength of the sections of the pillar entering the joint, in the presence of axial force;

$M_{N,pl,Rd,c} = M_{pl,Rd,c} (1-n)/(1-0{,}5a)$ conditioned by
$M_{N,pl,Rd,c} = M_{pl,Rd}$ (6.6)

$n = N_{Ed} / N_{pl,Rd}$

$a = A_v / A$ conditioned by $a \leq 0{,}5$

$\sum M_{pl,Rd,b}$ sum of the plastic resistance capacities at the moment bending of the sections of the beams entering the joint;

$V_{Ed,M}$ the shear force corresponding to the loading of the beam in the plastic areas, with the bending moments corresponding to the design values of the strength capacities, with the opposite sign (see 6.6.2, (4));

$V_{Ed,G}$ shear force caused by non-seismic actions in the load grouping including seismic action (see 6.6.2, (4));

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, according to the provisions of Table 6.2;

ω_{sh} the overstrength factor taking into account the strengthening effect of the steel, the value of which shall be determined in accordance with the provisions of Table 6.4;

$s_{h,c}$ the distance measured horizontally between the middle of the plastic zone of the beam and the vertical axis of the pillar.

(817) In the plastic areas of the pillars, the ratio of the shear force resulting from the calculation of the structure, V'_{Ed}, and the design value of the plastic shear force resistance capability shall meet the condition:

$$\frac{V'_{Ed}}{V_{pl,Rd}} \leq 0{,}5 \qquad (6.7)$$

(818) The slenderness of the pillar shall fulfil the condition:

(dddddddddd) in the plane of the frames in which the beams may form plastic joints:

$$\lambda \leq 0{,}85\pi\sqrt{\frac{E}{f_y}} \qquad (6.8)$$

(eeeeeeeeee) in the plane of frames in which plastic joints cannot form in beams:

$$\lambda \leq 1{,}30\pi\sqrt{\frac{E}{f_y}} \qquad (6.9)$$

where

λ slenderness of the pillar;

E modulus of elasticity of steel;

f_y design value of the steel flow limit.

6.6.4 Beam-pillar joints

(819) The beam-pillar joints shall be designed in accordance with the provisions of SR EN 1993-1-8.

(820) In structures where the optimal plastic mechanism according to 6.2.5 is formed by the development of plastic deformations in beams, beam-pillar joints and pillar-core panels in the joint area shall be carried out in such a way that the condition is met:

$$M_{con,Rd} \geq \omega_{rm} \omega_{sh} \left(M_{b,pl,Rk} + s_{h,con} V_{Ed,M} \right) + s_{h,con} V_{Ed,G} \qquad (6.1)$$

where:

$M_{con,Rd}$ the design value of the bending strength of the joint;

$M_{b,pl,Rk}$ characteristic value of the plastic capable moment of the beam in the beam-pillar joint;

$s_{h,con}$ the distance measured horizontally between the middle of the plastic zone of the beam and the vertical axis of the joint;

(821) The stresses shall be transferred from the beams to the pillars for the hypothesis of a rigid beam-pillar joint. At the core of the pillar, next to the soles of the beams, pairs of transverse stiffeners shall be provided, with a thickness at least equal to that of the sole of the beam and a total width at least equal to that of the sole of the beam.

(822) The core panels of the pillars in the area of the beam-pillar joints shall be made to meet the condition:

$$V_{\wp,Ed} \leq V_{\wp,Rd} \qquad (6.2)$$

where:

$V_{\wp,Ed}$ design value of the shear force in the panel:

$$V_{\wp,Ed} = \frac{M_{pl,Rd,i}^{cor} + M_{pl,Rd,j}^{cor}}{h_b - t_{fb}} \qquad (6.3)$$

$M_{pl,Rd,i}^{cor}$.the design values of the currently bending plastic resistance capabilities in the plastic areas of adjacent beams, corrected by the design at the front of the pillar;

t_{fb} the thickness of the sole of the beam;

h_b the total height of the section of the beam;

$V_{\wp,Rd}$ design value of the shear force resistance capacity of the pillar core panel, determined according to SR EN 1993-1-8, which shall be limited according to the condition:

$$V_{\wp,Rd} \leq V_{wb,Rd} \qquad (6.4)$$

$V_{wb,Rd}$ design value of the deformation capacity of the tangentially stressed core panel determined in accordance with SR EN 1993-1-8.

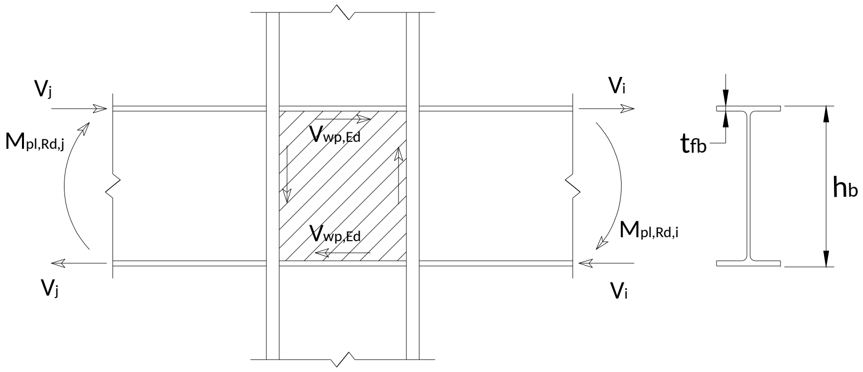


Figure 6.2 Beam-pillar joint: the core panel of the pillar.

(823) Total thickness of core panel boards t shall be chosen in such a way that the condition is met:

$$t=(t_w+t_{wsp})\geq\frac{[(h_b-2t_{bf})+d_c-2t_{c,f}]}{90} \tag{6.1}$$

where:

t_w the thickness of the core of the pillar (rolled profile);

t_{wsp} the thickness of the sheet doubling the core of the beam-pillar joint;

h_b the total height of the section of the beam;

t_{bf} the thickness of the sole of the beam;

d_c the width of the section of the pillar;

$t_{c,f}$ thickness of the pillar flange.

(824) If the condition (6.1) is not fulfilled, the core panel shall be further stiffened with stiffeners of thickness greater than or equal to $0{,}80\,t_w$, where t_w is the thickness of the core of the pillar

(825) When the beam-pillar joint is made by direct welding of the beam soles to the pillar soles, continuity stiffening shall be arranged on the core of the pillar in front of the beam soles. These stiffeners shall be made with sheet metal with a thickness greater than or equal to the thickness of the sole of the beam.

(826) The continuous reinforcements shall be attached to the pillar soles by deep penetration welding or raised welding on both sides. Welded joints shall be dimensioned so as to have a strength capacity greater than or equal to the minimum of:

- the strength of the continuous reinforcements;
- the maximum stress in the beam flange.

(827) The grips of the continuity stiffeners to the core of the pillar shall be dimensioned so as to have a resistance capacity greater than or equal to:

- the strength of the continuous reinforcements;
- the actual effort that is transmitted by the stiffening.

(828) In the area of the beam-pillar joint, the pillar soles shall be connected laterally, at the level of the upper sole of the beams. Each side support shall be projected at a force equal to $0{,}02 f_y t_{bf} b$, where t_{bf} *and* b are the dimensions of the beam flange.

(829) If the structure is designed to form plastic zones in beams, according to the optimal plastic mechanism established according to 6.2.5, the joints of the beams with the pillars shall be made in such a way as to respond in the elastic domain to the incidence of the design seismic action, corresponding to the ultimate limit state. The design values of the efforts shall be determined according to the plastic resistance capacity of the beam $M_{pl,Rd}$ and the shear force associated with the formation of plastic zones in the beam, V_{Ed}, assessed according to 6.6.2, (802).

(830) Total spinning capacity of the pillar-beam joint θ_{Rd}^{SLU} at the last limit state must be at least equal to $0{,}04\,rad$, for structures of ductility class DCH, respectively $0{,}03\,rad$ for those in class DCM.

Total spinning capacity θ shall be secured at cyclic loads without degradations of strength and rigidity of more than 20 %. This requirement is valid regardless of the location of the dissipative zones considered in the design.

$$\theta = \frac{\delta}{0{,}5L} \qquad (6.2)$$

where δ and L are the beam arrow in the middle of the opening and the beam opening (Figure 6.6).

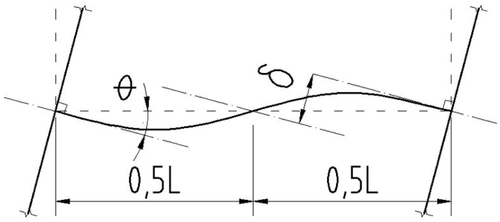


Figure 6.3 The significance of the deformation δ that is taken into consideration for calculation of the rotation θ

6.6.5 Fastening of pillars in foundations

(831) In the structures of buildings designed in the ductility classes DCM and DCH, plastic deformations can develop in the vicinity of the pillar base; the potentially plastic area near the base of the pillar shall provide plastic rotations compatible with the overall deformations, but at least 0.04 rad.

(832) The clamping of the foundation pillar shall be carried out in such a way as to ensure the transmission of the shear forces from both main directions of the section calculated with the relation (6.37):

$$V_{Ed,j} = \frac{2 M_{N,Rd,c}}{h_s} \qquad (6.1)$$

where:

$V_{Ed,j}$ the design value of the shear force in the grip;

$M_{N,Rd,c}$ design value of the plastic resistance capacity at the plastic bending moment, in the presence of axial force;

h_s level height.

6.7 Frames with centric wind bracing

6.7.1 Design criteria

(833) The centrically braced frames shall be designed in such a way that the plastic deformation of the stretched diagonals occurs before the joints failure, the formation of plastic zones in beams and pillars or the loss of the overall stability of the beams and pillars.

(834) The bracing diagonals shall be positioned in such a way that the structure has lateral displacements with close values, at each level and in any braced direction, for both directions of the seismic movement.

(835) At each level, the condition shall be met:

¿¿ (6.1)

where:

$A^{+¿¿}$ and $A^{-¿¿}$the areas of the vertical projections of the cross-sections of the stretched diagonals, when the horizontal seismic action has different meanings (Figure 6.7).

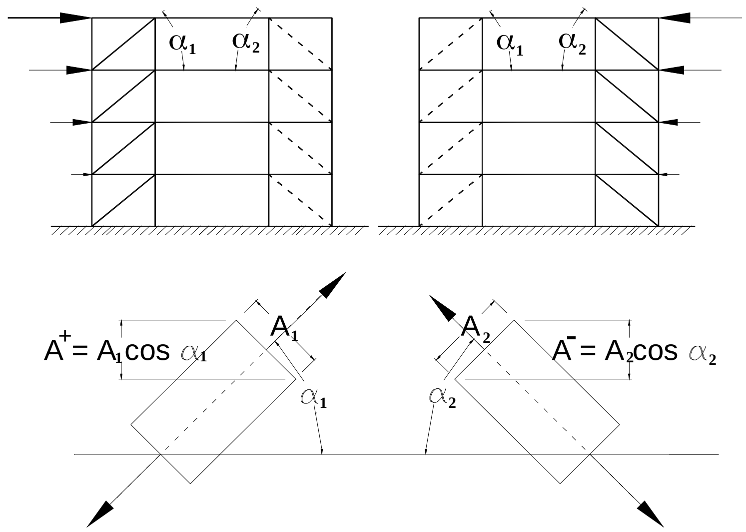


Figure 6.2 An illustrative representation of the provisions of 6.7.2. (3)

(836) The eccentric clamping of the bracing against the beam-pillar intersection joint in relation to one of the axes shall be limited to the height of the beam section at most. This shall be taken into account in the calculation of the structure.

(837) It is not allowed to use cables for diagonal elements calculated in the concept of high, medium or poorly dissipative behaviour.

6.7.2 Calculation particulars

(838) The structure shall be such that its strength is sufficient for all structural components and their joints to respond elastically under actions corresponding to the ultimate limit state other than seismic action.

(839) In the case of buildings designed for the ductility class DCH or DCM, the structural system shall be constructed in such a way as to meet the conditions of resistance to the ultimate limit state, corresponding to the fundamental grouping of loads, without the input of bracings.

(840) When performing the calculation of the structure by a linear static calculation method, it is considered that:

(ffffffffff) for structures in frames with 'X' or alternating bracings, where stretched and compressed diagonals do not intersect, (see Figure 6.7), classified as DCM ductility, only extended diagonals may be taken into account; compressed diagonals can be neglected only if the compressive strength of the bar is not more than half of its tensile strength. Centric braced frames, for which only stretched diagonals can be taken into account when designing in the ductility class DCM, are given in Table 6.3, (h), (i).

(gggggggggg) for two-tier 'V' or 'X' braced frame structures classified as DCM ductility, both stretched and compressed diagonals shall be taken into account.

(hhhhhhhhhh) for centrically braced structures of the DCH class of ductility, both stretched and compressed diagonals shall be taken into account.

(841) Calculation of the structure for centrically braced frame-type structural systems with continuous bracings developed over more than two openings and levels, not covered by Table 6.3, (h), shall be performed by non-linear static calculation or non-linear dynamic calculation.

(842) When calculating the structure with any type of centre bracings, for buildings designed for the ductility class DCH or DCM, stretched and compressed diagonals may be taken into account if the following conditions are met:

(iiiiiiiiii) a non-linear static or dynamic calculation is performed;

(jjjjjjjjjj) the cross-bars are digitised using finite elements which model the buckling of the compressed cross-bars. ( )

6.7.3 Calculation of the diagonals

(843) The cross-sections of diagonals shall be classified as class 1 sections for design in the DCH ductility class and as class 1 or 2 sections for design in the DCM ductility class, in accordance with the provisions of SR EN 1993-1-1. In the case of design in the DCH class of ductility, the conditions shall additionally be complied with:

(kkkkkkkkkk) for tubular diagonals with circular sections:

$$D/t \leq 47{,}4\,\varepsilon^2/\omega_{rm} \tag{6.1}$$

where:

D the outer diameter of the diagonal;

t the thickness of the diagonal cross-sectional wall;

$$\varepsilon = \left(\frac{235}{f_y}\right)^{0,5} \tag{6.2}$$

(llllllllll) for hollow diagonals with rectangular or square sections:

$$c/t \leq 19{,}4\,\varepsilon/\omega_{rm}{}^{0,5} \tag{6.3}$$

where:

c the width of the diagonal cross-sectional wall;

t the thickness of the diagonal cross-sectional wall.

(844) Diagonal length shall be taken to be equal to the interax length of the bar:

Note: This is the distance between the theoretical joints; the joints are considered to be the intersections of the diagonal axis with the axes of the pillars or beams.

(845) The buckling length of the diagonal shall be taken as:

(mmmmmmmmmmm) equal to the length of the bar perpendicular to the plane of the braced frame;

(nnnnnnnnnn) equal to 0.8 of the length of the bar in the plane of the braced frame;

(oooooooooo) equal to 0.8 of the length of the bar perpendicular to the plane of the braced frame in the case of diagonals in X;

(pppppppppp) equal to 0.5 of the length of the bar in the plane of the braced frame in the case of diagonals in X;

(846) By way of exception from (845), in the case of data obtained from experimental studies or tests, other values for buckling lengths may be adopted.

(847) In centrically braced frames with 'X' diagonal bracings, in buildings designed for ductility class DCM, where only stretched bars are taken into account, slenderness shall be limited according to the condition:

$$1{,}3\pi\sqrt{\frac{E}{f_y}} \leq \lambda \leq 2{,}5\pi\sqrt{\frac{E}{f_y}} \tag{6.4}$$

(848) In centrically braced frames with X-diagonal bracings, where both stretched and compressed bars are taken into account, slenderness shall be limited under the following conditions:

$$\begin{aligned} &\lambda \leq 2{,}0\pi\sqrt{\frac{E}{f_y}} \text{ for ductility class DCH} \\ &\lambda \leq 2{,}5\pi\sqrt{\frac{E}{f_y}} \text{ for ductility class DCM} \end{aligned} \tag{6.5}$$

(849) For centrically braced frames with bracings working at the stretch but not arranged in 'X', as depicted in Table 6.3, (h), (i) and Figure 6.7, slender *se* shall be limited according to the relation:

$$\lambda \leq 2{,}0\pi\sqrt{\frac{E}{f_y}} \tag{6.6}$$

(850) In centrically braced frames with inverted 'V' and 'V' bracings, slenderness shall be limited according to the relation:

$$\lambda \leq 2{,}0\pi\sqrt{\frac{E}{f_y}} \tag{6.7}$$

(851) On centrically braced frames with not more than two levels, no additional limitation for slenderness shall apply regardless of the arrangement of the bracings.

(852) Stretched diagonals shall be made in such a way as to fulfil the relation:

$$N_{pl,Rd} \geq N_{Ed} \tag{6.8}$$

where:

N_{Ed} the design value of the diagonal axial tensile force in the seismic grouping of loads;

$N_{pl,Rd}$ design value of the plastic tensile strength of the gross cross-section of the diagonal, determined in accordance with SR EN 1993-1-1.

(853) The strength of the compressed diagonals shall be checked on the basis of the provisions of SR EN 1993-1-1.

(854) For centric braced frames Table 6.3, (h), (i) or (iii), diagonals over the length of which areas of reduced cross-section are provided may be used, provided that all conditions are met:

(qqqqqqqqqqq) low section areas shall be located towards the ends of the bar so as not to impair the buckling strength of the diagonal;

(rrrrrrrrrrr) the length of the area with the reduced diagonal section, L_r, fulfils the condition:

$$L_r \geq 2c + 5{,}65\sqrt{t_f\left(b_{bf} - 2c\right)} \tag{6.9}$$

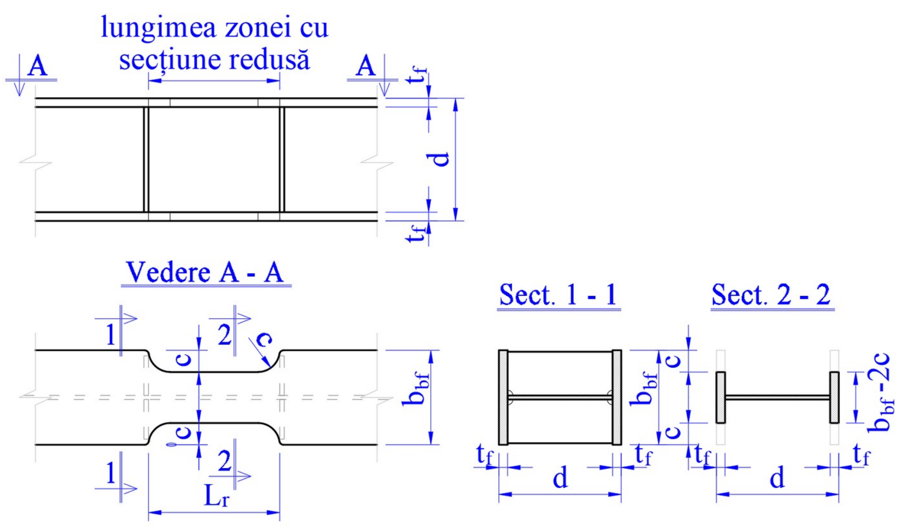
where:

L_r the length of the area with the reduced diagonal section;

A_{rs} sectional area of the reduced area $A_{rs} = t_f\left(b_{bf} - 2c\right)$;

c maximum reduction of the sole of the bar, upper limited to $0{,}25\,b_{bf}$;

t_f, b_{bf}, c shall be established in accordance with the representation of Figure 6.2.



Lungimea zonei cu secțiune redusă	Length of reduced section area

Vedere A - A	View A - A

Figure 6.2 Geometry of the reduced section area located at the ends of the bar

(sssssssssss) the reduced area shall be made with a class 1 section;

(tttttttttt) the cutting of the soles shall be carried out by a procedure which ensures a smooth cut so that no indentations or other defects appear that constitute cracked numbness; the roughness of the cut-out area shall be less than or equal to 13 µm, the connections between the cut-out and the unaltered sole shall be rounded and the edges of the soles in the cut-out areas shall be polished;

(uuuuuuuuuuu) the axial deflection capability of low-section areas shall be greater than the deflection requirement corresponding to the movement of the structure under design seismic action, corresponding to the ultimate limit state, determined by non-linear calculation.

6.7.4 Calculation of the beams and pillars

(855) The resistance capacity of the pillars and beams shall be checked in each seismic combination of loads. Strength and stability checks shall be carried out on the basis of the provisions of SR EN1993-1-1. The design values of the efforts shall be determined with the relations:

$$N_{Ed}=N_{Ed,G}+\Omega_T N_{Ed,E} \tag{6.1}$$

$$M_{Ed}=M_{Ed,G}+\Omega_T M_{Ed,E} \tag{6.2}$$

$$V_{Ed}=V_{Ed,G}+\Omega_T V_{Ed,E} \tag{6.3}$$

where:

N_{Ed}, M_{Ed}, V_{Ed} design values of axial force, bending moment and shear force;

$N_{Ed,G}$, $M_{Ed,G}$, $V_{Ed,G}$ axial force, bending moment and shear force, from non-seismic actions in the load grouping including seismic action;

$N_{Ed,E}$, $M_{Ed,E}$, $V_{Ed,E}$ axial force, bending momentum and shear force, from design seismic action;

Ω_T the value of the structural system overstrength, which shall be determined with respect to the relation:

$$\Omega_T=\omega_{rm}\omega_{sh}\Omega_d \tag{6.4}$$

Ω_d is the minimum value of the ratio $\Omega_{N,i}=N_{pl,Rd,i}/N'_{Ed,i}$ calculated for the extended diagonals of the wind bracing system of the frame.

$N_{pl,Rd,i}$ design value of the axial force plastic resistance capacity of the diagonal '*i*';

$N'_{Ed,i}$ the value of the axial force in the diagonal '*i*', resulting from the calculation of the structure in the grouping of actions that includes seismic action.

The value of Ω_d shall be calculated for each seismic design combination.

Note: Values of efforts N, $_{Ed}M_{Ed}$, V_{Ed} can be obtained from seismic groups, where the one-way seismic action is multiplied by Ω_T

(856) The value of overstrength Ω may be limited upwards so that the condition is met $\Omega_T \leq q$, where q is the behaviour factor of the structure. When designing buildings classified as class III or IV of importance and exposure to earthquakes, the overstrength values Ω_T may be used, as provided for in Table 6.5.

(857) For each horizontal orthogonal direction of the structure, the diagonals shall be such that the difference between the maximum value and the minimum value of the ratio $\Omega_{N.i}$, determined in accordance with (1), be less than or equal to 25 % of the maximum value.

(858) By way of exception from (3), where a difference between the maximum value and the minimum value of the ratio cannot be ensured $\Omega_{N.i}$, determined in accordance with (1), less than or equal to 25 % of the maximum value, the plastic mechanism of the structure shall be checked by non-linear, static or dynamic calculation.

(859) In frames with "V" or similar bracings, the beams shall be designed to take over the efforts produced by the seismic action applied to the beam by the bracings after the buckling of the compressed diagonal. These efforts shall be calculated by considering:

- a tensile force equal to $\omega_{rm}\omega_{sh}N_{pl,Rd}$ diagonally extended, where $N_{pl,Rd}$ is the design value of the plastic tensile strength of the diagonal;
- a compressive force equal to $0{,}3\,N_{b,Rd}$ in the compressed diagonal, where $N_{b,Rd}$ is the strength of the compressed diagonal determined in accordance with the provisions of SR EN 1993-1-1.

(860) In frames with ‘V’ or similar bracings, at the cross-section with the diagonals, beam side links shall be provided at both the upper and lower soles capable of balancing a lateral force equal to $0{,}02\,bt_f f_y$.

(861) In the frames with alternating diagonals (Figure 6.7), the pillars shall be designed considering the axial forces that develop in them when achieving the buckling resistance capacity of the diagonals, taking into account the resistance deviations of the material, $\omega_{rm}N_{b,Rd}$.

(862) The slenderness of the pillars within the wind braced plane shall be limited to

$$\lambda \leq 1{,}3\pi\sqrt{\frac{E}{f_y}} \tag{6.5}$$

where

λ slenderness of the pillar;

E modulus of elasticity of steel;

f_y design value of the steel flow limit.

(863) Continuity joints of the pillars shall be made at about 1/3 of the height of the floor and shall be designed in accordance with the provisions of SR EN 1993-1-8.

(864) In the case of centrically braced frames with two-level ‘V’ or ‘X’ bracings designed for the ductility class DCM or DCH, the beams of the braced frame shall be such that their bending rigidity, k_b, comply with the conditions:

$$k_b \leq 0{,}2 \cdot k_{br} \tag{6.6}$$

$$k_b = (48 \cdot \zeta \cdot E_s \cdot I_b / L_b^3) \qquad (6.7)$$

$$k_{br} = 2 \frac{A_{br} E_s}{L_{br}} \sin^2 \alpha \qquad (6.8)$$

where:

k_b bending rigidity of the beam;

k_{br} rigidity of diagonals intersecting the beam;

E_s modulus of elasticity of steel;

I_b moment of inertia of the beam section in relation to the axis of bending;

L_b beam length (opening);

ζ factor taking into account the connections at the ends of the beam: $\zeta = 1$, for the hinged bar, or $\zeta = 4$,for the embedded bar;

L_{br} length of diagonals;

A_{br} diagonal cross-sectional area;

α the angle that the diagonal bar makes with the horizontal direction.

(865) In the case of centrically braced frames with rigid beam-pillar mountings, when designing the pillars and beams, the condition regarding the hierarchy of the resistance capacities at the bending moment given at 6.6.3, (816) shall be verified.

Note: In order to comply with this provision, constructive measures may be applied to ensure against the occurrence of plastic deformations in pillars. Such measures are, for example: reduced sections at the ends of the collar beams in the case of rigid fastenings between the collar beams and the pillars.

6.7.5 Beam-pillar joints

(866) In the braced opening, the joint can be made articulated or rigid.

(867) When articulated, the joint shall allow the development of a free spin of 0.02 rad.

(868) When rigid, the joint must be made as a non-dissociative joint with the fulfilment of the requirements (869) and (870).

(869) In non-dissipative joints, the bending strength of the beam-pillar attachment shall fulfil the condition:

$$M_{j,Rd} \geq \omega_{rm} \omega_{sh} M_{pl,Rd} \qquad (6.1)$$

where:

$M_{j,Rd}$ the design value of the bending current joint strength determined in accordance with the provisions of SR EN 1993-8;

$M_{pl,Rd}$ the design value of the beam's currently bending plastic strength determined in accordance with the provisions of SR EN 1993-1.

(870) In non-dissipative joints, if the beam at the last level is positioned above the pillar, the pillar-beam joint complies with the following condition:

$$M_{j,Rd} \geq \omega_{rm} \omega_{sh} M_{N,Rd} \quad (6.2)$$

where:

$M_{N,Rd}$ capable moment of the pillar, in the presence of axial force.

6.7.6 Joints of bracing bars

(871) The clamping system of the bracing bars shall have the strength to take over the axial forces developed in the outstretched diagonals entering the flow, $N_{pl,Rd}$, and taking into account overstrength, ω_{rm}, and consolidation of the material , ω_{sh}.

(872) The clamping system of the bracing bars shall be checked at the efforts given by the relations (6.1), (6.2) and (6.3):

$$N_{T,j,Ed} = \omega_{rm} \omega_{sh} N_{pl,Rd} \quad (6.1)$$

$$N_{C,j,Ed} = \omega_{rm} N_{b,Rd} \quad (6.2)$$

$$M_{j,Ed} = \omega_{rm} \omega_{sh} M_{pl,Rd} \quad (6.3)$$

where:

$N_{T,j,Ed}$, $N_{C,j,Ed}$, $M_{j,Ed}$ design efforts, i.e. the axial tensile force ($N_{T,j,Ed}$), axial compression force ($N_{C,j,Ed}$) and the bending moment in the plane of the braced frame ($M_{j,Ed}$), used for sizing joints;

$N_{pl,Rd}$ the capacity of plastic resistance to axial strain of bracing;

$N_{b,Rd}$ the buckling strength of the diagonal;

$M_{pl,Rd}$ the plastic capable moment of the bracing section evaluated in the plane of the bracing frame.

6.8 Frames with eccentric wind bracing

6.8.1 Design criteria

(873) Frames with an eccentric wind bracing system must be designed so that the dissipative bars, which are special elements installed within the structure, are able to dissipate energy by forming plastic bending and/or shearing mechanisms.

(874) The structure shall be designed in such a way as to achieve an overall dissipative behaviour that is as uniform as possible.

(875) The rules given below are intended to ensure that the formation of plastic joints in the dissipative bars will occur before the loss of overall stability or the appearance of plastic joints in other structural elements such as pillars, bracings and/or beam segments adjacent to the dissipative bars.

(876) Dissipative bars can be horizontal or vertical.

6.8.2 Calculation of the dissipative bars

(877) The dissipative bars shall be made with double "T" sections made of hot-rolled profiles, "I" and/or "H", or welded sheets.

(878) The dissipative bar shall be made with a full core, without doubling plates, voids or holes.

(879) Dissipative bars are classified into 3 categories depending on how the plastic mechanism is developed:

- short dissipative bars, which consume energy predominantly by deformation in the post-elastic field of the bar from the shear force (main efforts);
- long dissipative bars, which consume energy predominantly by deformation of the bending moment section in the post-elastic field;
- intermediate dissipative bars, where the cross-section is plasticised due to a bending moment and the shear force.

(880) The strength of the dissipative bars shall be calculated with the relations (6.1) and (6.1):

$$M_{p,link} = f_y b t_f (d - t_f) \tag{6.1}$$

$$V_{p,link} = (f_y / \sqrt{3}) t_w (d - t_f) \tag{6.2}$$

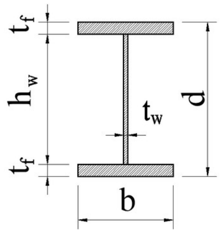


Figure 6.2 Dissipative bar notations

(881) If $N_{Ed}/N_{pl,Rd} \leq 0{,}15$ at both ends of the dissipative bar, the following requirements shall be complied with:

$$V_{Ed} \leq V_{pl,link} \tag{6.1}$$

$$M_{Ed} \leq M_{pl,link} \tag{6.2}$$

where,

M_{Ed}, M_{Ed}, V_{Ed} are the design efforts (axial force, bending moment and shear force) at both ends of the dissipative bar, resulting from the load grouping including seismic action

(882) If $N_{Ed}/N_{pl,Rd} > 0{,}15$, in relations (6.1) and (6.2) reduced values are used $V_{pl,link,r}$ and $M_{pl,link,r}$ instead of values $V_{pl,link}$ and $M_{pl,link}$:

$$V_{pl,link,r} = V_{pl,link} \left[1 - (N_{Ed}/N_{pl,Rd})^2 \right]^{0,5} \tag{6.3}$$

$$M_{pl,link,r} = 1{,}18\, M_{pl,link} \left[1 - (N_{Ed}/N_{pl,Rd}) \right] \tag{6.4}$$

(883) If $N_{Ed}/N_{pl,Rd} \geq 0{,}15$ the length of the dissipative bar, *e*, must satisfy the relation (6.5) if $R < 0{,}3$ and the relation (6.6) if $R \geq 0{,}3$:

$$e \leq 1{,}60\, M_{pl,link}/V_{pl,link} \tag{6.5}$$

$$e \leq (1{,}15 - 0{,}5\, R) 1{,}60\, M_{pl,link}/V_{pl,link} \tag{6.6}$$

Coefficient *R* has the expression:

$$R = N_{Ed} t_w \frac{d - 2t_f}{V_{Ed} A} \quad (6.7)$$

where A is the area of the rough section of the dissipative bar.

(884) If, at both ends of the dissipative bar, almost equal bending moments develop simultaneously (Figure 6.10, a), dissipative bars, regardless of the type of cross-section, will be classified according to the length *e*, as follows:

(vvvvvvvvvvv) if $e < 1{,}60\, M_{pl,link} / V_{pl,link}$, the dissipative bar is short;

(wwwwwwwwwww) if $e > 3{,}0\, M_{pl,link} / V_{pl,link}$, the dissipative bar is long;

(xxxxxxxxxxx) if $3{,}0\, M_{pl,link} / V_{pl,link} \geq e \geq 1{,}60\, M_{pl,link} / V_{pl,link}$, the dissipative bar is intermediate.

(885) When bending moments at the ends of the dissipative bar differ (Figure 6.10, b, c and d), dissipative bars, regardless of the type of cross-section, will be classified according to the length *e*, as follows :

(yyyyyyyyyyy) if $e < 0{,}80(1+\alpha) M_{pl,link} / V_{pl,link}$, the dissipative bar is short;

(zzzzzzzzzzz) if $e > 1{,}50(1+\alpha) M_{pl,link} / V_{pl,link}$, the dissipative bar is long;

(aaaaaaaaaaa) if $1{,}50(1+\alpha) M_{pl,link} / V_{pl,link} \geq e \geq 0{,}80(1+\alpha) M_{pl,link} / V_{pl,link}$, the dissipative bar is intermediate.

$$\alpha = \frac{|M_{Ed,A}|}{|M_{Ed,B}|} \quad (6.8)$$

where $M_{Ed,A}$ and $M_{Ed,B}$ are the design values of the bending moments at the ends of the dissipative bar, in the load grouping including seismic action, with $|M_{Ed,A}| < |M_{Ed,B}|$.

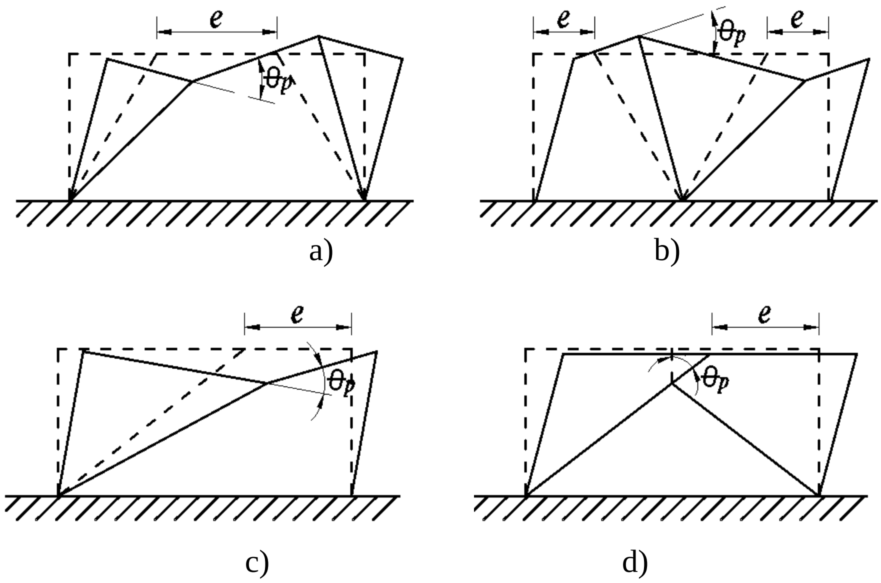


Figure 6.3 Eccentric braced frame configurations with equal moments at the ends of the dissipative bar (a) or unequal moments at the ends of the dissipative bar (b, c and d), illustrative representations

(886) Inelastic rotation angle of the dissipative bar, $\theta\, p$, between the dissipative bar and the element outside it, according to the meaning of the Figure 6.10, resulting from a non-linear calculation, shall be limited according to the relations:

(bbbbbbbbbbbb) $\theta_p \leq 0{,}08\, rad$, for short dissipative bars;

(cccccccccccc) $\theta_p \leq 0{,}02\, rad$, for long dissipative bars;

(dddddddddddd) θ_p will have a value determined by linear interpolation of the above values for intermediate dissipative bars.

(887) At the ends of the dissipative bar, next to the bracing, there will be pairs of stiffeners along the entire height of the core on both sides of it, at the distance of p from each other. These stiffeners shall have a summed width of at least $(b-2t_w)$, thickness $t_{st} \geq t_w$ or $t_{st} \geq t_{fd}$, when the soles of the diagonal thickness t_{fd} are in contact with the frame collar beam, but at least 10 mm. Where the soles of the diagonal are in contact with the frame collar beam, the weld to attach these stiffeners to the soles of the beam shall be made with the same type of weld and at least the same dimensions as those of the weld between the soles of the diagonal of the beam.

(888) Double-section ‘T’ dissipative bars shall be provided with stiffeners over the entire height of the core as follows:

(eeeeeeeeeeee) short dissipative bars (Figure 6.11), shall be provided with intermediate stiffeners located on the core at distances a which must comply with the conditions:

$$a \leq (30\, t_w - h_w/5) \text{ for } \theta_p = 0{,}08 \text{ rad}$$
$$a \leq (52\, t_w - h_w/5) \text{ for } \theta_p = 0{,}02 \text{ rad} \tag{6.1}$$

For $0{,}02\, rad < ¿\, 0{,}08\, rad$, value a shall be determined by linear interpolation.

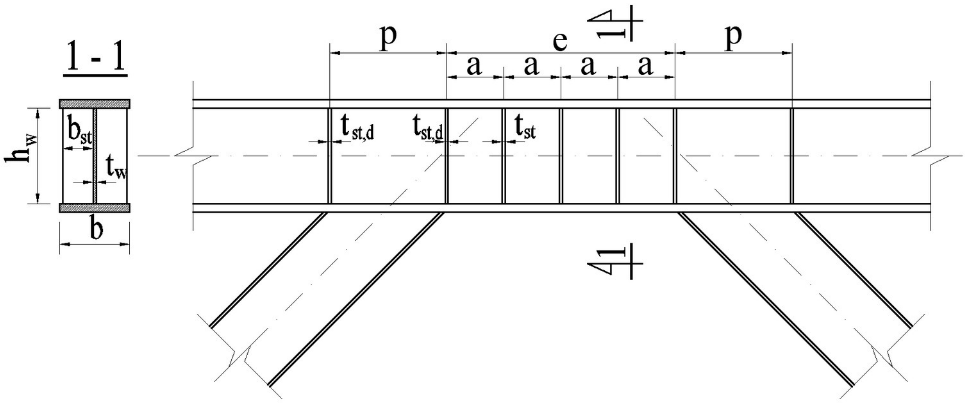


Figure 6.4 Arrangement of transverse stiffeners to short dissipative bars

(ffffffffffff) long dissipative bars (Figure 6.12) must be provided with intermediate stiffeners on both sides of the core, located at a distance of $\bar{c} \approx 1.5\, b$ from each end of the dissipative bar, which delimits the potentially plastic areas.

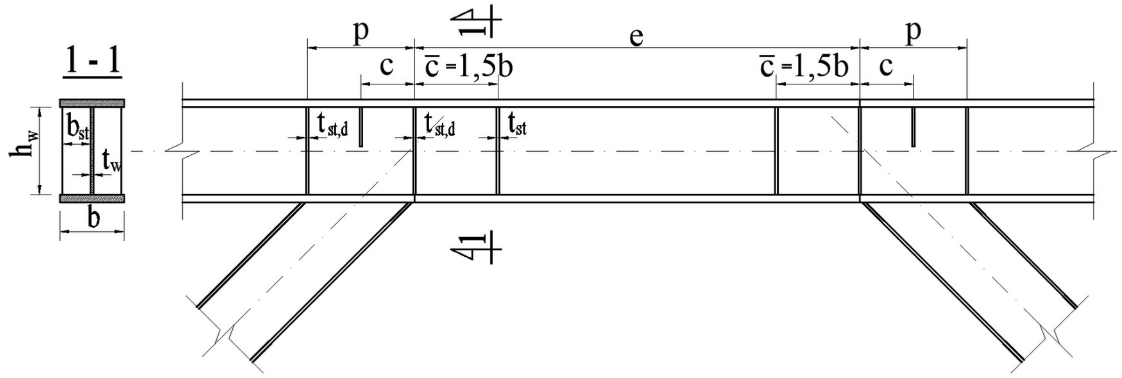


Figure 6.5 Arrangement of transverse stiffeners to long dissipative bars

(ggggggggggggg) Intermediate dissipative bars (Figure 6.13.), must be provided with stiffeners of the core that meet the requirements of a) and b) listed above.

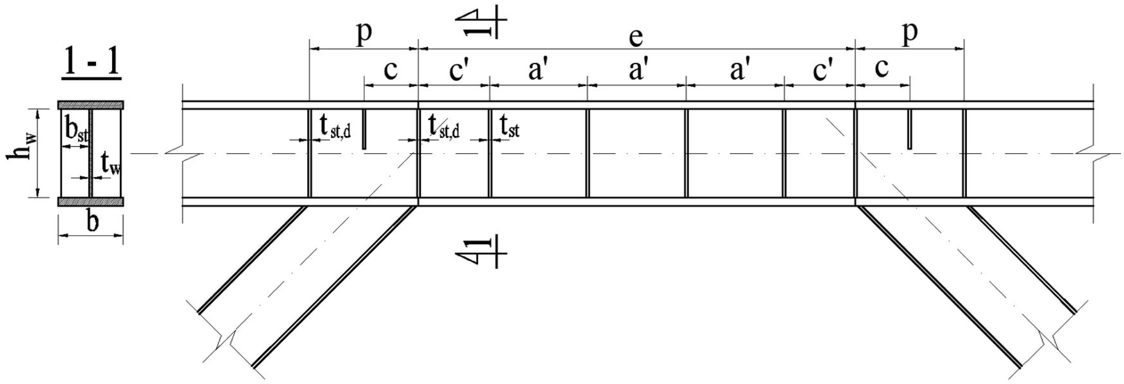


Figure 6.6 Arrangement of transverse stiffeners to intermediate dissipative bars

(hhhhhhhhhhhh) The rigidity of the core must be foreseen throughout its height. These stiffeners shall have a summed width of at least $(b-2t_w)$ and thickness $t_{st} \geq 0{,}8t_w$, but at least 10 mm.

(iiiiiiiiiiii) Alternatively, for dissipative bars with a height of less than 600 mm, the reinforcements can be installed on one side of the core only. The distance a will be the distance between two consecutive stiffeners, no matter on which side of the core of the dissipative bar they are located. In this case, the thickness t_{st} of the stiffening shall fulfil the conditions $t_{st} \geq t_w$ and $t_{st} \geq 10\,mm$, and the width of the stiffener shall fulfil the condition $b_{st} \geq (b-2t_w)/2$.

(jjjjjjjjjjjj) To avoid loss of stability of the upper sole of the collar beam in the diagonal intersection panel, transverse stiffeners shall be provided, with a height of at least $h_w/2$, at a distance c from the end of the dissipative bar, where c shall be determined with the relation:

$$c = minim(b, p/2) \quad (6.1)$$

where c, b and p are described in Figure 6.11, Figure 6.12 and Figure 6.13.

(889) No intermediate stiffeners are required on the core of dissipative bars longer than $5M_{pl,link}/V_{pl,link}$.

(890) Stiffeners of the dissipative bars provided according to (12) shall be fastened with full penetration deep weld or embossed weld on both sides of the stiffening. The

embossed welds of the stiffeners at the core of the dissipative bar shall have a strength greater than or equal to $\omega_{rm} f_y A_{st}$, where $A_{st}=t_{st} b_{st}$ is the area of the stiffening section. Embossed weld strength between stiffening and dissipative bar soles shall be greater than or equal to $\omega_{rm} f_y A_{st}/4$.

(891) At the ends of the dissipative bar, at both the upper and lower soles, side ties shall be provided, having a compressive strength equal to or greater than $0{,}06\,\omega_{rm} f_y b t_f$, where b and t_f are the dimensions of the section of the sole of the dissipative bar.

(892) For directing plastic joints in predominantly bent dissipative bars $e \geq 2{,}60\,M_{pl,link}/V_{pl,link}$, the width of the soles may be reduced in the vicinity of the ends of the dissipation bar, respecting the constructive conditions indicated in 6.6.2(11). The reduced section shall be verified at the ultimate limit state for the design stresses belonging to the group of loads which includes the seismic action.

6.8.3 Non-dissipative elements

(893) Non-dissipative elements: pillars, bracings and beam segments outside the dissipation bars shall be checked considering the worst case scenario of efforts. For strength and stability checks, SR EN 1993-1-1 shall be used as the reference normative document. Calculation efforts in the design seismic situation shall be determined with the relations:

$$N_{Ed}=N_{Ed,G}+\Omega_T N_{Ed,E} \tag{6.1}$$

$$M_{Ed}=M_{Ed,G}+\Omega_T M_{Ed,E} \tag{6.2}$$

$$V_{Ed}=V_{Ed,G}+\Omega_T V_{Ed,E} \tag{6.3}$$

where:

$N_{Ed,G}$, $M_{Ed,G}$, $V_{Ed,G}$ design values of axial force, bending moment and shear force, from non-seismic actions in the load grouping including seismic action;

$N_{Ed,E}$, $M_{Ed,E}$, $V_{Ed,E}$ design values of the axial force, bending moment and shear force, from the design seismic action;

Ω_T value of structural system overstrength:

$$\Omega_T=\Omega_d\,\omega_{rm}\,\omega_{sh} \tag{6.4}$$

Ω_d the overstrength factor, which shall be determined separately for each direction of seismic action, with the relations:

$\Omega_d = minin¿$ for short and intermediate dissipative bars with $e \leq 2{,}6\,M_{pl,link}/V_{pl,link}$;

$\Omega_d = minim\left(M_{pl,link,i}/M_{Ed,i}\right)$ for long and intermediate dissipative bars with $e > 2{,}6\,M_{pl,link}/V_{pl,link}$; (6.5)

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, according to the provisions of Table 6.2;

ω_{sh} the overstrength factor taking into account the strengthening effect of the steel, the value of which shall be determined in accordance with the provisions of Table 6.4;

$M_{Ed,i}, V_{Ed,i}$ design values of shear force and bending moment in the dissipative bar '*i*', in the load grouping which includes seismic action;

$M_{pl,link,i}, V_{pl,link,i}$, design values of the plastic resistance capacity at the bending moment and shear force, in the dissipative bar "i" according to 6.8.2 (4).

(894) Value of overstrength Ω_T shall be limited upwards so that the condition is met; $\Omega_T \leq q$, where q is the behaviour factor of the structure. In the case of a simplified calculation, overstrength values may be adopted Ω_T from Table 6.5.

(895) When the dissipative bar and the beam containing it are made of a single bar, the efforts in the beam segment located outside the dissipative bar can be determined according to the relations (6.73), (6.74) and (6.75), considering $\omega_{rm}=1,00$.

(896) Difference between the maximum values and minimum ratios Ω (in each direction of the structure) must be smaller than 25 %.

If a variation of less than 25 % of the ratio Ω_i cannot be ensured, the formation of the plastic mechanism of the structure shall be checked using a non-linear static or dynamic calculation method.

Note 1: Practically, the values of efforts N_{Ed}, M_{Ed} and V_{Ed} shall be obtained from seismic groups, where the one-way seismic action is multiplied by Ω_T.

(897) The slenderness of the pillars within the wind braced plane shall be limited to

$$\lambda \leq 1{,}3\pi\sqrt{\frac{E}{f_y}} \quad (6.6)$$

(898) For checking the cores of the beams adjacent to the dissipative bar in the event of a loss of local stability, SR EN 1993-1-1 shall be used as a reference normative document.

(899) In case of inverted 'Y' bracing (Figure 6.10, *d*), constructive measures shall be applied to ensure against the occurrence of plastic deformations in the pillars.

Note: Such measures are, for example, the use of reduced sections at the ends of collar beams and diagonals attached to pillars and/or bolted joints at the ends of collar beams and diagonals of pillars.

6.8.4 Joints of the dissipative bars

(900) The joints of dissipative bars or elements containing dissipative bars shall be designed for loads:

(kkkkkkkkkkkk) for short dissipative bars characterised by $e \leq M_{pl,link}/V_{pl,link}$:

$$N_{Ed,j}=\left(2\,\omega_{rm} f_y b_f t_f\right) \quad (6.1)$$

$$M_{Ed,j}=0{,}5\,e\,\omega_{rm}\omega_{sh}V_{pl,link} \quad (6.2)$$

$$V_{Ed,j}=V_{u,link}=\omega_{rm}\omega_{sh}V_{pl,link} \quad (6.3)$$

(llllllllllll) for short and intermediate dissipative bars characterised by $M_{pl,link}/V_{pl,link} < e \le 2{,}6\, M_{pl,link}/V_{pl,link}$:

$$V_{Ed,j} = V_{u,link} = \omega_{rm}\omega_{sh} V_{pl,link} \tag{6.4}$$

$$M_{Ed,j} = 0{,}5\, e\, V_{pl,link} \tag{6.5}$$

(mmmmmmmmmmmmm) for intermediate and long dissipative bars characterised by $e > 2{,}6\, M_{pl,link}/V_{pl,link}$:

$$M_{Ed,j} = M_{u,link} = \omega_{rm}\omega_{sh} M_{pl,link} \tag{6.6}$$

$$V_{Ed,j} = 2\, M_{pl,link}/e \tag{6.7}$$

(901) For the calculation of the joint, $N_{Ed,j}$ shall always be taken into account as an axial tensile force.

6.9 Design rules for inverted pendulum structures

(902) In inverted pendulum structures, pillars shall be checked for compression and bending, taking into account the most unfavourable combination of axial stresses and bending moments.

(903) Efforts N_{Ed}, M_{Ed} and V_{Ed} shall be used for verification, calculated according to 6.6.3, relations (6.23).

(904) The slenderness coefficient of the pillars shall be limited according to the relation:

$$\lambda \le 0{,}85\pi\sqrt{\frac{E}{f_y}} \tag{6.1}$$

(905) The coefficient of sensitivity to the relative displacement θ defined in 4.5.5 (2) shall be limited upwards to 0.20.

6.10 Buckling-restrained braced frames

6.10.1 Design criteria

(906) Buckling-restrained braces are dissipative elements that are designed to develop plastic deformations. The bracing are made of a steel core inserted into a system that prevents the core from buckling.

(907) Frames with restrained buckling braces shall be designed in such a way that the plastic deformation of the restrained buckling braces occurs before the plastic joints are formed or the overall stability in the beams and pillars is lost.

(908) In the construction of the structures, bracing bars with obstructed buckling shall be used, which have technical approval. The technical approval shall specify the strength and deformability characteristics of the bracing bars with hindered buckling under cyclic stress.

6.10.2 Calculation particulars

(909) The eccentric clamping of the buckling-restrained braces against the beam-pillar intersection joint in relation to one of the axes shall be limited to no more than the height of the beam section. This shall be taken into account in the calculation of the structure.

(910) It shall be considered that gravitational loads are only absorbed by the beams and pillars, without taking into account the wind bracing elements.

(911) Reversed 'V' or 'V' restrained buckling braces shall meet the following requirements:

(nnnnnnnnnnnn) the strength of the beams intersecting the bracings, their joints and adjacent elements shall be calculated under the assumption that the diagonals do not contribute to taking over the gravitational loads. For combinations including seismic action, the effect of bracing on the beam, expressed by a vertical and a horizontal force, shall be determined on the basis of the tensile and compressive strength provided by the manufacturer of the restrained buckling bars.

(oooooooooooo) The beams shall be made continuously between the pillars. In the section of intersection with the diagonals, at the upper and lower soles of the beam there, lateral connections capable of each assuming a lateral force equal to $0{,}02\, b\, t_f f_y$ shall be provided.

(912) In the calculation of the structure, both stretched and restrained buckling braces shall be taken into account.

(913) Restrained buckling braces may be used, for which the technical approval specifies the following characteristics: N_{Rd}, γ_{CT}, ω_{sh}, ω_{rm}, A, f_y, force-movement curve under cyclic stress, maximum tensile and compressive deflection capability, plane in which bracing failure occurs,

where

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel;

ω_{sh} the overstrength factor taking into account the reinforcing effect of the steel; this is obtained from the tensile tests on test specimens obtained from the material of the restrained buckling brace core and represents the ratio of the resulting maximum tensile force corresponding to a deformation of the test specimen twice the size of that caused by the seismic action of design corresponding to the ultimate limit state and the force corresponding to the initiation of the flow of steel;

γ_{CT} the ratio of the maximum compressive strength to the maximum tensile strength determined on the basis of the results of the qualification tests for a double deformation than that caused by the design seismic action, corresponding to the ultimate limit state;

A cross-sectional area of the steel core;

f_y nominal flow limit of the steel inside the core.

(914) Only restrained buckling braces for which the conditions are met may be used for the construction of steel structures:

$$\omega_{sh} \leq 1{,}50 \tag{6.1}$$

$$1{,}10 \leq \gamma_{CT} \leq 1{,}30 \tag{6.2}$$

(915) In non-linear analyses, the inelastic response of restrained buckling braces shall be modelled taking into account: the initial rigidity of the bar, the rigidity of the material after flow and the different values of tensile strength capacities versus compression. In the case of a non-linear dynamic analysis, the hysteretic behaviour of the bar shall also be taken into account.

6.10.3 Checking restrained buckling braces

(916) Plastic resistance capacity N_{Rd} of the section of the restrained buckling braces core supplied by the manufacturer shall be greater than the maximum axial tensile or compressive effort N_{Ed} developed in the bar in the grouping of loads that includes seismic action.

$$N_{Rd} \geq N_{Ed} \tag{6.1}$$

(917) Restrained buckling braces shall have a deformation capacity corresponding to twice the relative level displacement of the structure under design seismic action, corresponding to the ultimate limit state, but not less than 0.02 of the floor height.

6.10.4 Beams and pillars

(918) The pillars and beams shall be checked as the worst combination of efforts. For strength and stability checks, SR EN 1993-1-1 shall be used as the reference normative document. Calculation efforts in the design seismic situation shall be determined with the relations:

$$N_{Ed} = N_{Ed,G} + \Omega_T N_{Ed,E} \tag{6.1}$$

$$M_{Ed} = M_{Ed,G} + \Omega_T M_{Ed,E} \tag{6.2}$$

$$V_{Ed} = V_{Ed,G} + \Omega_T V_{Ed,E} \tag{6.3}$$

where

$N_{Ed,G}$, $M_{Ed,G}$, $V_{Ed,G}$ design values of axial force, bending moment and shear force, from non-seismic actions in the load grouping including seismic action;

$N_{Ed,E}$, $M_{Ed,E}$, $V_{Ed,E}$ design values of the axial force, bending moment and shear force, from the design seismic action;

Ω_T value of structural system overstrength:

$$\Omega_T = \omega_{rm} \omega_{sh} \gamma_{CT} \Omega_d \tag{6.4}$$

ω_{rm} the overstrength factor taking into account the variation of the flow limit of the steel, according to the provisions of Table 6.2;

ω_{sh} the overstrength factor taking into account the strengthening effect of the steel, the value of which shall be determined in accordance with the provisions of Table 6.4;

γ_{CT} correction factor for tensile strength;

Ω_d minimum value of ratio $\Omega_{d,i} = N_{Rd,i} / N_{Ed,i}$ calculated for the frame bracing, for each direction of the structure;

$N_{Rd,i}$ design value of plastic tensile stress capability of restrained buckling braces core *"i"*;

$N_{Ed,i}$ design value of the axial force that develops in restrained buckling braces 'i', in the load grouping that includes seismic action.

(919) Value of overstrength Ω_T shall be limited upwards so that the condition is met; $\Omega_T \leq q$, where q is the behaviour factor of the structure. In the case of a simplified calculation, overstrength values may be adopted Ω_T from Table 6.5.

(920) The difference between the maximum and minimum values of the ratio $\Omega_{d,i}$ (in each direction of the structure) must be less than 25 %. If a variation of less than 25 % of the ratio Ω_i^N cannot be ensured, the plastic mechanism of the structure must be checked using a non-linear static or dynamic calculation.

Note Effort values N, $_{Ed}M_{Ed}$ and V_{Ed} can be obtained from seismic groups, where seismic action multiplies by Ω_T.

(921) Constructive measures shall be applied to ensure against the occurrence of plastic deformations in the pillars (e.g. reduced sections at the ends of the collar beams in the case of rigid clamps between the beam collars and the pillars).

6.10.5 Joints of the wind bracing systems

(922) The joints of the wind bracing systems shall be sized so that they do not undergo plasticisation under the action of a force corresponding to the yielding of the steel core.

(923) The clamping system of the bracing bars with restrained buckling braces shall be checked at the efforts given by the relations:

$$N_{T,j,Ed} = \omega_{rm} \omega_{sh} N_{Rd} \quad (6.1)$$

$$N_{C,j,Ed} = \omega_{rm} \omega_{sh} \gamma_{CT} N_{Rd} \quad (6.2)$$

$$M_{j,Ed} = \omega_{rm} \omega_{sh} M_{Rd} \quad (6.3)$$

where:

$N_{T,j,Ed}, N_{C,j,Ed}, M_{j,Ed}$ the design values of the axial tensile force, compression and bending moment in the plane of the braced frame in the joint;

N_{Rd} design value of plastic resistance capabilities to axial core tensioning force with restrained buckling braces

M_{Rd} the plastic capable moment of the flared section of the core, outside the sheath, assessed in the plane of the braced frame

(924) Joints designed to be articulated shall be designed to assume axial forces in the plane of the joint at least equal to $N_{T,j,Ed}$, i.e. $N_{C,j,Ed}$, suitable for restrained buckling braces and compatible with the rotation associated with the permissible relative level displacement.

(925) In order to prevent cracking in the case of buckling with restrained buckling outside the plane of the braced frame, the clamping bracket of the bar at the ends shall be designed to assume a transverse force on its plane at least equal to the value of $V_{gp,Ed}$ to be determined with the relation:

$$V_{gp,Ed}=0{,}06\,N_{C,j,Ed} \tag{6.4}$$

(926) The clamping brackets of the restrained buckling brace shall be designed and checked in such a way as to avoid loss of local and global stability outside the plane of the braced frame. The prevention of loss of stability may be achieved by constructive measures using stiffeners and/or side links.

6.10.6 Beam-pillar joints

(927) The joint may be articulated or rigid and must meet the conditions:

(pppppppppppp) When the joint is articulated, it must enable the development of a rotation equal to 0.025 rad;

(qqqqqqqqqqqq) When performed rigidly, the joint shall be performed as a non-dissociative joint according to the provisions of 6.5.5.

6.11 Frames with shear panels

6.11.1 Design criteria

(928) Shear panels are made up of a core made up of hardened or non-hardened sheet metal panels, placed on the same vertical in the gaps in the plane of the unbraced frames, having as bordering elements the pillars and the frame collar beams. (see Figure 6.16.); The sheet metal panel is connected throughout the contour to the bordering elements with screws or embossed welding.

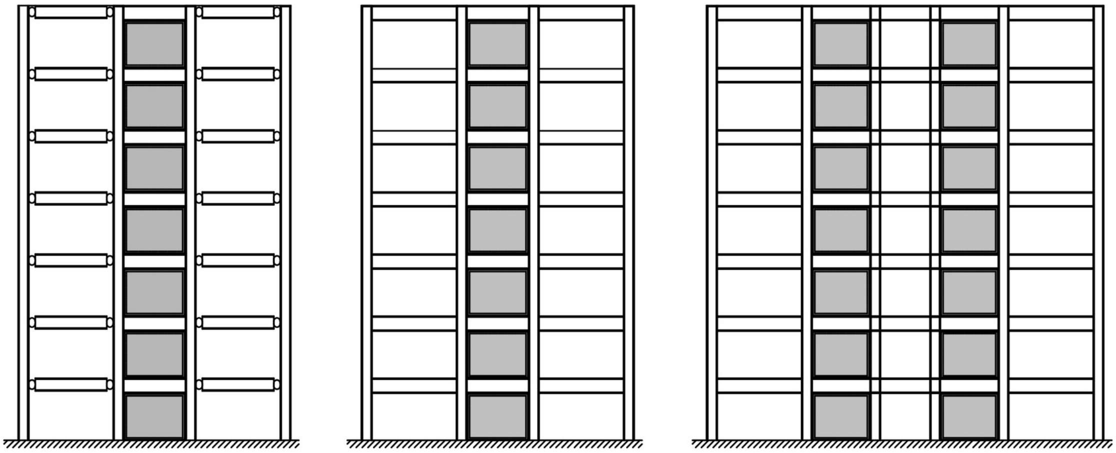
(929) Shear panel frames shall be designed in such a way that deformation in the post-elastic field of the panels occurs before the plastic joints form or the overall stability in the horizontal and vertical trim elements is lost.

(930) The following shear panel frame systems are used (Figure 6.14.):

(rrrrrrrrrrrr) single system, in which the shearing wall is conformed to take over all horizontal loads and the metal frames take over the gravitational stresses (Figure 6.14.a);

(ssssssssssss) coupled system, with coupling collar beams connecting two areas with shear panels (Figure 6.14.c).

(tttttttttttt) dual system, in which single or coupled shearing walls work together with metal frames to balance horizontal seismic action; the frames are designed to balance at least 25 % of the seismic action (Figure 6.14.b).



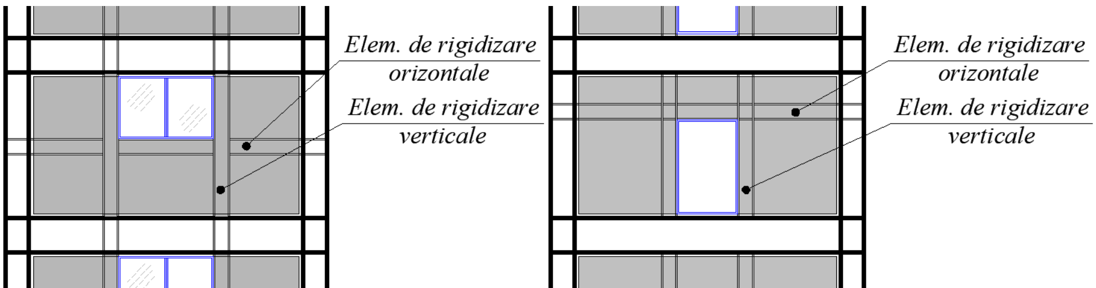
a. single system b. dual system c. dual system with coupling collar beams

Figure 6.1 Frame systems with shear panels – illustrative representation

(931) Shear panels take on the shear force produced by seismic action through the diagonal field of tension tensions. The deformation occurs in the elastic domain.

(932) Holes in the shear panels, if any, shall be provided throughout their contour with suitable plating in accordance with the provisions of SR EN 1993-1-1, SR EN 1993-1-5.

(933) Holes in the core of the shear wall panels shall be provided with rigid bordering elements (rolled or welded sheet profiles), extended horizontally and vertically over the entire width and height of the shear panel. These bordering elements are designed to remain in the elastic range under the forces generated by the development of flow efforts in the diagonal fields of tensile stress, developed in each subpanel delimited by them, with the exception of some areas at the extremities where the development of plastic joints under the action of the bending moment at the ends of the bordering elements is acceptable.



Elem. de rigidizare orizontale	Horizontal stiffeners
Elem. de rigidizare verticale	Vertical stiffeners

Figure 6.2 Gap bordering systems provided in the panel

(934) The first shear panel, at the base of the building, is provided at the bottom with a metal plating beam, which can be embedded in the foundation.

6.11.2 Calculation particulars

(935) Shear panels shall be made of S235, S275 and S355 steels. It is recommended to use low-strength steels.

(936) For design in ductility classes DCM or DCH, the structural system consisting only of the bordering elements of the sheet metal panels, pillars and beams - without

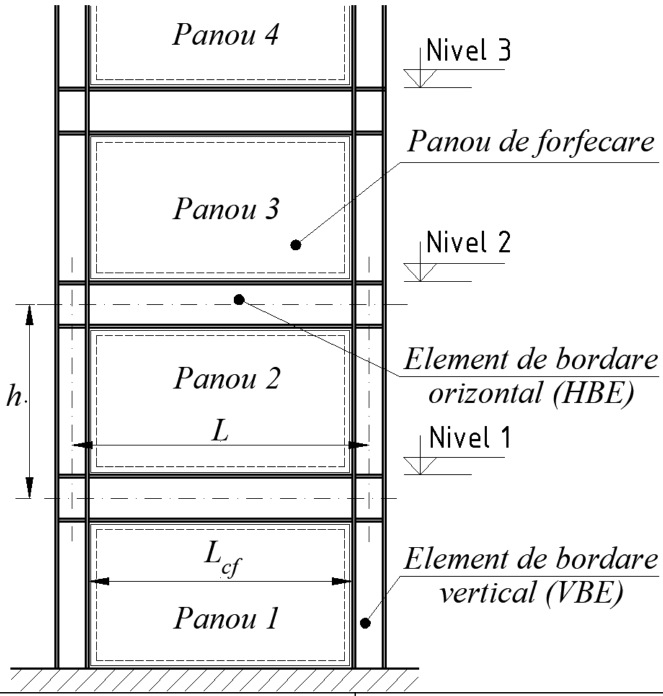
taking into account the strength of the sheets, shall be checked at the stresses produced by the gravitational loads in the fundamental grouping at ultimate limit states.

(937) The horizontal bordering elements shall be reclined in a horizontal plane at all intersections with the vertical bordering elements. Both soles of the bordering elements must be supported directly or indirectly. The capacity of the side supports shall be at least 2 % of the compressive strength of the sole ($0{,}02 f_y b_f t_f$).

(938) The ratio of the sides of the shear panel shall meet the condition:

$$0{,}8 < \frac{L}{h} \leq 2{,}5 \qquad (6.1)$$

where L and h shall be established in accordance with the representation of Figure 6.16.



Panou	Panel
Nivel	Level
Panou de forfecare	Shear panel
Element de bordare orizontal (HBE)	Horizontal Border Element (HBE)
Element de bordare vertical (VBE)	Vertical Border Element (VBE)

Figure 6.2 Basic composition of a frame with shear panels

6.11.3 Calculation of shear panels

(939) Shear panels shall be designed to balance the shear force produced by seismic action by means of the diagonal field of tensile stress.

(940) The present provisions shall apply if the design standards for metallic structures do not include other provisions.

(941) Checking the shear panels is given by the relation:

$$V_{Ed} \leq V_{Rd} \qquad (6.1)$$

where:

$$V_{Rd}=\frac{0{,}45 f_y t_w L_{cf}\sin(2\alpha)}{1{,}20} \qquad (6.2)$$

V_{Ed} the maximum design value of the shear force in the shear panel resulting from the load grouping including seismic action;

V_{Rd} the design value of the shear force resistance capability of the panel;

t_w the thickness of the sheet of the shear panel;

L_{cf} the distance between the faces of the soles of the casing posts;

f_y the design value of the flow limit of the steel used for the shear panel sheet;

α the angle of the diagonal field (angle of diagonal stretching bands) measured in relation to the vertical plane.

(942) The diagonal angle of field value α shall be determined according to the geometry of the wall and the properties of the bordering elements with the relation:

$$tg^4(\alpha)=\frac{1+\dfrac{t_w\cdot L}{2A_c}}{1+t_w\cdot h\cdot\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c\cdot L}\right)} \qquad (6.3)$$

where

t_w the thickness of the sheet of the shear panel;

A_b the area of the horizontal bordering element (beam);

A_c the area of the vertical bordering element (pillar);

I_c moment of inertia of the vertical bordering element;

L the distance between the axes of the vertical bordering elements;

h the distance between the axes of the horizontal bordering elements.

Angle α shall be limited below to 30° and above to 55°.

(943) The shear panel shall be designed in such a way that the versatility of its core fulfils the condition:

$$\frac{min\,(L,h)}{t_w}\le 25\sqrt{\frac{E}{f_y}} \qquad (6.4)$$

where L, h and t have the significance of $_w$(942).

Note: This limitation requires the veil to occur in the elastic range of behaviour.

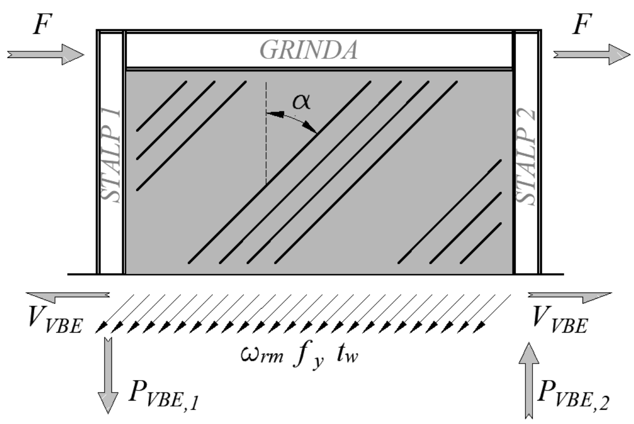


Figure 6.2 Shear panel forces and reactions

(944) Modelling of the sheet metal panel can be carried out:

(uuuuuuuuuuuu) by digitising it with type elements *shell*, in which the main axes rotate at an angle ”α” and which have negligible compression rigidity;

(vvvvvvvvvvvv) by digitising it with bar type parallel elements , with an area A_S , oriented by diagonal field (forming angle α with vertical), which are active only at stretching; this digitisation may be used for non-linear static calculation.

(945) The cross-sectional area of a diagonal element shall be determined by the relation:

$$A_s = \frac{[L\cos(\alpha) + h\sin(\alpha)]\,t_w}{n} \tag{6.1}$$

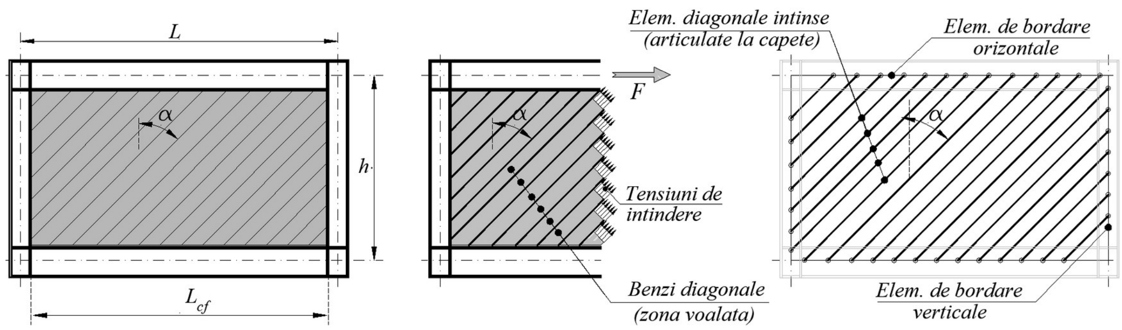
where:

L the width of the shear panel;

h height of the shear panel;

n the number of parallel diagonal elements; usually at least 10 diagonal elements are used. The width of a diagonal element is less than or equal to $60\,\varepsilon\,t_w$;

t_w the thickness of the sheet of the shear panel.



a. b. c.

Elem. diagonale intinse (articulate la capete)	Stretched diagonals elements (articulated at the ends)
Elem de bordare orizontale	Horizontal bordering element
Tensiuni de întindere	Tensile stresses
Benzi diagonal (zona voalata)	Diagonal strips (deformed area)
Elem. de bordare verticale	Vertical bordering elements

Figure 6.3 Calculation model of the shear panel by digitising it into diagonal stretched elements

(946) The sum of the horizontal projections of the diagonal areas is taken to be equal to the area of the homogeneous shear panel ($n A_s \sin\alpha$ ¿ $L_{cf} t_w$).

6.11.4 Calculation of plating elements - pillars and beams

(947) In the case of the design of the structure for the ductility class DCH, the formation of plastic joints in horizontal bordering elements (beams) is acceptable only in the vicinity of vertical bordering elements (pillars). Beams shall be made with sections of class 1. Beams shall be provided with side links meeting the requirements of6.6.2, (807). For directing the plastic joints in the beam, the width of the soles can be reduced in the vicinity of the beam-pillar joint according to 6.6.2, (809).

(948) In the case of the design of the structure in the ductility class DCM, the formation of plastic joints in horizontal bordering elements (beams), which will have sections of classes 1 or 2 sections, is not likely.

(949) In the case of design of the structure in ductility classes DCM or DCH, the vertical bordering elements (pillars) shall have sections of classes 1 or 2 sections.

(950) The horizontal bordering elements shall be designed for a force corresponding to the flow of the sheet metal panel.

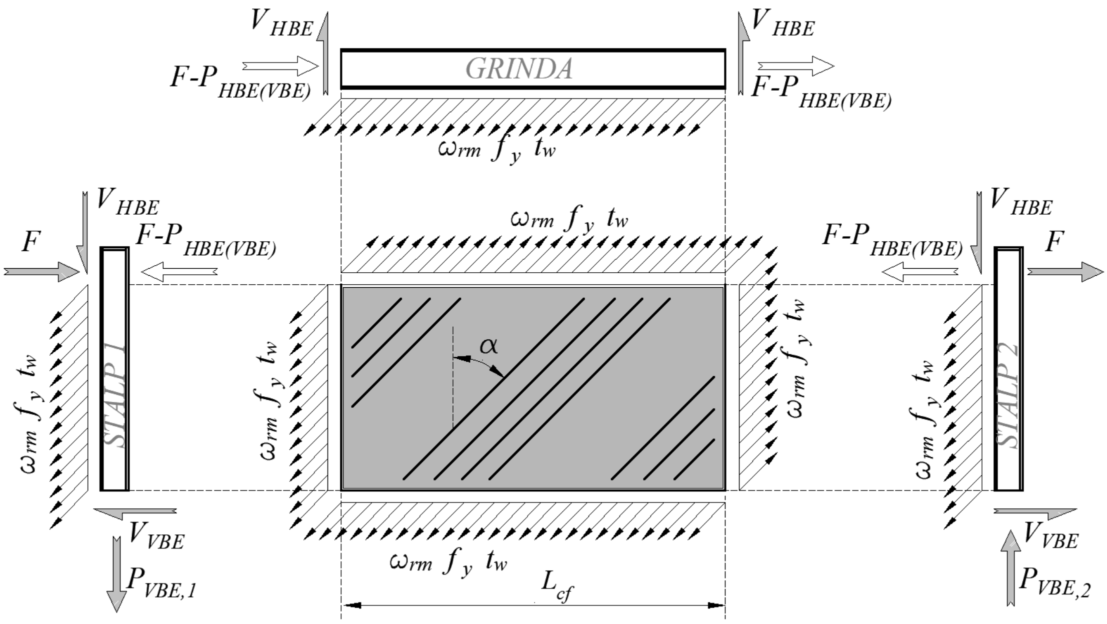


Figure 6.1 Effort status of sheet metal sheet (core) and bordering elements

(951) The thickness of the core of the bordering elements (horizontal and vertical) shall be taken at least equal to the thickness of the shear panel taking into account the overstrength of the steel of the panel. ($t_{wi} \geq t_w \omega_{rm}$)

(952) Horizontal bordering elements (beams) shall be checked at bending and shear force with the relations:

$$\frac{M_{Ed}}{M_{N,Rd}} \leq 1{,}00 \quad (6.1)$$

$$\frac{V_{Ed}}{V_{Rd}} \leq 0{,}50 \quad (6.2)$$

if $N_{Ed}/N_{Rd} > 0{,}15$, $M_{N,Rd} = M_{N,pl,Rd,b}$

if $N_{Ed}/N_{Rd} \leq 0{,}15$, $M_{N,Rd} = M_{Rd}$

$M_{N,pl,Rd,b}$ bending strength of the horizontal bordering section (beams) taking into account the influence of axial force - see relation 6.6.3.1(6.6);

$$M_{Ed} = \frac{w_u L_h^2}{8} + M_{Ed,G} \quad (6.3)$$

where:

L_h distance between (possible) potential plastic joints in the horizontal bordering element;

$$L_h = L - 2 s_h \quad (6.4)$$

s_h the distance between the axis of the vertical bordering element (pillar) in the axis of the potential plastic joint of the horizontal bordering element (beam); $s_h = ½ (d_c + d_b)$, where d is the section height of the vertical bordering element and $_cd_b$ is the section height of the horizontal bordering element;

L panel opening (distance between pillar axes);

w_u uniformly distributed load due to shear panel sheet plasticization, which can be calculated with the relation:

$$w_u = \omega_{rm} f_y (\Delta t_w) \cos^2 \alpha \quad (6.5)$$

$M_{Ed,G}$ the maximum bending moment along the beam produced by the gravitational loads, from the grouping of loads that includes the seismic action, considering the horizontal bordering element (beam) being simply leaned on the opening L_h;

w_g uniformly distributed load at the level of the horizontal bordering element of the load grouping including seismic action;

Δt_w the difference between the thicknesses of the cores of the shear panels adjacent to the horizontal bordering element (above and below the beam);

(953) The axial force at which the horizontal bordering elements (beams) are checked shall be calculated with the relation:

$$N_{Ed} = \frac{1}{2} \omega_{rm} f_y \left[t_{w,i} \sin(2\alpha_i) - t_{w,i+1} \sin(2\alpha_{i+1}) \right] L_{cf} \quad (6.6)$$

(954) The shear force at which the horizontal bordering elements (beams) are checked shall be calculated with the relation:

$$V_{Ed}=\frac{2M_{N,pl,Rd,b}}{L_h}+\frac{w_u}{2}L_{cf}+V_{Ed,G} \qquad (6.7)$$

$M_{N,pl,Rd,b}$ bending strength of the horizontal bordering section (beams) taking into account the influence of axial force - see relation 6.6.3.1(6.6);

(955) The ultimate versatility of the bordering elements for double sections 'T' shall be limited according to the relation:

$$\text{soles: } \frac{b_f}{2t_f}\leq 0{,}30\sqrt{\frac{E}{f_y}} \qquad (6.8)$$

$$\text{core: } \frac{h_c}{t_{wi}}\leq 2{,}24\sqrt{\frac{E}{f_y}} \qquad (6.9)$$

where:

f_y the flow limit;

b_f the width of the sole of the bordering elements;

t_f the thickness of the sole of the bordering elements;

h_c the height of the core of the bordering elements;

t_{wi} the thickness of the core of the bordering elements ($t_{wHBE};t_{wVBE}$);

E modulus of elasticity of steel.

(956) Bordering elements shall fulfil the conditions:

(wwwwwwwwwwww) for vertical bordering elements:

$$I_{VBE}\geq 0{,}0031t_w\frac{h^4}{L} \qquad (6.10)$$

(xxxxxxxxxxxx) for horizontal bordering elements:

$$I_{HBE}\geq 0{,}0031(\Delta t_w)\frac{L^4}{h} \qquad (6.11)$$

where:

L the distance between the axes of the vertical bordering elements;

h the distance between the axes of the horizontal bordering elements;

I_{VBE} moment of inertia of the section of the vertical bordering element;

I_{HBE} moment of inertia of the section of the horizontal bordering element;

t_w the thickness of the core of the shear panel;

Δt_w the difference between the thicknesses of the cores of the shear panels adjacent to the horizontal bordering element (above and below the beam).

(957) If the steel used in the horizontal bordering element is different from that used in the shear panel, the minimum core thickness of the bordering element is given in the relation:

$$t_{wHBE} \geq \frac{t_w \omega_{rm} f_y}{f_{yHBE}} \tag{6.12}$$

where:

t_{wHBE} the thickness of the core of the horizontal bordering element;

f_{yHBE} the flow limit of the steel used in the horizontal bordering element;

(958) In the case of design of the structure in the ductility class DCH, the vertical bordering elements (pillars) shall fulfil the condition:

$$\sum M_{N,pl,Rd,c} \geq \sum \left[\omega_{rm}\omega_{sh}\left(M_{pl,Rd,b} + s_h V_{Ed,M}\right)\right] \tag{6.13}$$

where:

$\sum M_{N,pl,Rd,c}$ is the sum of the bending strengths of the sections of the vertical bordering elements (pillars) entering the joint, in the presence of the axial force (calculated with the relation 6.6.3.1(6.6));

$\sum M_{pl,Rd,b}$ is the sum of the bending strengths of the sections of the horizontal bordering elements (beams) and the adjacent frame beams (if any) entering the joint;

$V_{Ed,M}$ is the shear force associated with the plasticization mechanism in the horizontal bordering element (beam) (see 6.6.2. (4));

ω_{rm} is given in paragraph 6.2.3 (see Table 6.2);

ω_{sh} is given in paragraph 6.5.5. (see Table 6.4);

s_h is the distance between the centre of the plastic joint of the beam and the axis of the pillar.

(959) Vertical bordering elements (pillars) shall be checked against the relation of:

$$\frac{N_{Ed}}{N_{b,zRd}} + \frac{M_{Ed}}{M_{Rd}} \leq 1 \tag{6.14}$$

where:

$$N_{Ed} = N_{Ed,G} + \Omega_T \cdot N_{Ed,E} \tag{6.15}$$

$$M_{Ed} = \frac{\omega_{rm} f_y \sin^2(\alpha) t_w h^2}{12} + M_{Ed,G} \tag{6.16}$$

Ω_T value of structural system overstrength: $\Omega_T = \omega_{rm}\omega_{sh}\Omega_d$;

$$\Omega_d = \frac{V_{Rd}}{V_{Ed}} \tag{6.17}$$

Note: Value of overstrength Ω_T is limited in such a way that the condition $\Omega_T \leq q$ is met; (where q is the behaviour factor of the structure – see Table 6.3). In the case of a simplified calculation, the values of structural system overstrength Ω_T can be adopted from Table 6.5.

V_{Ed} and V_{Rd} are given in Chapter 6.12.3;

$N_{Ed,G}$ and $M_{Ed,G}$ are the efforts from the non-seismic actions contained in the load grouping that includes seismic action.

6.11.5 Joints of shear panels

(960) The attachment of the shear panels to the horizontal and vertical bordering elements shall be dimensioned to the efforts of:

$$n_{HBE}=\omega_{rm} f_y \cos(\alpha) t_w \quad (6.1)$$

$$n_{VBE}=\omega_{rm} f_y \sin(\alpha) t_w \quad (6.2)$$

where:

n_{HBE} the effort per unit length to attach the panels to the horizontal bordering element facing in a direction perpendicular to the axis of the element;

n_{VBE} the effort per unit length to attach the panels to the vertical bordering element, oriented in a direction perpendicular to the axis of the element.

(961) Relations (6.1) and (6.2) shall be used for clamping panels to welded or screwed bordering elements.

(962) Screw joints shall be category A joints and shall be calculated in accordance with SR EN 1998-1-8.

(963) Screw joints shall be sized so that the shear strength of the screws must exceed the pressure resistance on the walls of the hole by at least 20 %.

(964) In the joint area beam-pillar the shear panel is cut circularly (quarter of a circle). It is recommended that the cut-out area be stiffened.

6.11.6 Joints of bordering elements

(965) The joints between the bordering elements (beam-pillar) shall be made as joints in dissipative areas according to the provisions 6.5.5.

6.12 Dual structures

(966) Dual structures with unbraced frames and braced frames working in the same direction shall be designed using a single factor q. Horizontal forces shall be distributed among the different frames in proportion to their elastic rigidity.

(967) The unbraced frames installed in the wind braced direction of the building shall be designed so that they can absorb at least 25 % of the design seismic action should the wind braced frames stop working. If this requirement is not satisfied, the structure shall be considered as a braced frame (centre or eccentric) and shall be designed according to the provisions of 6.7, 6.8 respectively 6.11.

(968) Unbraced and braced frames shall comply with the provisions of 6.6, 6.7, 6.8, and 6.10.

(969) Centrically braced frames and eccentrically braced frames in inverted ‘Y’ with rigid collar beam-pillar grips, where reduced sections of the beams are provided in the braced openings to avoid plastic deformations in the pillars, are assimilated to dual structures.

6.13 Construction control

(970) The construction control must ensure that the real structure corresponds to the designed one.

(971) To this end, in addition to the provisions of SR EN 1090-2 and the technical regulations in force concerning the quality of welded steel joints of constructions, the following requirements must be met:

(yyyyyyyyyyyy) the detailed engineering and installation drawings shall include details of the joints, the size and quality of the screws and welds, as well as the steel grade. On the drawings, the maximum permissible flow limit of the steel shall be noted $f_{y,max}$ which may be used by the manufacturer in dissipative areas;

(zzzzzzzzzzzz) the control of the tightening of the screws and the quality of the welds must be carried out in accordance with the provisions of the specific technical regulations;

(aaaaaaaaaaaaa) during construction, it shall be verified that the flow limit of the steel used in the bars and dissipation zones is that indicated in the design.

7 Composite structures

7.1 General information

7.1.1 Purpose and scope

(972) This chapter deals with the seismic action design of buildings with the main composite structure made of rolled steel and reinforced concrete, in seismic action.

(973) Specific technical regulations for structures of the SR EN 1994-1-1 series shall be used for the design of composite structures for types of actions other than seismic ones.

(974) This chapter also contains provisions on hybrid structures, consisting of structural components made of steel and structural elements made of reinforced or composite concrete.

(975) If for certain situations no specific provisions are given in this chapter, the provisions for reinforced concrete constructions in Chapter 5 or steel constructions in Chapter 6 contained in this code, as well as in the standards of the SR EN 1992-1-1 series, respectively SR EN 1993-1-1, may be applied, as appropriate.

7.1.2 Definitions

(976) The terms specific to this chapter are:

Composite structure: structure composed of composite structural components.

Composite structural component: structural component consisting of reinforced concrete and rolled steel, where the co-operation between reinforced concrete and rolled steel is manifested at section level. Rolled steel components may be unenclosed, partially encased or fully encased in reinforced concrete.

7.2 Design principles

7.2.1 Ductility classes

(977) Composite buildings shall be designed for one of the three classes of ductility defined in 4.1.2.

(978) Composite structures designed for the ductility class DCH or DCM shall have adequate energy dissipation capability under cyclic stress without significant reduction of resistance to horizontal and vertical forces.

(979) Buildings located in areas of moderate or high seismicity shall be designed for ductility class DCH or DCM.

(980) By way of exception from (373), in areas of moderate or high seismicity, buildings may be designed for the ductility class DCL if their overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations (q=1), irrespective of the location, where it is not possible to meet the design criteria specific to the ductility class DCH or DCM.

(981) Structures not falling within the types indicated in 7.2.2, (983), shall be designed for the ductility class DCL and their overall resistance capacity to horizontal

seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q = 1$).

(982) Structures designed for DCL shall be seismically designed on the basis of the provisions of SR EN 1994-1-1, together with the additional provisions given in this chapter, explicitly indicated for this class of ductility. The basic requirements of seismic design shall be those set out in Chapter 2.

7.2.2 Types of structures

(983) Composite buildings designed for seismic actions have the main structural system as follows:

(bbbbbbbbbbbbb) unbraced frames, made in composite solution with composite beams and pillars or in hybrid solution consisting of e.g. reinforced concrete pillars and steel or composite beams;

(ccccccccccccc) braced frames with centre bracings made in composite or steel solution;

(ddddddddddddd) braced frames with eccentric bracing. Eccentric bracing systems are those defined in 6.1.2, (686). The pillars and beams can be steel elements or composite elements and the dissipative elements shall be made of steel.

(eeeeeeeeeeeee) inverted pendulum structures. In these types of structures, the dissipative zone shall develop at the base of a single vertical composite elements, whilst most of the mass shall be concentrated at the top of the structure.

(fffffffffffff) composite structures with structural composite walls:

(xv) type 1 - composite reinforced concrete walls with rigid reinforcement in the end areas;

(xvi) type 2 - composite or reinforced concrete walls coupled with composite or steel beams;

(xvii) type 3 - composite walls with bulbs and belts with rigid reinforcement or made of metal elements and reinforced concrete panels.

(ggggggggggggg) dual composite structures made of composite walls and frames;

(hhhhhhhhhhhhh) torsion-sensitive composite structures.

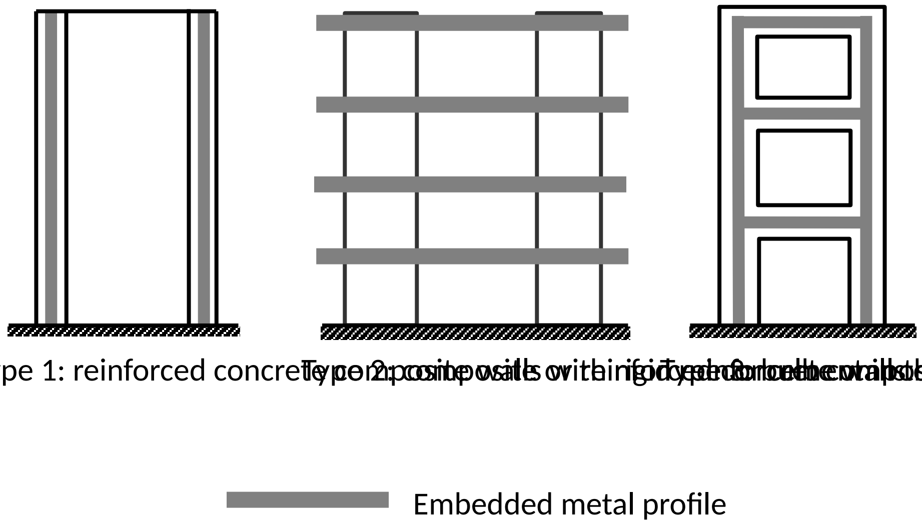


Figure 7.1 Illustrative representation - types of structures with composite walls

(984) Buildings with composite structures may have different structural systems on the two main horizontal orthogonal directions. In the design of the structure, design rules specific to each structural system shall be used, in the appropriate direction.

(985) By way of exception from (984), for composite structures with high torsion flexibility, the same type of structural system shall be used on the two orthogonal directions.

(986) If along a horizontal main direction, the building has a main structural system which is made up of two types, when designing the structure in the direction considered, the lowest value of the behaviour factor corresponding to the two types of structures shall be used.

(987) Structures that do not meet the condition of 4.2.2.1, (132), fall under the category of structures with high torsion flexibility regardless of the type of structural system defined according to 7.2.2, (983).

(988) All main structural components, regardless of the type of structural system, are designed for the same class of ductility.

7.2.3 Plastic mechanism

(989) For structures designed for the ductility class DCH or DCM, the favourable seismic response shall be achieved by forming a plastic mechanism with optimal energy dissipation capacity induced by horizontal seismic action.

(990) In unbraced frame-type structural systems, the plastic mechanism shall be formed by the development of plastic zones at the ends of the beams and at the base of the pillars, immediately above the conventional fixation-clamping section. Exceptions are structures in single-level frames where plastic zones can develop at both ends of the pillars.

(991) For structural frame-type systems with centric wind bracing, the plastic mechanism shall be formed by the development of stretched diagonal or stretched and compressed diagonal plastic zones, before the formation of plastic joints in beams or the loss of overall stability in beams and pillars.

(992) In the case of frame-type structural systems with eccentric wind bracing, the plastic mechanism shall be formed by the development of plastic bending and/or shearing zones in the dissipative bars, before the failure of the joints, flow or buckling of the beams and pillars. Dissipative bars shall be of steel and shall be classified in accordance with Chapter 6.

(993) In inverted pendulum structural systems, the plastic mechanism shall be formed by the development of a plastic joint at the base of the vertical structural element, just above the conventional fixation-clamping section.

(994) In structural systems such as composite structural walls, the plastic mechanism shall be formed by the development of plastic zones at the base of the composite walls, immediately above the conventional fixation-clamping section, and at the ends of the coupling beams, if any.

(995) In dual-type structural systems, structural walls and composite frames, the plastic mechanism shall be formed by the development of plastic zones at the ends of the beams, at the base of the composite walls and pillars, immediately above the conventional fixation-clamping section.

(996) For the control of the plastic mechanism, the design shall be carried out in accordance with the principles of the method of hierarchy of strength capabilities - the design-to-capacity method.

(997) Infrastructures and foundations respond in the elastic domain to the action of the design earthquake corresponding to the ultimate limit state.

7.2.4 Behaviour factors

7.2.4.1 Ultimate limit state

(1) The maximum values of the behaviour factor for horizontal seismic actions for checks at the last limit state shall be chosen according to the provisions of Table 7.1.

Table 7.1 Maximum behaviour factor values for horizontal seismic actions

Types of composite structures	Maximum value of the behaviour factor		
	Ductility class		
	DCH	DCM	DCL
(iiiiiiiiiiii) Composite frames without wind bracing:	4.50 α_u/α_1	3.0α_u/α_1	1.50
(xviii) One-level frames	$\alpha_u/\alpha_1=1{,}10$		
(xix) Frames with an opening and several levels and coupled walls	$\alpha_u/\alpha_1=1{,}20$		
(xx) Multi-open and multi-level frames	$\alpha_u/\alpha_1=1{,}30$		
(jjjjjjjjjjjj) Composite braced frames:			
(xxi) with wind bracing with stretched diagonals	4.00	2.50	1.50

(xxii) with V-diagonal bracing	2.50	2.00	1.50
(xxiii) with eccentric bracing $\alpha_u/\alpha_1=1,20$	$5.00\alpha_u/\alpha_1$	3.00	1.50
(kkkkkkkkkkkkk) Reverse pendulum structures	$2.00\alpha_u/\alpha_1$	2.00	1.50
(xxiv) Dissipation zones at the base of the pillars $\alpha_u/\alpha_1=1,00$			
(lllllllllllll) Structures with structural composite walls	$4.00\ \alpha_u/\alpha_1$	$3.0\ \alpha_u/\alpha_1$	1.50
(xxv) composite walls with composite end areas and reinforced concrete core	$\alpha_u/\alpha_1=1,10$		
(xxvi) composite or reinforced concrete walls coupled with steel or composite beams	$\alpha_u/\alpha_1=1,20$		
(xxvii) composite walls made up of reinforced concrete panels and frames for framing steel or reinforced concrete with rigid reinforcement	$\alpha_u/\alpha_1=1,20$		
(mmmmmmmmmmmmm) Torsion-sensitive composite structures	3.00	2.00	1.50

(2) The value of the ratio between the capable horizontal force of the structure and the horizontal force corresponding to the flow of the first structural element, α_u/α_1, shall fulfil the condition:

$$1,00 \le \alpha_u/\alpha_1 \le 1,30 \qquad (7.2)$$

(3) Ratio value α_u/α_1, for buildings in the class of importance and exposure to earthquakes III or IV, shall be determined according to the provisions of Table 7.1 or by non-linear static calculation.

(4) For buildings in class of importance and exposure to earthquake I or II, the value of the ratio α_u/α_1 shall be determined by non-linear static calculation. The value resulting from the calculation shall be upper limited to 1.30. If the report α_u/α_1 is not be determined by non-linear static calculation, it shall be taken to be 1.00.

(5) In structures designed for DCM or DCH with concrete walls, the maximum value of the behaviour factor shall be multiplied by the factor k_w defined in 5.2.4.1, (407).

(6) In the case of an irregular building, the maximum value of the behaviour factor shall be reduced in accordance with the provisions of 4.5.1.1.

(7) The value of the behaviour factor resulting from the application of the provisions of this paragraph shall be lower limited to 1.00.

7.2.4.2 Service limit state

(8) The value of the behaviour factor for horizontal seismic actions for service limit state checks shall be 1.50.

7.2.5 Local effects caused by interaction with non-structural walls

(9) For the assessment of local effects caused by interaction with non-structural components, the provisions of 5.2.5. shall apply.

7.2.6 Foundations and infrastructures

(1) When designing infrastructures and foundations, the provisions of the specific technical regulations together with the additional provisions given in this chapter shall apply.

(2) Additional provisions regarding the design of infrastructures and foundations for composite wall constructions are given in technical regulation CR 2-1-1.1.

7.2.7 Modelling for calculation

(3) For buildings with concrete structure, when calculating the design value of seismic action, the fraction of the critical damping of the building, ξ, for all modes of vibration shall be considered to be equal to 4 %:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad \xi=4\% \tag{7.1}$$

(4) The rigidity of composite concrete sections in the compressed area shall be calculated in order to determine the values of the sectional efforts by converting them into equivalent sections, taking into account an equivalence coefficient

$$n=E_a/E_{cm} \tag{7.2}$$

where:

E_a and E_{cm} are the steel elastic modulus and the concrete elastic modulus for short-term loads.

(5) In calculating the rigidity of composite sections, the contribution of stretched concrete shall be neglected.

(6) Two flexural rigidities can be considered for composite beams: $E_a I_1$ for the positive moment zone, taking into account the actual slab width in the compressed zone, and $E_a I_2$ for the negative moment zone, considering the reinforcement in the actual width of the stretched slab, where I_1 and I_2 are the moments of inertia of the equivalent steel sections in the positive and, respectively negative moment zones.

(7) The actual width of the slab for the calculation of rigidity shall be determined according to the provisions of Table 7.2.

Table 7.1 Partial effective width of the composite beam slab in the joint area b_e

b_e	Structural conditions in the area of the beam-pillar joint	b_e for calculating the capable moment M_{Rd} (plastic)	b_e for calculating the rigidity *EI* (elastic)
A. Inner pillar	There is or there is not a transverse beam with additional reinforcement A_T and A_S	For M^- : 0.1 l For M^+ : 0.075l	

B1. External pillar	There is a transverse marginal beam resting on the pillar in which the longitudinal reinforcements are anchored , with total connection to the slab and additional reinforcements in the slab A_T and A_S,	For M^- : 0.1 l For M^+ : 0.075 l	For M^- : 0.05 l For M^+ : 0.0375 l
B2. Pillar pillar	There is a slab strip overhanging from the pillar, where the longitudinal reinforcements are anchored using loops and additional reinforcements	For M^- : 0.1 l For M^+:$b_c/2+0.7h_c/2$ or $h_c/2+0.7b_c/2$	
B3. External pillar	There is an additional device fixed to the pillar sole, which has a width b_{el} larger than the width of the pillar sole b_c, where the longitudinal reinforcements within the slab are not anchored	For M^- : 0 For M^+:$b_c/2 \leq b_{e,max}$ $b_{e,max} = 0.05l$	For M^- :0 For M^+ : 0.0375 l
B4. External pillar	There is no transversal element or the longitudinal reinforcements are not anchored to the pillar	For M^- : 0 For M^+: $b_c/2$ or $h_c/2$	For M^- : 0 For M^+ : 0.025 l

where:

M^-, M^+ indicates the calculation situations of the value of the partial effective slab width b_e (in the negative and respectively, positive moment zone). For the negative moment, as the slab concrete is fissured, the partial effective slab width b_e shall include the tensile reinforcements which play a role in determining the capable moment and rigidity.

l the interax opening of the beam;

b_c the width of the pillar perpendicular to the axis of the beam;

h_c the height of the section of the pillar;

b_{el} the width of the additional element welded to the pillar;

A_S *and* A_T additional reinforcements placed in the plate in the pillar area ((A_s longitudinal reinforcement, and A_T transverse reinforcement). The calculation relations for these reinforcements and for the resultant of the compression efforts in the slab are given in Annex C of SR EN 1998-1.

(8) A simplified calculation of the beam rigidity can be carried out, considering that the entire composite beam has a constant equivalent moment of inertia equal to:

$$I_{eq} = 0{,}6\, I_1 + 0{,}4\, I_2 \qquad (7.3)$$

where I_1 and I_2 are defined in accordance with (6).

(9) For composite pillars, the equivalent rigidity shall be calculated with the relation:

$$(EI)_c = 0{,}9(E I_a + 0{,}5 E_{cm} I_c + E_s I_s) \tag{7.4}$$

where

I_a, I_c, I_s are the moments of inertia of the reinforcement, concrete and, respectively, rigid steel sections.

7.3 Seismic performance criteria

7.3.1 General information

(10) The provisions of this section apply to the main structure, with a role in balancing the seismic action.

(11) In the seismic design of composite structures, the provisions given in this section apply together with the provisions specific to the other technical regulations for the design of composite buildings, according to 7.1.1, (973).

7.3.2 Resistance

(12) Composite buildings shall meet the condition of resistance to horizontal action laid down in 4.3.2.1.

(13) The design value of the resistance capacity shall be greater than or equal to the design value of the effort in the section considered. This condition shall be fulfilled for all main structural components along their entire length.

(14) In the case of major seismic components subjected to bending, with or without axial force, and shear force, the conditions are met:

$$M_{pl,Rd} \geq M_{Ed} \tag{7.1}$$

$$0{,}50\, V_{pl,Rd} \geq V_{Ed} \tag{7.2}$$

where

$M_{pl,Rd}$ design value of the bending strength;

M_{Ed} design value of the bending moment;

$V_{pl,Rd}$ design value of the strength capacity to the shear force;

V_{Ed} the design value of the shear force.

(15) In the case of major seismic components subjected to bending with axial force, the design value of the bending tensile strength shall be determined taking into account the design value of the axial force and shear force. The assessment shall be made separately for each direction and sense of seismic action.

(16) Steel bracings and dissipative elements of frames braced with centre or eccentric bracings shall be designed in accordance with Chapter 6.

(17) In the case of main seismic components subjected to the centric axial force, the resistance condition shall be ensured by limiting the normalized axial effort according to the provisions of this technical regulation.

(18) Stability, strength and rigidity to horizontal seismic actions of structures is not ensured by the torsion response of structural components. The torsional strength and

rigidity of structural components shall be neglected in seismic design. Exceptions may be made for some inverted pendulum structural systems where the torsion response of structural components is necessary to ensure stability, resistance and rigidity to seismic action and the torsion resistance capacity must be explicitly verified.

7.3.3 The transfer of stresses and deformations between the steel and the concrete

(19) To ensure that the composite action is manifested within the entire stress range, the stresses and deformations transfer shall be ensured between the steel component and the reinforced concrete component by adherence, friction or connectors. The achievement of the design values of the bending moments with axial force and the capable shear force of the composite elements is conditioned by ensuring an efficient collaboration between the reinforced concrete and steel components.

(20) For calculating the design value of longitudinal slip capable of adhesion and friction τ_{Rd}, the following design values of tangential effort (values from SR EN 1994-1-1 multiplied by 0.5) shall be used between steel and concrete components:

- 0.33 N/mm^2, for fully embedded steel sections having a coating of 100 mm or more;
- 0.10 N/mm^2, for the soles of partially embedded profiles;
- 0, for the cores of partially embedded profiles;
- 0.275 N/mm^2, for the interior of circular pipes filled with concrete;
- 0.20 N/mm^2, for the interior of rectangular pipes filled with concrete.

(21) The design values of the slip forces shall be taken to be equal to the values associated with the dissipation mechanism multiplied by a partial safety coefficient $\gamma_{Rd}=1{,}20$.

(22) In the case of composite steel beams with reinforced concrete slab, the adhesion between the concrete and the steel section sole shall be neglected, in taking over the tangential efforts, the slipping being entirely taken over by the connectors.

(23) For composite pillars, the distribution of vertical reactions transmitted by the beams in joints between the reinforced concrete and steel components shall be ensured, the distribution being proportional to the rigidity of these components.

(24) In the case of composite pillars where adhesion and friction cannot fully ensure the transfer of tangential stress associated with the dissipation mechanism, by exceeding the design values of the tangential stress given at 7.3.3, (20), connectors shall be provided to ensure full connection and retrieval of the design slip forces.

7.3.4 Ductility

(25) Composite buildings shall be constructed in such a way as to meet the conditions of ductility under horizontal seismic action provided in 4.3.1.2.

7.3.5 Stability

(26) Composite buildings shall be constructed in such a way as to meet the conditions of stability under seismic action provided in 4.3.1.3.

7.3.6 Rigidity

(27) Composite buildings shall meet the rigidity conditions under horizontal seismic action given on 4.3.2.

(28) The design value of the permissible level relative displacement shall be determined in accordance with the provisions 4.3.2.1, (238).

7.4 Stresses design values

(29) This chapter contains provisions on the determination of the design values of the efforts that are developed in the main structural components for strength checks.

7.4.1 Buildings designed for DCH or DCM ductility class

(30) The design value of an effort caused by seismic action shall be equal to the maximum value of that effort that develops as a result of the incidence of seismic design action.

(31) When establishing the design values of efforts, other than those producing deformations in plastic areas corresponding to the optimal mechanism, the efforts producing plastic deformations in critical areas shall be determined by multiplying the design values of the strength capacities by a partial safety coefficient assessing the uncertainties in the strength capacity calculation model, mainly caused by the post-elastic steel reinforcement effect, γ_{Rd}.

(32) The design effort values shall be determined by:

(nnnnnnnnnnnn) transforming the efforts resulting from the calculation of the structure by a linear static calculation method, in order to quantify the non-linearity of the structural response caused by the design seismic action, in accordance with the principles of the resistance capacity hierarchy method;

or

(oooooooooooo) directly, by non-linear calculation.

(33) The determination of the design values of the efforts of the main structural components, based on the efforts resulting from the calculation of the structure by a linear static calculation method, shall be carried out according to the provisions of chapters 5 and/or 6.

7.4.1.1 Beams

(34) The design values of bending moments in the plastic areas of the beams shall be determined by calculating the structure in the seismic grouping.

(35) The design values of the bending moments in the beams in the elastic response area shall be established from the beam balance in the situation of plastic zones formation, considering also the loads acting transversely on the axis of the beam in the seismic grouping.

(36) The values of the shear forces shall be established from the balance of the beam in the situation of the formation of the plastic mechanism, considering the loads acting transversely on the axis of the beam in the seismic grouping, according to the provisions of the Chapter 5, in the case of composite reinforced concrete beams with rigid reinforcement, and in accordance with the provisions of Chapter 6, for composite steel beams with reinforced concrete slabs.

7.4.1.2 Pillars

(37) For composite reinforced concrete pillars made with embedded frame profiles without bracing, the design values of the efforts shall be determined with the relations provided for in 5.4.1.2.

(38) For composite pipe pillars filled with concrete or with partially embedded steel section that are part of structures in unbraced or braced frames, with centric or eccentric bracings, the sectional design efforts shall be determined according to the provisions of Chapter 6.

7.4.1.3 Joints

(39) The design value of the shear force in the joint is determined from its equilibrium when forming the plastic mechanism, distinctly for each direction of seismic action and for each direction of calculation.

7.4.1.4 Coupling walls and beams

(40) The design effort values for walls and coupling beams shall be determined in accordance with CR 2-1-1.1.

7.4.1.5 Diaphragms

(41) The provisions of this paragraph are applied to the design effort in the diaphragms constituted by the floors subjected to stress by loads in the median plane.

(42) The design values of the diaphragm efforts shall be determined on the basis of the efforts associated with the mobilisation of the overall plastic mechanism of the structure, taking into account the impreciseness of the calculation.

(43) The forces in a diaphragm shall be determined by considering its equilibrium under the action of horizontal forces and the design values of the shear forces in the structural elements that load the diaphragm in the horizontal direction.

7.4.1.6 Infrastructures and foundations

(44) The design values of the efforts and deformations in the infrastructure elements shall be established considering their balance under the efforts related to the superstructure and the support efforts on the ground.

(45) When designing the infrastructure and foundations, the maximum values of the efforts related to the superstructure, corresponding to the situation of the formation of the plastic mechanism, and the loads acting directly on them shall be considered.

(46) The design values of the efforts and deformations in the infrastructure elements shall be established considering the land-structure interaction.

(47) In the case of elements with insulated foundations, the design values of the efforts at the base of their critical area, E_{Fd}, shall be determined by transforming the effort values resulting from the linear static calculation with the equation:

$$E_{Fd} = E_{F,G} + \gamma_{Rd} \Omega E_{F,E} \tag{7.1}$$

where:

$E_{F,G}$ the sectional effort produced by actions other than seismic action that are included in the seismic grouping;

$E_{F,E}$ sectional effort resulting from the calculation at seismic design action;

Ω bending overstrength factor of the wall;

γ_{Rd} partial safety coefficient taking into account the uncertainty contained in the strength assessment, to be chosen according to the relation 5.3.2.1(5.4).

(48) Further provisions on the establishment of design effort values for buildings with composite wall structure are given in technical regulation CR 2-1-1.1.

7.4.2 Buildings designed for DCL ductility class

(49) The design values of sectional efforts shall be determined in accordance with 5.4.2.

7.5 Resistance capacity

7.5.1 Beams

(50) The calculation of the bending strength and shear force of the beams shall be made on the basis of the specific provisions of the Romanian standard SR EN 1994-1-1, together with the additional provisions given in this paragraph.

(51) In critical areas of composite beams the following conditions shall be met:

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1{,}00 \tag{7.1}$$

$$\frac{N_{Ed}}{N_{pl.Rd}} \leq 0{,}15 \tag{7.2}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} \leq 0{,}50 \tag{7.3}$$

where:

M_{Ed}, N_{Ed}, V_{Ed} design values of sectional beam efforts;

$N_{pl,Rd}$, $M_{pl,Rd}$, $V_{pl,Rd}$ the design values of the plastic capable stresses of the beam;

(52) The design values of the capable efforts of composite beams shall be determined in accordance with the provisions of SR EN 1994-1-1.

7.5.1.2 Composite steel beams with reinforced concrete slabs

(53) For the calculation of the bending strength capacity, the active width of the slab $b_{eff,Rd}$ shall be determined according to Table 7.3.

Table 7.1 Active slab width for calculating the plastic strength capacity of steel beams with concrete slab

Effort in the slab	Position of the pillar	Cross-sectional element	$b_{eff,Rd}$
Tension	Interior	Cross-reinforcement in the slab	b_{eff}

Tension	Exterior	Longitudinal reinforcement anchored in a facade beam or slab area in cantilever and transverse reinforcement in the slab	b_{eff}
Tension	Exterior	Without longitudinal reinforcement anchored in a facade beam or slab area in cantilever and transverse reinforcement in the slab	0.0
Compression	Interior/exterior	There are transverse beams with connectors and transverse reinforcement in the slab	b_{eff}
Compression	Interior/exterior	There are no cross beams with connectors, but there is cross reinforcement in the slab	$b_d + 0.7 h_c$
Compression	Exterior (perimeter frames)	There are no cross beams with connectors, but there is cross reinforcement in the slab	b_c

where

b_{eff} active width of calculation defined in accordance with SR EN 1994-1-1;

b_c the width of the pillar perpendicular to the axis of the beam entering the joint;

h_c height of the pillar.

(54) If the slab is completely disconnected from the beams around a pillar on a circular area of diameter $2b_{eff}$, where b_{eff} is the greatest value of the active widths of the beams entering the joint, then the plastic bending resistance capacity shall be determined by considering only the metal profile.

(55) The slab is considered to be completely disconnected if there is no contact between the slab and any element arranged in the transverse direction.

Note: Such elements can be pillars, connectors, gussets, etc.

(56) In the dissipative areas of the beams, bars shall be arranged in the slab to ensure the integrity of the slab and ensure the transfer of efforts from the beam to the pillar. The method of calculation and detail is given in Annex C of SR EN 1998-1.

7.5.1.3 Reinforced concrete composite beams with rigid reinforcement

(57) For the design of reinforced concrete composite beams with rigid reinforcement, the provisions of SR EN 1994-1-1 and the provisions of the chapters 5 and/or 6 shall be complied with, unless they are contrary to the provisions of this Chapter.

(58) The actual width of the slab for calculating the beams at the bending strength limit state shall be determined in accordance with 5.5.1.1. Slab reinforcements are considered to be active at a negative moment if they are placed on the width of the b_{eff} and whether they are properly anchored.

(59) For checking the shear force of the beams, the design shear force V_{Ed} shall be distributed between the reinforced concrete section, $V_{Ed,c}$, and steel, $V_{Ed,a}$, in relation to the values of the capable design moments of these components, $M_{Rd,c}$ and steel $M_{Rd,a}$.

(60) Relations for calculating design values of capable efforts $M_{pl,Rd}$, of composite beams are given in SR EN 1994-1-1.

(61) $V_{pl,Rd,c}$ and $V_{pl.Rd,a}$ shall be determined in accordance with the provisions of Chapters 5 and/or 6. Verification of the ability to withstand shearing shall be carried out with the relations:

$$\frac{V_{Rd,a}}{V_{pl,Rd,a}} \leq 0,5 \tag{7.1}$$

$$\frac{V_{Ed,c}}{V_{pl,Rd,c}} \leq 1,00 \tag{7.2}$$

(62) In structures designed for class DCM or DCH, the areas at the ends of the beams with the length $l_{cr} = 1,50\, h_w$ measured from the front of the pillars or areas of the same length situated on either side of a section of the beam field in which it touches $M_{pl,Rd}$, where h_w is the height of the core of the beam.

7.5.2 Pillars

7.5.2.1 Reinforced concrete composite pillars with rigid reinforcement

(63) The design effort values for composite pillars shall be determined in accordance with the provisions of 5.4.1.2.

(64) In critical areas of reinforced concrete composite pillars with rigid reinforcement, the following conditions shall be met:

$$\frac{M_{Ed}}{M_{pl,Rd}} \leq 1,0 \tag{7.1}$$

$$\frac{N_{Ed}}{N_{pl.Rd}} \leq 0,3 \tag{7.2}$$

$$\frac{V_{Ed,c}}{V_{pl,Rd,c}} \leq 1,0 \tag{7.3}$$

$$\frac{V_{Ed,a}}{V_{pl,Rd,a}} \leq 0,5 \tag{7.4}$$

(65) The design values of the efforts shall be determined in such a way as to favour the development of the favourable seismic energy dissipation mechanism. At a certain level, the moments in the pillars and beams can be redistributed under the conditions of achieving the joint balance and keeping the level shear force constant.

Design values of the resistance capacity at the bending moment, $M_{pl,Rd}$, of composite pillars shall be determined in accordance with the provisions of SR EN 1994-1-1.

(66) Design value of shear force resistance capacity $V_{pl,Rd}$ of a pillar shall be determined as the sum of the design values of the strength of the reinforced concrete components, $V_{pl,Rd,c}$, and steel, $V_{pl,Rd,a}$, determined in accordance with Chapters 5 and/or 6 of this technical regulation.

The shear force acting on a composite pillar shall be distributed between the reinforced concrete section, $V_{Ed,c}$, and steel, $V_{Ed,a}$, in relation to the design values of the bending moments of these components, $M_{Rd,c}$ and steel $M_{Rd,a}$.

7.5.2.2 Composite pillars made of concrete-filled tubing

(67) For the design of steel pipe pillar filled with concrete or filled and embedded in concrete, the provisions of SR EN1994-1-1, paragraph 6.7 shall be complied with.

(68) In the case of main dissipative structural components made of concrete-filled pipes, the design value of the shear force resistance capacity of the pillar shall be determined taking into account only the contribution of the steel section or only the contribution of the reinforced concrete core, considering the steel pipe as cross-reinforcement.

7.5.2.3 Composite pillars with steel section partially embedded in reinforced concrete

(69) For the design of the pillars with the steel section partially embedded in reinforced concrete, the provisions of SR EN1994-1-1 shall be complied with.

(70) In the case of main dissipative structural components, the design value of the shear force resistance capacity shall be determined taking into account only the contribution of the steel section. Exceptions are made where special measures are taken to mobilize the shear force resistance capacity of reinforced concrete by making transverse connections between the concrete and the steel element.

7.5.3 Frame joints

(71) For the design of composite and hybrid joints, the provisions of chapters 5 and/or 6.

(72) Composite joints are dimensioned with a degree of assurance above the dissipative areas of adjacent elements so that nonlinear deformations develop in them.

(73) During seismic action, the integrity of the compressed concrete of the slab around the pillars shall be ensured by the provision of additional reinforcements. Slab reinforcements, located in the joint area, shall comply with the composition conditions set out in Annex C of SR EN 1998-1.

(74) When designing welded or screwed joints of elements in the joint, the following condition shall be met:

$$R_d > 1{,}5\, R_{fy} \qquad (7.1)$$

where

R_d the design value of the effort capable of joining;

R_{fy} the design value of the capable stresses of the dissipative elements being joined.

(75) In the case of composite beam-pillar joints where the steel panel of the joint is fully embedded in the concrete, the strength of the joint shall be calculated as the sum of the contribution of the reinforced concrete and the steel panel of the joint to be determined according to the provisions of the Chapters 5 and/or 6, if the following conditions are met:

$$0{,}6 \leq h_b / h_c \leq 1{,}4 \tag{7.2}$$

$$V_{j,Ed} \leq V_{j,Rd} \tag{7.3}$$

where:

h_b and h_c are the dimensions of the joint panel equal to the height of the steel section of the beam and, respectively, the pillar;

$V_{j,Ed}$ the design value of the shear force in the joint associated with the formation of plastic joints in critical areas of adjacent composite beams, calculated from the design values of the resistance at the bending moment of the reinforced concrete components, $M_{Rd,c}$, and steel, $M_{Rd,a}$, also considering the steel overstrength factor.

$V_{j,Rd}$ design value of the shear force resistance capacity of the composite joint.

Design value of joint shear force resistance capacity, $V_{j,Rd}$, shall be determined as the sum of the design values of the shearing strength capabilities of the reinforced concrete components, $V_{j,Rd,c}$, and steel, $V_{j,Rd,a}$, of the joint, determined in accordance with Chapters 5 and/or 6 of this technical regulation.

(76) When designing composite joints consisting of composite steel beams with reinforced concrete slabs and composite or reinforced concrete pillars, the following measures shall be applied:

(ppppppppppppp) vertical stiffeners are placed at the front of the pillar;

(qqqqqqqqqqqqq) the shear force in the beams is distributed between the additional vertical reinforcements welded to the beam base and the steel section of the pillar.

(77) When designing hybrid joints consisting of steel or composite beams and reinforced concrete pillars, the following measures shall be applied:

(rrrrrrrrrrrrr) the steel beam passes continuously through the joint;

(sssssssssssss) vertical stiffeners are arranged at the front of the pillar;

(ttttttttttttt) in the vicinity of the vertical stiffeners in the joints, additional vertical reinforcements welded to the soles of the beam are arranged in the pillars, having a design value of tensile strength equal to the shear force of the design of the steel beam. Concrete in the area of these reinforcements shall be confined with transverse reinforcement that meets the conditions of paragraph 7.6.3.1.

(78) Hybrid joints consisting of reinforced concrete pillars and steel beams shall not be not used for structures of classes DCH and DCM.

7.5.4 Composite walls

(79) For the calculation of the design values of the strength capacities and for the construction of composite walls, the provisions of Chapters 5 and/or 6 and technical regulation CR 2-1-1.1 shall be complied with.

(80) In the case of composite walls, it is considered that the shear force is taken over entirely by the reinforced concrete core of the wall and the bending moment by the whole wall.

(81) The transfer of tangential efforts between the areas at the ends of the wall and the reinforced concrete panel of the wall core shall be carried out through connectors, through bars welded to the steel section or bars passed through the holes of the rigid reinforcement.

7.6 Rules of composition

(82) The main structural components shall comply with the composition conditions set out in this paragraph.

(83) The geometry of the section, reinforcement and versatility of the steel profiles shall be established in conjunction in such a way that the failure of the bending sections, with or without axial force, does not occur by crushing the compressed concrete before the flow of the steel profile or the stretched longitudinal reinforcement. This condition applies to beams, pillars and walls for structures of classes DCH and DCM.

(84) The provisions on the quality of materials, the composition and reinforcement of the main structural components are laid down in a differentiated manner for critical and current areas.

(85) The length and position of the critical areas of the main structural components shall be differentiated according to the type of element, the stress state and the ductility class, in accordance with the provisions of this paragraph. The part of the element that is located outside critical areas shall be considered a current area.

7.6.1 Quality of materials

(86) For concrete and steel reinforcements, the provisions of Chapter 5, and for steel metal profiles, the provisions of Chapter 6 shall be complied with.

(87) The maximum class of concrete is C40/50.

7.6.2 Slenderness of steel sections walls which make up the composite elements

(88) Compressed areas of composite elements with the section of steel not enclosed in concrete shall comply with the suppleness conditions laid down in Chapter 6 of this code. In the case of dissipative zones of composite elements with the steel section embedded in the concrete, the suppleness is less than or equal to the maximum limits laid down in Table 7.4.

> Note: The ductility of the dissipative composite elements subjected to compressive and bending forces depends on preventing the occurrence of local instability phenomena in the steel elements. Therefore, the slenderness of the steel section walls must be limited.

(89) Maximum ratio limits c/t_f given in Table 7.4 may be increased if the special connection details of the soles referred to in paragraph 7.6.2 are used.

Table 7.1 Maximum limits of the suppleness of the walls of steel sections of composite elements depending on the behaviour factor *q*

Ductility class of the structure	DCH	DCM
Behaviour factor q	$q \geq 3.50$	$1.50 < q < 3.50$
Soles of the partially embedded sections I or H (c/t_f)	9ε	14ε
Concrete-filled rectangular tube sections (h/t)	24ε	38ε
Concrete-filled circular tube sections (d/t)	$80\varepsilon^2$	$85\varepsilon^2$
Soles of sections I or H of the BAR elements (c/t_f)	23ε	35ε

Cores of sections I or H of BAR elements or partially embedded in concrete (d/t_w)	96ε	150ε
Rectangular tubes filled and embedded in concrete (h/t)	72ε	100ε
Circular tubes filled and embedded in concrete (d/t)	$150\varepsilon^2$	$180\varepsilon^2$
$\varepsilon=(235/f_y)^{0.5}$		

where:

c/t_f is the ratio between the width and the thickness of the sole wing;

d/t_w the ratio between the height and the thickness of the steel section core;

d/t the ratio of the maximum external dimension to the wall thickness of the pipe,

f_y the characteristic value of the steel flow limit (in N/mm^2).

ε correction factor to be determined by the relation:

$$\varepsilon=(235/f_y)^{0.5} \qquad (7.2)$$

7.6.2.2 Composite steel beams with reinforced concrete slabs

(90) Composite beams shall be designed considering a degree of total or partial connection according to the provisions of SR EN 1994-1-1. Minimum degree of connection η, defined according to EN 1994-1-1, is greater than 0,80 and the strength of the connectors for negative moment zones is greater than the tensile strength of the reinforcements.

(91) If non-ductile connectors are used, the full connection between the steel beam and the reinforced concrete slab shall be made.

(92) The strength of connectors calculated in accordance with SR EN 1944-1-1 shall be reduced by multiplying it by 0.75.

(93) To ensure ductility, in dissipative areas the relative height of the compressed area of the section of the composite beam shall be limited, x/h_b, so that the condition is met:

$$\frac{x}{h_b}<\frac{\varepsilon_{cu2}}{\varepsilon_{cu2}+\varepsilon_{ud}} \qquad (7.1)$$

where

ε_{cu2} the final specific deformation of the concrete taking into account the confinement, $\varepsilon_{cu2}=0,0045$;

ε_{ud} the design value of the ultimate specific deformation of the steel.

(94) The condition of ductility imposed by the relation (7.22) is considered fulfilled if the relative height of the compressed area of the section of the composite beam, x/h_b, is less than the values given in Table 7.5.

Table 7.1 Maximum values of the relative height of the compressed area of concrete, x/h_b ,to ensure ductility of composite steel beams with reinforced concrete slab

Ductility class	q	$f_y\ (N/mm^2)$	Maximum values x/h_b
DCM	$1{,}50<q\leq 4{,}00$	355	0.27
		235	0.36
DCH	$q>4{,}00$	355	0.20
		235	0.27

(95) In the case of structures in the DCH for which the coefficient of behaviour is less than 5.00, they shall be considered for the limits for buildings of class DCM.

7.6.2.3 Reinforced concrete composite beams with rigid reinforcement

(96) In dissipative structures, the areas at the extremities of the beams with the length of $l_{cr}=1{,}50\,h_b$ are considered critical areas, where h_b is the height of the cross-section of the beam, measured from the front of the pillars or areas of the same length on either side of a section of the beam field in which it touches $M_{pl,Rd}$ in seismic design combinations.

(97) Ensuring local ductility requirements in these areas shall be carried out fulfilling the conditions of 5.7.3.1.

(98) The longitudinal reinforcement complies with the provisions given in 5.7.3.1.1.

(99) The transverse reinforcement complies with the provisions given in 5.7.3.1.2.

7.6.3 Pillars

7.6.3.1 Reinforced concrete composite pillars with rigid reinforcement

(1) In dissipative composite structures, the areas at the ends of the pillars shall be designed as dissipative areas for which measures to ensure ductility are taken.

(2) The length of the critical zones of composite pillars shall be calculated with the following relations:

$$l_{cr}\geq maxim\left(1{,}5\,h_c;\,l_{cl}/6;\,600\,mm\right)\ \text{for DCH} \tag{7.1}$$

$$l_{cr}\geq maxim\left|h_c;\,l_{cl}/6;\,450\,mm\right|\ \text{for DCM} \tag{7.2}$$

where:

h_c height of the composite pillar section

l_{cl} free length of the pillar.

(3) If $l_{cl}/h_c<3$, the entire length of the pillar is considered critical.

(4) In the critical areas of the composite pillars, if the relation 7.5.2.1(7.2) is not fulfilled, in order to ensure sufficient plastic swivelling capability, a transverse confinement reinforcement shall be provided so as to achieve a sectional ductility at least equal to the sectional ductility obtained if the above condition is met.

(5) The longitudinal reinforcement complies with the provisions given in 5.7.3.2.1.

(6) The transverse reinforcement complies with the provisions given in 5.7.3.2.2.

(7) In dissipative areas, the diameter of the confinement clamps d_{bw} to prevent local buckling of the compressed sole, shall comply with the condition:

$$d_{bw} \geq \left(\frac{b_f t_f}{8}\right)\left[\left(\frac{f_{ydf}}{f_{ydw}}\right)\right]^{0.5} \tag{7.3}$$

where:

b_f, t_f the width and thickness of the sole;

f_{ydf}, f_{ydw} the design values for the flow limit of the sole steel and transverse reinforcement.

(8) In critical areas, the distance between two consecutive longitudinal bars connected to the corner by clamps or cramps shall be less than or equal to 200 mm for ductility class DCM and 150 mm for ductility class DCH.

(9) The structural provisions with regard to anchoring and joining the reinforcements installed in composite pillars shall be similar to those stipulated in Chapter 5 for reinforced concrete pillars.

(10) The concrete coating of the rigid reinforcement shall be equal to or greater than 75 mm for elements of class DCM and 100 mm for elements of class DCH.

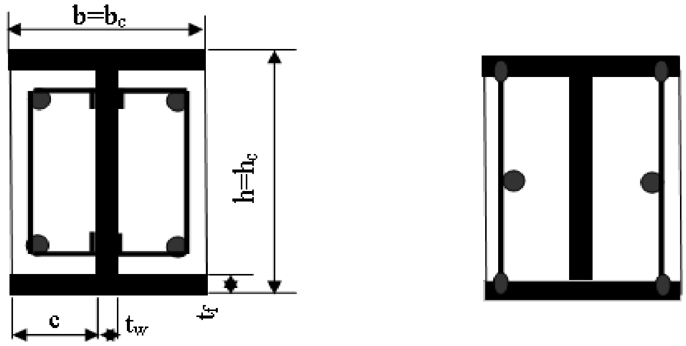
7.6.3.2 Composite pillars made of concrete-filled tubing

(11) The relation between the ductility class and the limit slenderness of the tube walls is given in Table 7.4.

7.6.3.3 Composite pillars with steel section partially embedded in reinforced concrete

(12) In critical areas of composite elements with steel section partially embedded in concrete, distances s between transverse reinforcements shall comply the conditions laid down in 5.7.3.2.2 and in SR EN 1994-1-1.

(13) In the case of pillars where additional reinforcements welded to the soles are provided to limit the local buckling to the distance s_l measured along the pillar, as depicted in Figure 7.2, b, where s_l meets the condition $s_l < 0{,}50\,c$, then the limits of the report c/t_f data in Table 7.4 can be increased by 50 %. If s_l meets the condition $0{,}50\,c \leq s_l < c$, the limits of the report c/t_f may be established by linear interpolation between the maximum values specified in Table 7.4 and those corresponding to the 50 % increase.



a) core welded clamps b) straight bars welded to the soles

Figure 7.1 Transverse reinforcement of composite elements that are partially embedded in concrete

(14) Diameter of additional reinforcements provided in accordance with (13) is greater than or equal to 8 mm and shall fulfil the condition 7.6.3.1(7.3).

(15) Additional reinforcements shall be welded to the soles at both ends, and the strength of the welds shall be greater than the design value of the tensile strength of the reinforcements. These reinforcements shall have a concrete coating between 20 mm and 40 mm.

7.6.4 Frame joints

(16) When designing composite joints consisting of composite steel beams with reinforced concrete slabs and composite or reinforced concrete pillars, the following measures shall be taken:

- vertical stiffeners are placed at the front of the pillar;
- the shear force in the beams is distributed between the additional vertical reinforcements welded to the beam base and the steel section of the pillar.

(17) When designing hybrid joints consisting of steel or composite beams and reinforced concrete pillars, the following composition conditions apply:

- the steel beam continuously passes through the joint;
- at the front of the pillar, vertical stiffeners shall be provided;
- in the vicinity of the vertical stiffeners, additional vertical reinforcements welded to the soles of the beam shall be provided in pillars having a design value of tensile strength greater than or equal to the design value of the shear force in the steel beam. Concrete in the area of these reinforcements shall be confined with transverse reinforcement that meets the conditions of paragraph 7.6.3.1.

(18) Hybrid joints consisting of reinforced concrete pillars and steel beams shall not be used for structures of classes DCH and DCM.

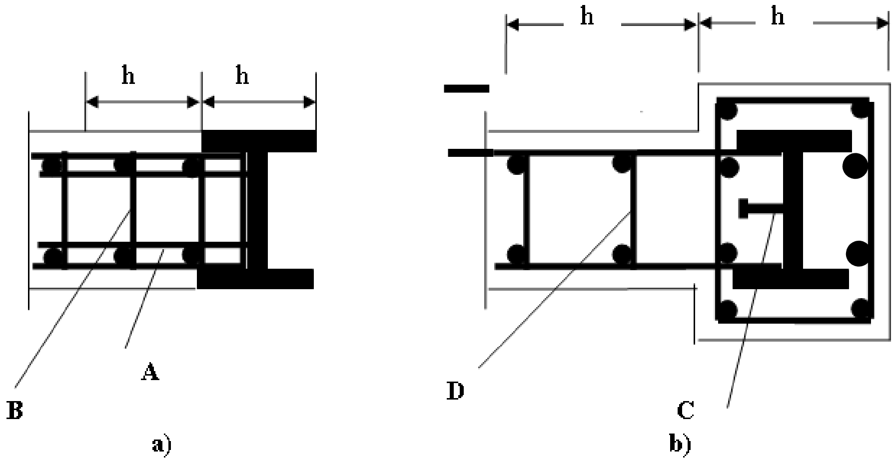
7.6.5 Composite walls

(19) When designing composition walls, the rules of composition and reinforcement given in technical regulation CR 2-1-1.1 shall be applied.

(20) Reinforced concrete panels of composite walls comply with the provisions of the construction and sizing of reinforced concrete walls given in Chapter 5.

(21) The areas at the ends of the section of the walls with a rigid reinforcement totally embedded in the concrete shall be designed according to the provisions of 7.6.3.1.

(22) Areas at the ends of the section of rigid reinforcement walls with partial embedding in concrete shall be designed according to the provisions of 7.6.3.3.



a) End element partially embedded in concrete, used in type 1 systems
b) End element fully embedded in concrete, used in type 1 systems
A = bars welded to the pillar B = transverse reinforcement
C = connectors D = cramps

Figure 7.1 Details for the end zones of composite walls

(23) The transfer of tangential efforts between the areas at the ends of the wall and the reinforced concrete panel of the wall core shall be carried out through connectors, through bars welded to the steel section or bars passed through the holes of the rigid reinforcement.

(24) Coupling collar beam of steel or composite with reinforced concrete slab have a sufficient length of incorporation into the reinforced concrete wall, capable of transmitting to the wall the moments and shear forces of the design of the coupling beam. Embedding length l_e shall be measured from the first reinforcement row of the end zones (Figure 7.4). The embedding length is greater than or equal to $1{,}50h$, where h is the height of the coupling beam.

(25) Vertical reinforcements welded to the soles of the beam with a tensile strength equal to the design value of the beam's shearing strength shall be placed in the wall in the area of the coupling beam. 2/3 of the area of this reinforcement shall be located in the first half of the encapsulation length. The reinforcement shall be extended symmetrically above and below the soles of the coupling beam with a length equal to the anchorage length. In this area, the transverse reinforcement shall comply with the conditions given in 7.6.3.2.

(26) In the case of the ductility class DCM, the confinement reinforcement of the end elements of the composite walls shall be disposed on a distance equal to h, and for the ductility class DCH this distance shall be extended along the length of the wall to $2h$, but at least for a distance of $0.15 l_w$. (h is the height of the section of the end element in the plane of the wall, see Figure 7.1, and l_w is the length of the wall).

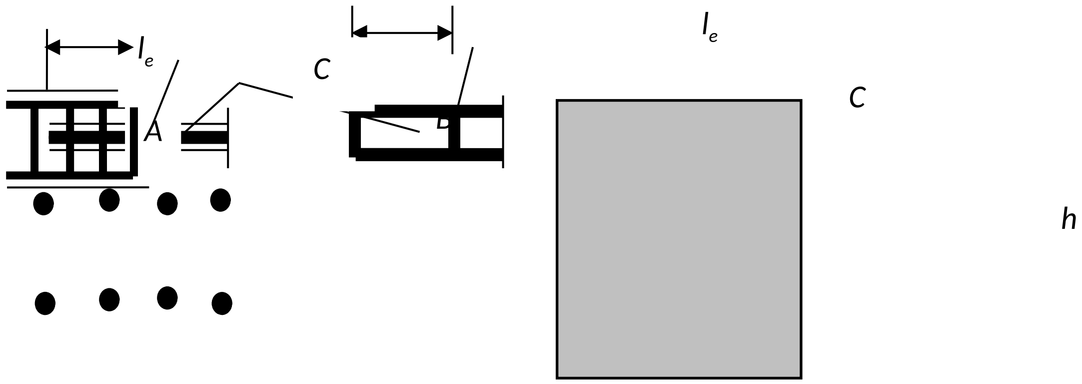
(27) The connection of the steel panel with the framing frame shall be carried out continuously with welding or screws.

(28) The minimum concrete embedding thickness of the steel panel shall be 200 mm, with at least 100 m on each side of the panel.

(29) The minimum percentage of reinforcement of the embedding concrete is equal to 0.25 %, in both directions.

(30) The steel panel shall be connected to the embedding concrete using welded connectors or cramps which pass through holes made in the steel panel.

Holes in the steel panel of the core of the composite wall shall be stiffened.



A=Additional wall reinforcement in the zone where the steel beam is embedded
B = Steel Coupling Beam C = Vertical stiffeners

Figure 7.2 Steel coupling beams used in reinforced concrete walls and embedding details for ductility class DCH

8 Masonry buildings

8.1 General information

8.1.1 Purpose and scope

(32) This chapter contains provisions for the seismic design of buildings in which the main structures contain walls made of masonry, which are main structural components, hereinafter referred to as masonry structures.

(33) For the design of buildings with masonry structure for actions other than seismic actions, the specific technical regulations and, where appropriate, the Romanian standards of the SR EN 1996-1 series shall be used.

8.1.2 Definitions

(34) The terms specific to this chapter are:

Lintel: a building element consisting of a beam of reinforced concrete, masonry, metal or wood placed above a void in a masonry wall to support the masonry section above it;

Belt: masonry confinement element arranged in a horizontal direction, predominantly applied to the stretch under the design seismic action, corresponding to the final limit state;

Beam: structural component, predominantly subjected to the bending moment and shear force, where the average normalized axial effort is less than 0.10, having the ratio between the free opening and the cross section height greater than 3.

Intersection between walls: a place where two or more walls are crossed; intersections are usually cross-shaped or in the shape of the letters ‘T’ or ‘L’.

Wall (structural wall): vertical structural component, made of masonry, having a ratio between the length of the cross-section and the thickness of its core greater than or equal to 4.

Wall coupled: wall, part of an assembly of vertical elements, to which it is connected by coupling collar beams, so that the axial force that develops in the wall, as a result of horizontal loads, ensures that at least 30 % of the rollover moment of the assembly, in the phase of plastic mechanism, is taken over in the direction of calculation.

Insulated wall: wall connected to the rest of the structure by horizontal elements, plates or beams, with rigidity and low bending strength.

Wall with post behaviour: wall with rigidity and strength capacity significantly higher than those of the horizontal structural components with which it is connected, so that the achievement of strength capacity is expected to occur only at its base.

Window recess-behavioural wall: wall of similar or less rigidity than that of the horizontal structural components with which it is connected, so that the bending strength is expected to be reached at the bottom or at the top of the wall.

Note: The window recess-behavioural wall can be considered as having the rotation locked at both ends, for seismic calculation in its plane.

Coupling collar beam: horizontal structural element, having the ratio between the free opening and the height of the cross section less than or equal to 3, rigidly connected at the ends with two vertical structural elements.

Riser: masonry confinement element arranged in a vertical direction, applied predominantly to stretching or compression at the design seismic action, corresponding to the ultimate limit state;

Structure of unreinforced masonry - simple: a masonry-walled structure which does not contain a sufficient quantity of reinforcement to be considered as reinforced masonry;

Reinforced masonry structure: a masonry-walled structure in which reinforcing bars or nets are embedded in mortar or concrete in such a way that all materials work together to withstand the effects of action;

Confined masonry structure: structure with masonry walls incorporating reinforced concrete or reinforced masonry confinement elements arranged in a vertical and horizontal direction.

Confined masonry structure with joint reinforcement: confined masonry structure where reinforcements are arranged in the horizontal joints of the masonry.

Masonry: assembly of masonry elements placed according to an established pattern and connected to each other by mortar.

Dentification masonry: confined masonry where the masonry elements at the edge of the pillar are not vertically aligned from one row to another, but offset by a minimum of 5 cm, so as to create concrete wedges for the masonry to work with the concrete in the risers.

8.2 Design principles

8.2.1 General information

(35) Buildings with masonry structure shall be designed in such a way as to obtain a spatial behaviour.

(36) The main structure of buildings with masonry structure is made up of floors and walls.

(37) Masonry walls that are main structural components shall be arranged aligned with two horizontal orthogonal directions; they connect to each other at intersections and to the floors adjacent to them.

(38) The normalized axial compression effort caused by non-seismic actions corresponding to the seismic design combination in masonry walls that are main structural components, calculated using the characteristic value of the masonry compressive strength, shall be limited according to the relation:

$$v_i = \frac{N_{Ed,i}}{A_{p,i} \cdot f_k} \leq 0{,}2 \qquad (8.1)$$

where

i the index of the masonry wall in the direction considered;

$N_{Ed,i}$ axial force in the wall i, caused by non-seismic actions corresponding to the seismic design combination;

f_k characteristic value of the compressive strength of the masonry;

$A_{p,i}$ horizontal section area of the wall i, corresponding to the axial force $N_{Ed,i}$.

(39) The normalized axial compression effort caused by non-seismic actions corresponding to the seismic design combination in masonry walls that are secondary structural components, calculated using the characteristic value of the masonry compressive strength, shall be upper limited to 0.40.

(40) Buildings with masonry structure shall be carried out in cumulative compliance with the following provisions:

(uuuuuuuuuuuuu) in multi-storey buildings, the vertical distance between the floors is less than or equal to 4.00 m;

(vvvvvvvvvvvvv) the diaphragms and their attachments to the masonry walls shall be such that they prevent the walls from moving in a direction perpendicular to their plane;

(wwwwwwwwwwwww) walls, beams, belts and/or diaphragms shall be connected to each other;

(xxxxxxxxxxxxx) beams, belts and/or diaphragms shall be made in such a way as to withstand the effects of the actions and to transmit the effects of the actions of the walls to which they are connected.

8.2.2 Ductility classes

(41) Masonry buildings shall be designed for the ductility class DCL or DCM.

(42) Only buildings with masonry structure enclosed with reinforced concrete elements and buildings with masonry structure enclosed with reinforced concrete elements and steel reinforcement in joints may be designed for the ductility class DCM, if the specific provisions of this Chapter are fully complied with.

(43) Buildings with unreinforced masonry structure shall be designed for ductility class DCL, subject to the specific provisions for this class of ductility given in this chapter.

(44) By way of exception from the provisions of (42) and (43), masonry buildings with other types of structures may be designed on the basis of specific technical regulations containing provisions on their seismic design in accordance with the provisions of Chapters 1-4 of this technical regulation, for the ductility class DCL or DCM.

8.2.3 Plastic mechanism

(45) For structures designed for the DCM ductility class, the favourable seismic response is conditioned by the formation of a plastic mechanism with optimal energy dissipation capacity induced by horizontal seismic action.

(46) The favourable plastic deformations of the main structural components occur due to the overcoming of the specific flow deformation of the stretched ductile reinforcements as a result of the bending of the structural components, with or without axial force.

(47) In structural systems with masonry walls, the favourable plastic mechanism is formed by the occurrence of favourable plastic deformations in the walls and at the ends of the coupling collar beams, if they exist and have a configuration that allows the development of favourable plastic deformations at their ends.

Note: Depending on the configuration of a wall, it may have a post or window recess behaviour; plastic joints may appear, at a certain level, at different rates in different walls.

Note: In many common configurations for masonry buildings, coupling collar beams have a reduced bending plastic deformation capacity.

(48) For the control of the plastic mechanism, the design shall be carried out in accordance with the principles of the method of hierarchy of strength capabilities - the design-to-capacity method.

(49) The hierarchy of the resistance capacities must ensure, at the incidence of seismic design action, the resistance capacities of the structural elements higher than the efforts that can produce breaks of fragile type, such as:

(yyyyyyyyyyyyy) breaking under shearing force in inclined sections;

(zzzzzzzzzzzzz) breaking at slipping forces, along joints or other pre-cracked sections;

(aaaaaaaaaaaaaa) breaking of the reinforcements anchorages.

(50) Infrastructures and foundations shall be made in such a way as to respond in the elastic domain to the design seismic action, corresponding to the ultimate limit state.

8.2.4 Behaviour factors

8.2.4.1 Ultimate limit state

(51) This paragraph contains provisions for establishing the maximum value of the behaviour factor, q, for horizontal seismic actions.

(52) Maximum values of the behaviour factor, q, shall be determined for each horizontal orthogonal direction of the building.

Note: Different values can be considered for the two horizontal orthogonal directions.

(53) For buildings designed for the ductility class DCL, the maximum value of the behaviour factor, q, is equal to:

(bbbbbbbbbbbbbb) 1.00, in the case of buildings with semi-rigid or flexible diaphragms;

(ccccccccccccc) 1.50, in the case of buildings with rigid diaphragms.

(54) For buildings designed for the ductility class DCM, the maximum value of the behaviour factor, q, is less than or equal to 3.50.

(55) For buildings designed in the ductility class DCM, on a regular basis in the horizontal and vertical planes, the maximum values of the behaviour factor, q, shall be chosen as set out in Table 8.1, depending on how the structure conforms to the seismic direction of action and the type of masonry.

Table 8.1 Maximum behaviour factor values, *q*

Compliance level of the structure	Type of masonry	q
There are at least 6 walls of different lengths in the direction of seismic action	Confined masonry	3.00
	Confined reinforced masonry in joints	3.50
There are no more than 5 walls of	Confined masonry	2.00
	Confined reinforced masonry in joints	2.50

(56) In the case of buildings which are irregular in the horizontal plane or in the vertical plane, the maximum value of the behaviour factor, q, shall be reduced in accordance with the provisions of 4.5.1.1.

(57) In application of the provision of (55), walls in one direction shall be considered to have different lengths if the length of any main wall in the direction of seismic action is less than 80 % of the length of the longest main wall in that direction. In this assessment, 20 % of the main walls in that direction that have the longest length shall be excluded.

> Note: for the application of this provision, the length of a wall means the length of the core of the horizontal section.

> Note: the number of walls to be excluded is obtained by relating 20 % to the total number of main structural walls in the direction of calculation and rounding up to the first integer.

(58) Maximum values of the behaviour factor given in Table 8.1 shall be reduced by 20 % if the total length of the cores of the longest 20 % of the main structural walls in one direction is greater than 40 % of the total length of the cores of the main structural walls in that direction.

(59) The maximum value of the behaviour factor resulting from the application of the provisions of this paragraph shall be limited to less than 1.00.

8.2.4.2 Service limit state

(60) The value of the behaviour factor for horizontal seismic actions for service limit state checks shall be 1.50 for buildings designed in ductility class DCM and 1.00 for buildings designed in ductility class DCL.

8.2.5 Modelling for calculation

(61) Establishing the efforts and deformations caused by seismic action in buildings with masonry structure shall be made by overall structural calculation, on three-dimensional models.

(62) By way of exception from (61), in the case of buildings with a masonry structure, the determination of the efforts and deformations caused by seismic action may be done on flat – unidirectional models, provided that all of the following conditions are met:

(ddddddddddddd) the building is classified in class III or IV of importance and exposure to earthquake;

(eeeeeeeeeeeee) the building has only 1 or 2 above-ground levels;

(fffffffffffff) horizontal diaphragms are made as rigid diaphragms;

(gggggggggggggg) masonry walls are aligned with two horizontal orthogonal directions, without exception, and have post behaviour.

(63) For buildings with masonry structure, when calculating the design value of seismic action, the fraction of the critical damping of the building, ξ, shall be considered equal to 8 %.

(64) The rigidity of the walls shall be assessed taking into account deformations in the bending, shearing and axial force.

(65) The rigidity of cracking masonry walls shall be taken to be equal to 50 % of the rigidity corresponding to the elastic, uncracked response.

(66) Horizontal structural components of masonry are considered in the calculation model only if they form with adjacent masonry walls a momentary connection in the direction of calculation.

(67) The modulus of elasticity of the masonry of a horizontal structural component shall be calculated on the basis of the characteristic value of the compressive strength of the masonry from the direction of calculation.

(68) The bending rigidity and shear force of a horizontal structural component which cracks can be considered equal to 25 % of the rigidity corresponding to the elastic response — uncracked, for unreinforced masonry, and 50 % of the rigidity corresponding to the elastic response — uncracked, for reinforced or confined masonry.

(69) By way of exception from (68), the bending rigidity and shear force of a horizontal structural component may be determined on the basis of the provisions of SR EN 1996-1-1.

(70) For reinforced confinement elements cracking, the rigidity shall be assumed to be equal to 50 % of the rigidity corresponding to the elastic — uncracked response.

(71) A compound horizontal element, made of masonry and reinforced concrete belts or lintels, can be modelled as two or more parallel elements, neglecting the collaboration between them.

(72) For diaphragms, the pattern must express their actual rigidity, except when the diaphragms are rigid. The criteria for classifying diaphragms in rigid, semi-rigid or flexible are given in 4.2.6.

(73) A monolithic reinforced concrete floor with a thickness of 100 mm or more, or a floor with prefabricated elements and over-concreting of 60 mm or more, may be considered rigid if it is continuously reinforced in both directions and on both sides with a stretched reinforcement coefficient of more than 0.002 and the reinforcements are anchored in the supporting belts or beams.

(74) Semi-rigid diaphragms shall be modelled minimally by finite surface elements with membrane behaviour or by compressed connecting rod-cross-ties systems.

(75) The contribution of flexible diaphragms to the overall structural response to seismic actions may be neglected in the calculation of the structure, except for their corresponding oscillating mass that is distributed to the supporting walls.

(76) For the calculation of the structure using the non-linear static calculation method, simplified constitutive laws customized for each structural component are used.

(77) For non-linear calculation, the walls can be modelled using a force-displacement, elastic-perfect plastic bilinear response law, in which the rigidity corresponds to the properties of the cracked section, and the floor area extends to θ_{CN}, where θ_{CN} is the relative displacement of the level near collapse. After θ_{CN}, the resistance to horizontal forces of the walls shall decrease to a residual value, which can be considered equal to 0. It can be assumed that the walls retain their capacity at axial force and after θ_{CN}.

(78) For non-linear calculation, the bending strength and shear force of horizontal elements in reinforced or confined masonry and reinforced concrete belts can be calculated by neglecting the effect of axial force. The relation of shear force-rope rotation can be considered elastic-perfectly plastic. Horizontal structural components and belts can be shaped with elastic-perfect plastic compressive behaviour, having an unlimited deformation capacity. When stretching, the horizontal structural components of non-wired masonry can be shaped with an elastic behaviour with brittle fracture; horizontal elements of reinforced or confined masonry and belts can be shaped with an elastic-perfect plastic behaviour with unlimited deformation capacity.

(79) For nonlinear calculation, plates and diaphragms can be modelled as having elastic behaviour.

8.3 Seismic performance criteria

8.3.1 General information

(80) The provisions of this section apply to the main structure, with a role in balancing the seismic action.

8.3.2 Resistance

(81) The walls shall be checked for effects both in their plane and perpendicular to the plane.

(82) Buildings with a masonry structure shall be constructed in such a way as to comply with the condition of resistance to horizontal action laid down in 4.3.2.1.

(83) All structural components shall be made such that the design value of the resistance capacity is greater than or equal to the design value of the effort in the section considered. This condition shall be fulfilled for all main structural components along their entire length.

(84) In the case of major seismic components subjected to bending, with or without axial force, and shear force, the following conditions shall be met:

$$N_{Rd} \geq N_{Ed} \tag{8.1}$$

$$M_{Rd} \geq M_{Ed} \tag{8.2}$$

$$V_{Rd} \geq V_{Ed} \tag{8.3}$$

where

N_{Rd} design value of the compressive strength capacity;

N_{Ed} design value of the axial compression force;

M_{Rd} design value of the bending strength;

M_{Ed} design value of the bending moment;

V_{Rd} design value of the strength capacity to the shear force;

V_{Ed} design value of the shear force.

(85) Partial safety coefficients for the calculation of masonry design resistances to the ultimate limit state for seismic grouping of loads shall be taken from Table 8.2

Table 8.1 Partial safety coefficients for calculating the design values of masonry resistance to the ultimate limit state, in seismic design combinations

Category of elements	Mortar	Type of inspection in execution		
		Low	Normal	Special
Category I	For general use, for prescription, prepared on site	2.40	2.20	1.90
	For general use, for prescription, industrial preparation, industrial semi-finished	2.20	1.90	1.80
	For general use or for thin, high-performance joints	Not allowed	1.80	1.80
Category II	For general use, for prescription, prepared on site	2.70	2.50	2.20
	For general use, for prescription, industrial preparation, industrial semi-finished	2.40	2.20	2.00

8.3.3 Ductility

(86) Buildings with masonry structure shall be constructed in such a way as to meet the conditions of ductility under horizontal seismic action given at 4.3.1.2.

(87) The design value of the permissible relative displacement level for checks at the ultimate limit state, for buildings with masonry walls, where at least one wall achieves shear force strength before reaching bending strength, is given in Table 8.3.

Table 8.1 Permissible values of relative level displacement at ultimate limit state — walls yielding to shear force

Type of masonry elements	Limit of relative level displacements		
	Unreinforced masonry	Confined masonry	Reinforced masonry
Blocks of burnt clay, group 1	0.007	0.009	0.010
Blocks of burnt clay, group 2	0.003	0.004	0.004
Autoclaved aerated concrete AAC, group 1	0.004	0.005	0.006

Other elements	0.002	0.002	0.003

(88) The design value of the relative displacement level allowed for checks at the ultimate limit state, for buildings with masonry structure where all the walls reach the bending strength before reaching the shearing strength capacity, is provided in Table 8.4.

Table 8.2 Permissible values of relative level displacement at final limit state — walls yielding to bending

Type of masonry elements	Limit of relative level displacements		
	Unreinforced masonry	Confined masonry	Reinforced masonry
Blocks of burnt clay, group 1	0.014	0.018	0.020
Blocks of burnt clay, group 2	0.006	0.008	0.008
Autoclaved aerated concrete AAC, group 1	0.008	0.010	0.012
Other elements	0.004	0.004	0.006

(89) By way of exception from (87) and (88), other admissible values may be considered if they are specified in a Romanian standard or in a technical approval.

(90) The deforming capacity of horizontal elements in unreinforced masonry in terms of rope rotation, for checks at the ultimate limit state, is provided in Table 8.5.

Table 8.3 Allowable values in terms of rope rotation – horizontal elements of unreinforced masonry

Type of masonry elements	Deformation limit
Blocks of burnt clay, group 1	0.028
Blocks of burnt clay, group 2	0.012
Autoclaved aerated concrete AAC, group 1	0.016
Other elements	0.008

(91) The value of the displacement gain factor, c, for checks at the ultimate limit state shall be equal to 1.50.

8.3.4 Stability

(92) Buildings with masonry structure shall be constructed in such a way as to meet the conditions of stability under seismic action given at 4.3.1.3.

8.3.5 Rigidity

(93) Buildings with masonry structure shall be constructed in such a way as to meet the conditions of rigidity under horizontal seismic action given at 4.3.2.

(94) The design value of the permissible relative level displacement for checks at the service limit state shall be equal to half of the design value of the permissible relative level displacement for checks at the ultimate limit state.

8.4 Stresses design values

8.4.1 Buildings designed for DCM ductility class

(95) The design value of an effort caused by seismic action is the maximum value of that effort that develops as a result of the incidence of seismic design action.

(96) The design effort values shall be determined by:

(hhhhhhhhhhhhh) transforming the efforts resulting from the calculation of the structure by a linear static calculation method, in order to quantify the non-linearity of the structural response caused by the design seismic action, in accordance with the principles of the resistance capacity hierarchy method;

or

(iiiiiiiiiiiii) directly, by non-linear calculation.

8.4.1.1 Walls

(97) The design value of the bending moments in the walls subjected to bending is determined with the relation:

$$M_{Ed} = M'_{Ed} \tag{8.1}$$

where

M_{Ed} design value of the bending moment;

M'_{Ed} bending moment resulting from the calculation of the structure in the seismic grouping.

(98) The design values of the shearing forces in the structural walls shall be determined with the relation:

$$V_{Ed} = \gamma_{Rd} \Omega V'_{Ed} \tag{8.2}$$

where:

V_{Ed} design value of the shear force;

V'_{Ed} shear force resulting from the calculation of the structure in the seismic grouping;

γ_{Rd} factor taking into account different sources of overstrength, $\gamma_{Rd} = 1{,}20$;

Ω wall bending overstrength factor in the calculation section

$$\Omega = \frac{M_{Rd}}{M_{Ed}} \tag{8.3}$$

M_{Ed} design value of the bending moment;

M_{Rd} the design value of the bending strength.

Product value $\gamma_{Rd} \Omega$ in the equation (8.2) shall be limited lower according to the relation:

$$1{,}50 \le \gamma_{Rd} \Omega \tag{8.4}$$

Product value $\gamma_{Rd}\Omega$ in the equation (8.2) shall be upper limited according to the relation:

$$\gamma_{Rd}\Omega \leq 1{,}50\,q \qquad (8.5)$$

(99) Design values of axial forces in a structural wall, N_{Ed}, shall be established on the basis of the balance of the wall at the complete formation of the plastic mechanism of the structure.

(100) By way of exception from (99), the design values of the axial forces in an isolated wall may be considered equal to the axial forces resulting from the calculation of the structure by a linear static calculation method in the seismic grouping.

8.4.1.2 Coupling beams and collar beams

(101) The design values of bending moments in coupling beams and collar beams where plastic joints are formed shall be considered equal to the values of bending moments resulting from the calculation of the structure in the seismic grouping.

$$M_{Ed} = M'_{Ed} \qquad (8.1)$$

(102) In the case of beams and coupling beams that respond elastically to the design seismic action, the design values of bending moments and shear forces shall be determined on the basis of the balance of efforts in the situation of complete formation of the plastic mechanism of the structure.

(103) In the case of coupling beams and collar beams that elasto-plastically respond to design seismic action, the design values of the shear forces shall be determined with the relation:

$$V_{Ed} = V'_{Ed,G} + q\,V'_{Ed,E} \qquad (8.2)$$

where:

$V'_{Ed,G}$ the design value of the shear force resulting from the calculation of the structure for actions other than seismic action included in the seismic grouping, corresponding to the ultimate limit state;

$V'_{Ed,E}$ the design value of the shear force resulting from the calculation of the structure for seismic action, included in the seismic grouping, corresponding to the ultimate limit state;

V_{Ed} design value of the shear force;

q the behaviour factor used to determine the design seismic force;

(104) By way of exception from (103), in the case of beams made of reinforced concrete that respond elasto-plastically to the design seismic action, the design values of the shear forces in the beams shall be determined from the beam balance in the case of the formation of the plastic mechanism, considering also the loads acting transversely on the axis of the beam in the seismic grouping. The values of the maximum bending moments that load the beam at the ends in the event of the formation of the plastic mechanism, $M^{\square}_{d,b}$, shall be calculated with the relation:

$$M^{\square}_{d,b} = \gamma_{Rd} M^{\square}_{Rd,b} \qquad (8.3)$$

$M_{Rd,b}^{\square}$ the design value of the bending capability of the beam, for the direction of rotation corresponding to the direction of action of the horizontal forces;

γ_{Rd} partial safety coefficient equal to 1.20.

(105) In the case of beams at which plastic joints develop at the ends, the design values of the shear forces shall be determined with the relations:

$$V_{Ed}=V'_{Ed,G}\pm\frac{M_{d,b}^{1}+M_{d,b}^{2}}{l_{cl}} \quad (8.4)$$

where:

$V'_{Ed,G}$ the design value of the shear force resulting from the calculation of the structure for actions other than seismic action, which are included in the seismic grouping, corresponding to the ultimate limit state;

l_{cl} free opening of the beam.

$M_{d,b}^{1}$ the maximum value of the bending moment that loads the beam at end 1, in the event of the formation of the plastic mechanism, corresponding to the direction of rotation corresponding to the seismic action;

$M_{d,b}^{2}$ the maximum value of the bending moment loading the beam at end 2, in the event of formation of the plastic mechanism, for the same direction of rotation as $M_{d,b}^{1}$.

(106) Design values of the axial forces in a coupling beam or collar beam, N_{Ed}, shall be established on the basis of its equilibrium in the state of plastic mechanism.

(107) By way of exception from (106), the design values of the axial forces in the coupling beams or collar beams may be taken to be zero.

8.4.1.3 Diaphragms

(108) The design values of the efforts in the diaphragms are to the efforts associated with the mobilisation of the overall plastic mechanism of the structure and shall take into account the imprecision of the calculation by multiplication by a partial safety coefficient equal to 1.20.

(109) The efforts in a diaphragm shall be determined by considering its equilibrium under the action of horizontal forces and the design values of the shear forces in the structural elements that load the diaphragm in the horizontal direction.

(110) By way of exception from (109), the design values of the efforts in the diaphragms, can be considered equal to the efforts resulting from the linear static calculation of the structure, considering the effects of seismic action multiplied by q.

8.4.1.4 Infrastructures and foundations

(111) The design values of the efforts and deformations in the infrastructure elements are obtained by considering the field-structure interaction.

(112) The design values of the efforts and deformations in the infrastructure elements are obtained considering their balance under the efforts related to the superstructure and the support efforts on the ground.

(113) When designing the infrastructure and foundations, the maximum values of the efforts related to the superstructure, corresponding to the situation of the formation of the plastic mechanism (also taking into account the partial safety coefficients that quantify the uncertainties in the model for calculating the resistance capacities) and the loads that act directly on them shall be considered.

(114) In the case of masonry structural walls, the design values of the efforts at their base, at the level of the conventional fixation-clamping section, E_{Fd}, shall be determined by transforming the effort values resulting from the linear static calculation with the equation:

$$E_{Fd} = E_{F,G} + \gamma_{Rd} \Omega E_{F,E} \tag{8.1}$$

where:

$E_{F,G}$ the sectional effort produced by actions other than seismic action, which are included in the seismic grouping;

$E_{F,E}$ the sectional effort resulting from the calculation of the design seismic action, corresponding to the ultimate limit state;

γ_{Rd} factor taking into account different sources of overstrength, $\gamma_{Rd} = 1{,}20$;

Ω bending overstrength factor of the wall.

(115) Product value $\gamma_{Rd} \Omega$ in the equation (8.1) shall be limited upwards according to the relation:

$$\gamma_{Rd} \Omega \leq 1{,}50\, q \tag{8.2}$$

8.4.2 Buildings designed for DCL ductility class

(116) The design effort values shall be established on the basis of the effort resulting from the calculation of the structure by a linear calculation method.

(117) The design values of the bending moments in the walls are equal to those resulting from the calculation of the structure by a linear calculation method.

(118) The design values of bending moments and shear forces in coupling beams and collar beams are equal to those resulting from the calculation of the structure by a linear calculation method.

(119) The design values of the shear forces in the walls shall be equal to the shear forces resulting from the calculation of the structure by a linear calculation method, in the seismic grouping, multiplied by 1.20:

$$V_{Ed} = 1{,}20\, V'_{Ed} \tag{8.1}$$

where:

V_{Ed} design value of the shear force;

V'_{Ed} the value of the shear force resulting from the calculation of the structure in the seismic grouping.

(120) The design values of the diaphragm loads, constituted by the slabs subjected to loads parallel to their median plane, are equal to the efforts resulting from the linear

static calculation of the structure, considering the effects of seismic action multiplied by 1.20.

(121) The design values of the efforts in infrastructure and foundations are equal to the efforts resulting from the linear static calculation, considering the effects of seismic action multiplied by 1.20.

8.5 Resistance capacity

(122) The calculation of the strength capacities of walls and coupling collar beams that include elements for masonry shall be made on the basis of the specific provisions of technical regulation CR 6, on the basis of other specific technical regulations or, in the absence of such provisions, on the basis of the provisions of the Romanian standard SR EN 1996-1-1.

(123) The calculation of the resistance capacities of the reinforced concrete elements, other than the confinement elements, shall be made on the basis of the specific provisions of the Romanian standard SR EN 1992-1-1.

(124) In addition to (2), in the case of reinforced concrete elements in which plastic joints are formed, the additional provisions of Chapter 5 for elements of the ductility class DCM shall be complied with.

8.6 Structure

(125) This paragraph contains minimum composition provisions for masonry structures.

(126) The provisions of this paragraph applies to buildings designed for the ductility class DCL and those designed for the ductility class DCM, except where the ductility class to which the provision refers is explicitly mentioned.

8.6.1 Materials

8.6.1.1 Masonry elements

(127) When making masonry structural components for structures designed for the ductility class DCM, masonry elements with standardised compressive strength shall be used, f_b, determined in accordance with SR EN 772-1, which cumulatively fulfils the conditions:

	$f_{bv} \geq 5\,N/mm^2$	(8.1)
	$f_{bh} \geq maxim\,(0{,}1 f_{bv}; 1{,}5\,N/mm^2)$	(8.2)

where

f_{bv} the value of resistance to normal stresses on the horizontal joint;

f_{bh} the value of resistance to stresses parallel to the horizontal joint, in the plane of the wall;

(128) When making masonry structural components for structures designed for the ductility class DCL, masonry elements with standardised compressive strength shall be used, f_b, determined in accordance with SR EN 772-1, which cumulatively fulfils the conditions:

	$f_{bv} \geq 3\,N/mm^2$	(8.3)
	$f_{bh} \geq maxim\,(0{,}1\,f_{bv};1{,}5\,N/mm^2)$	(8.4)

(129) In areas with moderate or high seismicity, only masonry elements of category I shall be used, according to the classification in technical regulation CR 6.

(130) In the case of buildings designed for the ductility class DCM, masonry elements that meet all of the following conditions shall be used:

(jjjjjjjjjjjjj) are fired clay elements meeting the performance requirements of SR EN 771-1 or autoclaved aerated concrete elements meeting the performance requirements of SR EN 771-4;

(kkkkkkkkkkkkk) are masonry elements of category I;

(lllllllllllll) are elements that allow complete filling with mortar of horizontal and vertical joints between them;

(mmmmmmmmmmmmm) are elements classified in groups 1 or 2, according to the provisions of SR EN 1996-1-1, which satisfy the requirements of Table 8.6.

Table 8.1 Properties of masonry elements

Property	Group 1	Group 2	
	Seismic area:		
	Low, moderate or high	Low	Moderate or high
Total volume of gaps (% of gross volume)	≤ 25 %	≤ 55 %	≤ 45 %
Volume of each void (% of gross volume)	≤ 12.5 %	≤ 12.5 %	≤ 12.5 %
Declared value of external wall thickness (mm)	-	≥ 8	≥ 12
Declared value of internal wall thickness (mm)	-	≥ 5	≥ 10
Area of a single void (mm^2)	-	-	≤ 1,200
Continuous inner vertical walls along the entire length of the element	-	-	Yes

8.6.1.2 Mortars

(131) When making masonry structural components in buildings designed for the ductility class DCM, mortars with compressive strength shall be used, established according to SR EN 1015-11, which meets the condition:

$$f_m \geq 5\,N/mm^2 \quad (8.1)$$

where f_m is the mean value of the compressive strength of the mortar.

(132) When making masonry structural components in buildings designed for the ductility class DCL, mortars with compressive strength shall be used, established according to SR EN 1015-11, that meet the condition:

$$f_m \geq 2{,}5\ N/mm^2 \quad (8.2)$$

(133) When building masonry structures , general purpose mortars, industrial preparations, industrial or performance semi-finished products shall be used.

(134) By way of exception from (133), general-purpose mortar prepared at the site may be used in buildings in classes III and IV of importance and exposure to earthquakes with a total above-ground height less than or equal to 8.00 m, located in areas of low seismicity.

(135) By way of exception from (133), thin joint mortar may be used for buildings designed for the ductility class DCL with 1 or 2 levels, including the attic, if any, with total above-ground height less than or equal to 8.00 m.

(136) For buildings designed in the ductility class DCM, masonry mortars shall comply with the provisions of SR EN 998-2.

8.6.1.3 Masonry

(137) When making masonry structural components of buildings designed for the ductility class DCM, masonry with the strength capacities established according to the provisions of SR EN 1052-1, SR EN 1052-2 and/or SR EN 1052-3 above or equal to the minimum values indicated in Table 8.7 shall be used.

Table 8.1 Minimum strengths of masonry of the main structural components

Minimum allowable masonry strength (N/mm^2)	Class of importance and earthquake exposure of the building	
	III or IV	I or II
Characteristic compression resistance perpendicular to the joints, f_k	2.50	3.00
Characteristic compressive strength parallel to the joints, f_{Kh}	0.65	0.80
Initial characteristic shear strength, f_{vk0}	0.20	0.25

8.6.1.4 Concrete

(138) The characteristic value of the compressive strength of concrete in the main structural components meets the condition:

$$f_{ck} \geq 20\ N/mm^2 \quad (8.1)$$

(139) When choosing the quality of concrete, the specific durability requirements given in the specific technical regulations shall also be taken into account.

(140) Concrete elements shall be carried out in compliance with the provisions of technical regulations NE 012/1 and NE 012/2.

8.6.1.5 Reinforcements

(141) When making the main structural components of buildings designed for ductility class DCM, steel reinforcements of ductility class B or C shall be used, according to SR EN 1992-1-1.

(142) In the case of the use of steel reinforcements for the construction of structures designed for the ductility class DCL, steel meeting the quality requirements of SR EN 1992-1-1 shall be used.

(143) When carrying out the main structural components under the conditions specified in (1) and (2) only profiled steel bars shall be used.

(144) By way of exception from (1), (2) or (3), other types of reinforcements may be used in the construction of the main structural components of buildings designed for the ductility class DCL only on the basis of specific technical regulations containing provisions on the seismic design of structures for buildings, in accordance with the basic requirements of seismic design given in this technical regulation, or on the basis of technical approvals drawn up in accordance with 1.1, (14).

8.6.2 General composition of structures

(145) The number of above-ground levels above the conventional fixation-clamping section shall be limited in accordance with the provisions of this paragraph, in accordance with the type of structural system used, the seismicity of the site and the density of the walls.

(146) For the application of these provisions, the attic or circulating bridge shall be considered as an above-ground level.

(147) For buildings located in areas with low seismicity, the maximum permitted number of above-ground levels shall be 5.

(148) For buildings located in areas with moderate or high seismicity, the maximum permitted number of above-ground levels shall be equal to 4.

(149) The number of levels shall be limited in accordance with the provisions of Table 8.8:

Table 8.1 Provisions on the composition of masonry structures

Number of levels	Seismic activity	Minimum density of structural walls ($p\%$)		Type of structural system allowed
		at the first Level (Building base)	at the upper levels with the masonry structure	
1	Low	3.0%	Not applicable	Any type
	Moderate	4.0%	Not applicable	Any type

	High	4.5%	Not applicable	Any type
2	Low	3.0%	3.0%	Any type
	Moderate	4.0%	4.0%	Any type
	High	4.5%	4.5%	Confined masonry or reinforced masonry
3	Low	4.0%	3.0%	Confined masonry or reinforced masonry
	Moderate	5.0%	4.0%	Confined masonry or reinforced masonry
	High	5.5%	4.5%	Confined reinforced masonry in joints or reinforced masonry
4	Low	5.0%	4.0%	Confined masonry or reinforced masonry
	Moderate	6.0%	5.0%	Confined reinforced masonry in joints or reinforced masonry
	High	7.0%	5.5%	Confined reinforced masonry in joints or reinforced masonry
5	Low	6.0%	5.0%	Confined reinforced masonry in joints or reinforced masonry

(150) The density of structural walls is determined with the relation:

$$p = 100\frac{A_{min}}{S} \quad (8.2)$$

where

p density of structural walls;

A_{min} the minimum area of the cores of the structural walls at a level, which are the main structural components, corresponding to the two horizontal orthogonal directions, which is determined with the relation:

$$A_{min} = min\left(A_x; A_y\right)$$

A_x sum of the horizontal areas of the cores of the structural walls, which are the main structural components, arranged parallel to the direction *ox*;

A_y sum of the horizontal areas of the cores of the structural walls, which are the main structural components, arranged parallel to the direction *oy*;

S the surface of the floor above the level considered.

(151) In applying the relation (8.2), at the level considered, the sum of the horizontal areas of the cores of the structural walls shall be determined, for each direction, in the most unfavourable horizontal section, corresponding to the minimum value of the sum.

(152) In the case of buildings designed in the ductility class DCM, walls which are not parallel to the main directions shall be considered as secondary structural elements and their area shall not be considered for the application of the provision (149).

(153) In the case of buildings designed in the ductility class DCL, located in areas with low or moderate seismicity, in application of the provision (149), for walls not parallel to the main directions, the area corresponding to the product of the thickness of the wall core and the length of its projection in the main direction considered shall be considered.

(154) In the case of buildings designed in the ductility class DCM, the distance between structural walls measured in a horizontal direction perpendicular to their plane shall be upper limited to 8.00 m. This condition shall apply to each main horizontal direction of the building.

(155) In the case of buildings designed in the ductility class DCM, the area of a floor area bounded by masonry walls constituting main structural components shall be limited to a maximum of 40.0 m^2.

(156) In the case of unreinforced masonry structures, pillars shall be arranged on the perimeter of the building at all intersections between the masonry walls that constitute the main structural components.

8.6.3 Walls

(157) The thickness of structural walls shall cumulatively meet the conditions:

$$t \geq 240\,mm \tag{8.1}$$

$$t \geq \frac{h_s}{15} \tag{8.2}$$

where:

t the thickness of a structural wall;

h_s level height.

(158) For all structural walls, only one type of masonry elements shall be used.

(159) The walls shall be made with woven masonry, without continuous vertical joints on several rows of bricks.

(160) When making the walls of buildings designed for the ductility class DCM, the vertical and horizontal joints of the masonry shall be completely filled with mortar.

8.6.4 Containment elements from reinforced concrete

(161) Confining masonry shall be achieved by the arrangement of reinforced concrete pillars and reinforced concrete belts, connected to each other.

(162) At the level of the conventional fixation-clamping section, the longitudinal reinforcement in the pillars shall be anchored in the foundations or in the main structural components of the infrastructure, as appropriate.

(163) Concrete from confinement elements, regardless of their type, shall be poured after making masonry.

(164) Effective collaboration between pillars and masonry shall be carried out:

(nnnnnnnnnnnnnn) by dentifications made at the masonry interface – vertical confinement element

and/or

(oooooooooooooo) through reinforcements arranged in the horizontal joints of the masonry, placed at vertical distances of not more than 3 layers and adequately anchored.

(165) The pillars shall be arranged continuously, on the entire height of the wall.

(166) In the case of confined masonry, with or without reinforcement in joints, minimal pillars shall be arranged in the following positions:

(pppppppppppppp) at the intersections between the masonry walls constituting the main structural components;

(qqqqqqqqqqqqqq) at the free ends of the walls;

(rrrrrrrrrrrrrr) at both ends of gaps in the wall with an area greater than 2.5 m^2 , for buildings located in areas with low or moderate seismicity, or 1.5 m^2 for buildings located in areas with high seismicity.

(ssssssssssssss) in the wall field in such a way that the distances between the vertical axes of the pillars measured along the wall are less than or equal to 5.00 m.

(167) By way of exception 8.6.4, (166), (rrrrrrrrrrrrrr), the arrangement of the pillar on one of the edges of the void may be omitted if the adjacent wall section is enclosed by belts and a reinforced concrete pillar located at the intersection of two walls not more than 750 mm from the edge of the void.

(168) Regardless of the type of structural system, at the level of each floor, continuous belts shall be made, along the entire length of the structural walls.

(169) Irrespective of the type of structural system, if the height of a wall is equal to or greater than 3.50 m, including the size of the upper belt, intermediate belts shall be fitted throughout its length in such a way that the distance between the centres of the belts is equal to or less than 3.50 m and equal to or greater than 1.30 m.

(170) Regardless of the type of structural system, if there are walls above the last floor, they will be fitted with belts at the top. This provision also refers to gables, attics or interior walls in the attic.

(171) Intermediate belts can be made with open contours, with interruptions next to some gaps in the walls, provided that pillars are made at both ends of the respective gaps.

(172) By way of exception from (168), in areas with low seismicity, the floor belts may be interrupted near the gaps in the masonry walls of the staircase, if they are lined with pillars at both ends and if a belt is made at the intermediate floor level between the two pillars.

8.6.4.1 Pillars

(173) The pillars shall be made of reinforced concrete with a cross-section with the size of any side greater than or equal to 240 mm.

(174) Longitudinal reinforcement shall be carried out with steel bars having a diameter of 12 mm or more.

(175) The minimum total percentage of longitudinal reinforcement is:

(ttttttttttttt) 1.00 % for buildings located in areas with high seismicity;

(uuuuuuuuuuuuuu) 0.80 % for buildings located in areas with moderate seismicity;

(vvvvvvvvvvvvvv) 0.60 % for buildings located in areas with low seismicity.

(176) The joining and anchoring of the longitudinal reinforcements shall be made according to the provisions of SR EN 1992-1-1, considering a tensile effort equal to:

(wwwwwwwwwwwwww) $1{,}20 f_{yd}$, in critical areas of walls;

(xxxxxxxxxxxxxx) f_{yd}, in current areas;

where f_{yd} is the design value of the steel flow limit.

(177) The overlapping length of the longitudinal reinforcements shall be taken to be greater than or equal to:

(yyyyyyyyyyyyyy) 60φ in the vicinity of the conventional fixation-clamping section of the building;

(zzzzzzzzzzzzzz) 50φ as to the remainder;

where φ is the minimum diameter of the overlapping reinforcement.

(178) Longitudinal reinforcements shall be anchored in the belts at the top of the walls.

(179) Cross reinforcement shall be done with closed clamps made of steel bars with a diameter of 6 mm or more.

(180) The distance between the clamps shall be limited upwards to 150 mm.

(181) At the ends of the pillars at each level, for a length greater than or equal to 600 mm from the end sections, the distance between the clamps shall be limited upwards to 100 mm.

(182) In areas where longitudinal reinforcements overlap, the distance between the clamps shall be limited to a maximum of 100 mm, in compliance with the specific provisions of SR EN 1992-1-1.

8.6.4.2 Belts

(183) Belts shall be made of reinforced concrete in such a way that the cross-sectional area is equal to or greater than 500 cm^2.

(184) The cross-section width of a belt shall meet the conditions:

$$b_w \geq 240\,mm \tag{8.1}$$

$$b_w \geq \frac{2}{3} t \qquad (8.2)$$

where

b_w belt cross-section width;

t the width of the wall on which the belt rests.

(185) The height of the cross-section of a belt from the floor or intermediate landing of a staircase shall meet the condition:

$$h_w \geq 250\, mm \qquad (8.3)$$

(186) Longitudinal reinforcement shall be done with steel bars of diameter greater than or equal to 12 mm.

(187) The fixing and anchoring of the longitudinal reinforcements shall be made according to the provisions of SR EN 1992-1-1, considering that the tensile effort is equal to the design value of the flow limit of the steel, f_{yd}.

(188) The overlapping length of the longitudinal reinforcements shall be taken to be greater than or equal to 60φ, where φ is the minimum diameter of the reinforcement being joint by overlapping.

(189) At the intersections between the walls, the continuity of the belts located on the two directions shall be ensured. Continuity of belts shall be achieved by anchoring longitudinal bars in belts perpendicular to a length of at least 60φ.

(190) The minimum total percentage of longitudinal reinforcement is:

(aaaaaaaaaaaaaaa) 1.00 % for buildings located in areas with high seismicity;

(bbbbbbbbbbbbbbb) 0.80 % for buildings located in areas with moderate seismicity;

(ccccccccccccccc) 0.60 % for buildings located in areas with low seismicity.

(191) Cross reinforcement shall be done with closed clamps made of steel bars with a diameter of 6 mm or more.

(192) The distance between the clamps shall be limited upwards to 150 mm.

(193) At the ends of the belts, for a length greater than or equal to 600 mm from the end sections, the distance between the clamps shall be upper limited to 100 mm.

(194) In areas where longitudinal reinforcements overlap, the distance between the clamps shall be limited to a maximum of 100 mm, in compliance with the specific provisions of SR EN 1992-1-1.

8.6.5 Lintels

(195) Lintels shall be made in such a way as to lean on pillars or walls made of masonry. When leaning against masonry, a leaning length greater than or equal to 300 mm shall be provided.

(196) In buildings designed for the DCM ductility class, gap lintels with a free opening greater than 2.0 m shall be made of monolithic cast reinforced concrete.

(197) By way of exception from (196), lintels with other compositions may be used, on the basis of specific technical regulations containing provisions on the seismic

design of structures for buildings, in accordance with the basic requirements of seismic design given in this technical regulation.

(198) Prefabricated lintels shall not be used if they rest on pillars that are disposed according to the provisions 8.6.4, (166) or 8.6.2, (156).

(199) The cross-section height of a reinforced concrete lintel shall meet the condition:

$$h_w \geq \frac{1}{12} l_{cl} \tag{8.1}$$

where

h_w the height of the cross-section of the lintel;

l_{cl} free opening of the void.

(200) By determining the height of the level and the proportions of the arrangement of the belts, the remaining free space between the upper part of the lintel and the lower part of the belt located above it shall be completed with a whole number of masonry layers.

8.6.6 Reinforcement of masonry in horizontal joints

(201) This paragraph refers to reinforced masonry walls in its horizontal joints.

(202) In buildings designed for class DCM, located in areas of moderate or high seismicity, and in buildings designed for class DCL, located in areas of high seismicity, steel reinforcements shall be provided at the intersections of structural walls. A minimum of two bars with a diameter of 8 mm or more shall be provided at a vertical distance less than or equal to the size of three masonry sieves. These reinforcements shall extend in the masonry for a minimum length of 1.00 m from the front of the perpendicular wall.

(203) In buildings designed for class DCM located in areas of moderate or high seismicity, and in buildings designed for class DCL located in areas of high seismicity, the horizontal reinforcement indicated in (202) shall also be provided in the parapet areas situated under the window voids with a width of more than 1.00 m. These reinforcements shall be extended in the masonry for a length of at least 1.00 m on either side of the edges of the void or shall be anchored in the reinforced concrete pillars adjacent to the void, if any.

(204) In the case of the use of steel reinforcements, if the condition of resistance to shear force requires joint reinforcements, a minimum of two bars with a diameter of 8 mm or more shall be provided. The vertical distance between the reinforced joints shall be less than or equal to:

(dddddddddddddd) the size of two layers, in the case of masonry elements with heights equal to or greater than 188 mm;

(eeeeeeeeeeeeee) the size of three layers, in the case of masonry elements with heights of less than 188 mm.

(205) The horizontal mortar coating of steel joint reinforcements shall be chosen in such a way as to ensure their corrosion protection.

Note: For common mortars, a 4 cm coating generally provides anti-corrosion protection of joint reinforcements

(206) Horizontal reinforcements used to ensure the ability to withstand shear force shall be anchored to capacity in the vertical confinement elements at the ends of the wall.

8.6.7 Floors

(207) In buildings classified as DCM, the floors shall be made as rigid diaphragms, in accordance with the provisions of 4.2.6.

(208) In the case of buildings with underground levels, the floor above them shall be made of reinforced concrete.

8.6.8 Infrastructure and foundations

(209) Reinforced concrete foundations shall be used in buildings designed for the ductility class DCM.

(210) In buildings with surface foundations made of reinforced concrete, the following foundation solutions shall be used:

(fffffffffffffff) foundation beams;

(ggggggggggggggg) continuous soles;

(hhhhhhhhhhhhhhh) general foundation frame;

(iiiiiiiiiiiiiii) isolated foundations connected to each other by beams.

(211) Propping masonry walls on foundation elements and/or infrastructure elements shall be carried out continuously, along their entire length.

(212) In the case of underground levels of buildings, perimeter walls and floors shall be made of reinforced concrete. Reinforced concrete walls of underground levels shall be carried out in accordance with the provisions of Chapter 5.

(213) The bases of the perimeter walls in masonry shall be made of reinforced concrete.

9 Timber structures

9.1 General information

9.1.1 Purpose and scope

(214) This chapter contains provisions on the seismic design of buildings with a structure made of solid wood, glued laminated wood, cross-laminated wood and wooden panels, joined with adhesives, by sealing and/or with metallic jointing elements, hereinafter referred to as wooden buildings.

(215) When designing timber structures, the provisions of technical regulation NP 005 apply together with the additional provisions given in this chapter.

9.1.2 Definitions

(216) The following terms are used in this chapter:

Semi-rigid joints: Joints with significant flexibility, the influence of which must be considered in the structural calculation, for which displacements are blocked and rotations are limited, depending on the characteristics of the components.

Rigid joints: Joints with negligible flexibility that possess rigidity when rotating the joint, the displacements and rotations of the elements in the joint being blocked. (e.g. glued joints).

Articulated joints: Joints made in such a way that they allow free rotation between the joining elements.

Rodded joints: Joints with metal elements required perpendicular to their axis or required when plucking. In some situations the rods can be made of wood.

Sealing joints: joints between two elements where the efforts are transferred through the contact area. The metal joining elements have the role of keeping the joined parts in place without participating in the transmission of the efforts.

Adhesive joints: joints to ensure continuity by extending wooden elements or for joining two or more elements and forming an element that behaves like a single structure, using a special adhesive.

Ductility: the plastic deformation capacity of a structure without significant reduction in rigidity and strength.

Location: the location in the territory of an activity, by specifying a portion of land to be organized spatially, corresponding to a certain functionality.

Building: above-ground and, where appropriate, underground construction, with rooms for sheltering people, materials, etc.

Natural wood: unprocessed wood elements such as logs, balls, handles or pillars, etc.

Solid wood: wood elements processed in various assortments such as planks, cabinets, beams, etc.

Glued laminated wood: wooden structural element made by successively gluing two or more wooden slats under well-defined temperature and pressure conditions.

Cross-laminated wood: structural wood product composed of at least three layers, of which at least three are orthogonally bonded, comprising layers of wood or layers of wood-based panels – CLT.

Wooden panels - structural panels type bone frame from square wooden battens and collaborative wooden structural board, fixed with nails or screws to wooden battens.

Laminated veneer lumber (LVL): product made by gluing together several veneer sheets laid with the fibres oriented in the same direction.

Double oriented fibre boards: wood-based panel consisting of several layers of wood fibres of different shape and thickness, cross-oriented, glued together with synthetic and hot-pressed adhesives - OSB.

Wood chipboard - wood-based panel manufactured under pressure and heat from wood particles (wood flakes, chips, sawdust and the like) with the addition of glue - PAL.

Fibreboard: wood-based panel made from wood fibres mixed with binders based on formaldehyde resins and hot-pressed - PFL.

Structural system of cross-laminated wooden walls: structural system in which the walls and horizontal diaphragms of cross-laminated wood make up the take-over system of vertical and horizontal forces.

Structural wall system made of wood panels: structural system in which the wooden panels and horizontal diaphragms make up the take-up system of vertical and horizontal forces.

Structural system of beam walls: structural system in which the walls of overlapping beams take over the vertical and horizontal loads. The beam walls can be made of round, semi-round or squared wood. Floor diaphragms are made of wood.

Wooden spatial frame structural system: structural system in which vertical as well as horizontal loads are taken over by spatial frames.

Structural system of braced frame type: wooden frame-type structural system with articulated joints with linear elements or surface elements designed to ensure the spatial stability of the construction and to take over horizontal loads.

Cantilever structural system: structural system consisting of vertical consoles (walls, pillars) with the preservation of the continuity of the wooden walls throughout the height of the building.

Arched dome structural system with two or three joints: structural wooden system in which the vertical and horizontal loads are taken over by the spatial arches that constitute an undeformable geometrical system in its plane. Geometric deformity in the longitudinal direction is achieved by a bracing system consisting of linear elements or surface elements.

Arched dome structural system: structural wooden system where vertical and horizontal loads are taken over by the constituent wooden arches.

9.2 Design strategies

9.2.1 Types of structures

(217) Wooden buildings designed for seismic actions shall be made with the main structural system of the type:

(jjjjjjjjjjjjjjj) structural system of cross-laminated wood walls;

(kkkkkkkkkkkkkkk) structural system of wood-panel walls;

(lllllllllllllll) structural system of beam walls;

(mmmmmmmmmmmmmmm) wooden spatial frame structural system;

(nnnnnnnnnnnnnnn) structural system of braced frame type;

(ooooooooooooooo) cantilever-type structural system;

(ppppppppppppppp) dome-type structural system with two- or three-joint arches;

(qqqqqqqqqqqqqqq) dome-type structural system with arches.

9.2.2 Ductility classes

(218) The main seismic structures made of wood shall be seismically designed for:

(rrrrrrrrrrrrrrr) high or medium dissipative behaviour

(sssssssssssssss) poorly dissipative behaviour.

(219) Wooden structures shall be made in such a way that plastic deformations occur:

(ttttttttttttttt) in joints, if they are made with metal elements;

(uuuuuuuuuuuuuuu) in addition, if specially designed energy dissipation systems are used.

(220) Structural components made of wood shall be made in such a way as to respond elastically to the design seismic action, corresponding to the ultimate limit state.

(221) Joints not made with metallic elements and joints made with axially loaded metal rods shall be made in such a way as to respond elastically to the seismic design action, corresponding to the ultimate limit state.

(222) Structures with high or medium dissipative behaviour are designed to respond elasto-plastic to design seismic action, plastic deformations being directed to dissipative areas. In this approach, buildings shall be designed for the ductility class DCM or DCH, subject to the provisions specific to those ductility classes given in this chapter.

(223) Structures with poorly dissipative behaviour are designed to respond elastically to the seismic action of the design, without producing significant incursions of steel into the plastic field. These structures shall be designed for the ductility class DCL, subject to the provisions specific to this ductility class given in this chapter.

(224) Wooden buildings having:

(vvvvvvvvvvvvvvv) structural system of cross-laminated wooden walls,

(wwwwwwwwwwwwwww) structural wall system made of wood panels, or

(xxxxxxxxxxxxxxx) framework structural system;

shall be seismically designed for ductility class DCL, DCM or DCH.

(225) Wooden buildings having:

(yyyyyyyyyyyyyyy) structural system of beam walls;

(zzzzzzzzzzzzzzz) braced frame type structural system, with bolt type joints;

(aaaaaaaaaaaaaaa) dome-type structural system with two- or three-joint arches;

(bbbbbbbbbbbbbbb) cantilever-type structural system;

or

(ccccccccccccccc) dome-type structural system with arches.

shall be seismically designed for ductility class DCL.

(226) Buildings located in areas of moderate or high seismicity shall be designed for ductility class DCH or DCM.

(227) By way of exception from (226), in areas of moderate or high seismicity, buildings may be designed for the ductility class DCL if their overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q=1$), irrespective of the location, where it is not possible to meet the design criteria specific to the ductility class DCH or DCM.

(228) Structures not falling within the types indicated 9.2.1, (217), shall be designed for seismic actions for the ductility class DCL so that their overall resistance capacity to horizontal seismic actions, corresponding to the elastic response, is greater than or equal to the seismic requirement corresponding to the design spectrum of horizontal accelerations ($q=1$).

(229) The main structures shall be seismically designed for the ductility class DCL on the basis of the provisions of Chapters 1, 2, 3 and 4 of this technical regulation and the provisions of SR EN 1995-1-1, together with the provisions explicitly indicated for this class of ductility in this chapter.

(230) Structures made of wood panels with bracing plates fixed to the wood frame with metal staples shall be designed for the ductility class DCL.

(231) All main structural components, regardless of the type of structural system, shall be designed for the same class of ductility.

9.2.3 Plastic mechanisms

(232) In the case of design for DCM or DCH, plastic deformations occur in joints made with steel elements.

(233) The positions of the plastic deformation zones for structures that are designed for ductility class DCH or DCM are laid down in technical regulation NP 005.

9.2.4 Behaviour factors

9.2.4.1 Ultimate limit state

(234) The behaviour factor shall be chosen according to the capacity of the structure to dissipate earthquake-induced energy. The maximum values of the behaviour factor for different types of structures and classes of ductility are provided in Table 9.1.

Table 9.1 Maximum values of behaviour factors for horizontal seismic actions, *q*

Structural system	Ductility class		
	DCH	DCM	DCL
Structural systems of cross-laminated wood walls	3.20	2.30	1.50
Structural wall system made of wood panels	4.00	2.50	1.50
Structural system of beam walls	-	-	1.50
Frame-type structural system with one level and one opening	3.30	2.00	1.50
Frame type structural system, multiple openings and one level	3.60	2.00	1.50
Frame-type structural system, multiple openings and multiple levels	3.90	2.00	1.50
Braced frame structural system	-	-	1.50
Cantilever structural system	-	-	1.50
Arched dome structural system with two or three joints	-	-	1.50
Arched dome structural system	-	-	1.50

(235) In the case of an irregular building, the maximum value of the behaviour factor shall be reduced in accordance with the provisions of 4.2.2, compared to the values laid down in Table 9.1

(236) The maximum value of the behaviour factor resulting from the application of the provisions of this paragraph shall be lower limited to 1.00.

9.2.4.2 Service limit state

(237) The maximum behaviour factor value for horizontal seismic actions for service limit state checks shall be 1.50 for buildings designed for ductility class DCM or DCH and 1.00 for buildings designed for ductility class DCL.

9.2.5 Modelling for calculation

(238) For timber-structured buildings, when calculating the design value of seismic action, the fraction of the critical damping of the building, ξ, for all modes of vibration, shall be assumed to be equal to 4 %:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad \xi=4\% \qquad (9.1)$$

(239) The calculation of wooden structures shall be made in accordance with the provisions of Chapter 4 together with the provisions given in technical regulation NP 005.

(240) Horizontal wooden diaphragms, designed in accordance with technical regulation NP 005, may be modelled as rigid flat diaphragms if conditions a) and b), c) or d) are met:

(ddddddddddddddd) their openings do not significantly affect the overall rigidity of the floor plan: a compact floor in which the ratio of the dimensions in the two main directions does not exceed 2.0; floor for which recesses not situated along the perimeter are less than 10 % of the floor area;

(eeeeeeeeeeeeeee) for all structural types calculated in the ductility class DCL, the floor diaphragm and joints shall be designed so as to be able to transfer seismic force to the vertical resistance structure using an overstrength factor γ_d equal to 1.50;

(fffffffffffffff) for all types of structures other than cross-laminated timber structures and wood-panel walled structures designed for ductility class DCM or DCH, the diaphragm and its joints shall be made in such a way as to transfer seismic force in the plane to the vertical resistance structure, using an overstrength factor of 2.00;

(ggggggggggggggg) for cross-laminated and walled wooden structures designed for ductility class DCM or DCH, the diaphragm and its joints shall be such as to transfer seismic force in the plane to the main vertical structural components.

(241) Wood-concrete composite floors can be considered rigid diaphragms if they meet the following conditions:

(hhhhhhhhhhhhhhh) their openings do not significantly affect the overall rigidity of the floor plan: for a floor of a compact shape in accordance with (240), withdrawals of less than 20 % of its surface may be assumed not to significantly affect the overall rigidity in the plan;

(iiiiiiiiiiiiiii) over-concreting must be at least 50 mm thick and must be connected to all wooden structural elements.

9.3 Seismic performance criteria

9.3.1 General information

(242) The provisions of this section apply to the main structure, with a role in balancing the seismic action.

(243) In the seismic design of wooden structures, the provisions given in this chapter apply together with the specific provisions of the other technical regulations for the design of wooden buildings, according to the 5.1.1, (369).

9.3.2 Resistance

(244) Wooden structures shall be constructed in such a way as to meet the condition of resistance to horizontal action laid down in 4.3.1.1, (199) and (201).

(245) The design value of the resistance capacity shall be greater than or equal to the design value of the effort in the section considered. This condition shall be fulfilled for all main structural components along their entire length.

(246) The design value of the resistance capacity shall be determined in accordance with the provisions of NP 005.

(247) For structures classified in the ductility class DCM, the resistance capacity of non-dissipative zones may be limited above the value corresponding to the elastic response of the structure to the design seismic action, corresponding to the ultimate limit state. In the case of the use of reduced spectrum in design, these efforts shall correspond to a value of the behaviour factor q equal to 1.00.

9.3.3 Ductility

(248) Wooden structures shall meet the conditions of ductility under horizontal seismic action laid down in 4.3.1.2.

(249) In the case of wooden buildings, it is not necessary to fulfil the condition relating to the limitation of horizontal movements under design seismic action, corresponding to the ultimate limit state.

9.3.4 Stability

(250) Wooden structures shall be made in such a way as to meet the conditions of stability under seismic action given at 4.3.1.3.

9.3.5 Rigidity

(251) Wooden structures shall be made in such a way as to meet the conditions of rigidity under horizontal seismic action given at 4.3.2.1.

(252) The design value of the permissible level relative displacement shall be determined in accordance with the provisions of 4.3.2.1, (238).

9.4 Stresses design values

(253) The design effort values for timber structures shall be established in accordance with the provisions of technical regulation NP 005.

9.5 Composition conditions

(254) Wooden structures shall be constructed in accordance with the provisions of technical regulation NP 005.

10 Non-structural components

10.1 Purpose and scope

(255) This chapter contains provisions on the design of non-structural components of buildings required for seismic actions.

(256) All building components, with the exception of structural components, shall be considered as non-structural components. Non-structural components can be attached to the structure or other non-structural components.

(257) The provisions of this chapter concern:

- non-structural components;
- attachments of non-structural components to the main structure or other non-structural components;
- structural or non-structural components to which the non-structural components are attached.

(258) Non-structural components shall be constructed in such a way as to meet the basic requirements of seismic design given in chapter 2 by:

(jjjjjjjjjjjjjjj) fulfilling the provisions regarding the limitation of the horizontal movements of the building to the service limit state and the final limit state, according to the provisions of Chapter 4,

and

(kkkkkkkkkkkkkkk) meeting the seismic performance criteria given in this chapter for different categories of components, in accordance with their role.

(259) Non-structural components and their attachments shall be constructed in such a way as to meet the seismic performance criteria given at 10.2. The effects of design seismic action on non-structural components shall be determined in accordance with the provisions of 10.4.

(260) A non-structural component:

(lllllllllllllll) shall be designed, in full compliance with the provisions of technical regulations or specific Romanian standards, in accordance with the performance requirements of seismic design, if such normative documents are in force;

(mmmmmmmmmmmmmmm) shall be selected for use on the basis of the provisions of the technical approvals, which shall include information on the suitability for use under seismic stress conditions, under dynamic, alternating cyclic conditions, in accordance with the performance requirements of the seismic design, if no Romanian technical regulations or standards are available with provisions specific to the type of component and the conditions of application.

(261) Non-structural components shall be designed or selected in accordance with the main structural system type.

10.2 Seismic performance criteria

(262) A non-structural component shall meet the seismic performance criteria taking into account:

(nnnnnnnnnnnnnnnn) direct effect, caused by horizontal accelerations and forces acting on the component as a result of seismic action;

(oooooooooooooooo) the indirect effect, caused by the deformations imposed on the component by the relative horizontal displacements of its gripping points, as a result of seismic action.

(263) For the establishment of seismic performance criteria, the role of a non-structural component shall be classified as follows:

(pppppppppppppppp) essential role in the functioning of the building, for components whose termination at design seismic action is only accepted for the time necessary to replace the power supply or non-structural components supporting it;

(qqqqqqqqqqqqqqqq) supporting and/or feeding a critical non-structural component;

(rrrrrrrrrrrrrrr) secondary role for the functioning of the building, for components whose cessation of operation over a long period of time is accepted, not preventing the activity in the building.

(264) Essential non-structural components shall be determined by the investor and/or beneficiary through the design theme. Non-structural components supporting and/or supplying essential components shall be determined by the designer.

(265) Seismic performance criteria for non-structural components, in accordance with the basic requirements of seismic design set out in Chapter 2, are:

(ssssssssssssssss) non-structural components which by falling may endanger the safety of building users and/or persons located in public spaces adjacent to the building shall retain their position at the design seismic action corresponding to the ultimate limit state;

> Note: For the purposes of this provision, public spaces adjacent to buildings are the spaces immediately adjacent to the building to which access by persons is permitted, where non-structural components may collapse.

(ttttttttttttttt) non-structural components which, by their dislocation, fall and/or damage, are capable of hindering or restricting the movement of persons on the building escape routes shall retain their position at the design seismic action corresponding to the ultimate limit state;

> Note: This category also includes non-structural walls, floors, suspended ceilings, finishes and furniture on escape routes.

(uuuuuuuuuuuuuuuu) structural components with a role in the evacuation of building users shall retain their stability and shall not break at the design seismic action, corresponding to the ultimate limit state;

(vvvvvvvvvvvvvvvv) in the case of class I buildings of importance and exposure to earthquakes, the non-structural essential and supporting components shall retain their function, without the need for repair, at the design seismic action corresponding to the service limit state, considering the design values of the spectral accelerations at the service limit state established in accordance with the provisions of Chapter 3 multiplied by 2.00;

> Note: This is the case, for example, of doors at fire stations.

(wwwwwwwwwwwwwwww) in the case of buildings in classes of importance and exposure to earthquakes II, III and IV, non-structural components shall retain their function at the incidence of design seismic action corresponding to the service limit state, without requiring repair;

Note: In the sense of the provisions of par. (vvvvvvvvvvvvvvvv) and (wwwwwwwwwwwwwwwww), only repairs essential to ensure the function of the component shall be considered, in accordance with the applicable basic quality requirements and the provisions of the architectural design and installations. Restoring the aesthetic appearance of an architectural component does not fall into this category.

(xxxxxxxxxxxxxxxx) Gas, hot water or steam installations and containers containing significant quantities of toxic or explosive substances, the damage of which may endanger the safety of building users and/or nearby persons, shall retain their integrity when seismic design action corresponding to the ultimate limit state.

(266) For the purposes of the provision of (265), (ssssssssssssssss), any non-structural component with a mass of more than 10 kg that may fall from a height of 3.00 m or more shall be considered to endanger the safety of building users and/or persons located in public spaces adjacent to the building, as appropriate.

(267) The investor and/or beneficiary may establish, through the design theme, performance criteria for non-structural components for seismic action corresponding to the service limit state, additional to those provided for in (265), in order to limit their degradation.

10.3 Safety check of non-structural components

(268) Non-structural components shall be constructed in such a way as to meet the seismic performance criteria for horizontal forces and displacements generated by seismic action, determined in accordance with 10.4, corresponding to the limit state considered, as follows:

(yyyyyyyyyyyyyyyy) the design value of the seismic force acting on the non-structural component, F_{CNS}, is less than the permissible force of the non-structural component, $F_{CNS,adm}$, established in accordance with seismic performance criteria;

$$F_{CNS} \leq F_{CNS,adm} \qquad (10.1)$$

(zzzzzzzzzzzzzzzz) design value of the relative horizontal displacement between the attachment points of the non-structural component, d_{rCNS}, is smaller than the permissible relative displacement, $d_{rCNS,adm}$,established in accordance with seismic performance criteria.

$$d_{rCNS} \leq d_{rCNS,adm} \qquad (10.2)$$

(269) Attachments of non-structural components shall be such that the design value of their resistance capacity is greater than the connecting forces corresponding to the seismic force acting on the non-structural component or the relative horizontal displacement of the non-structural component, multiplied by a safety coefficient equal to 1.30:

$$R_d > 1{,}30\, E_d \qquad (10.3)$$

where:

R_d the design value of the clamping resistance capability;

E_d the design value of the binding force in the clamp.

(270) Verification of the deformation capacity of the facades hanging from the structure, including the glazed facades, shall be done considering the values of the horizontal displacements of the structure caused by the design seismic action, at the clamping points, multiplied by a safety coefficient equal to 1.30.

(271) Structural components shall be such that when loaded with the connecting forces with the non-structural components determined in accordance with (269), they meet the seismic design criteria given in Chapter 4.

(272) Non-structural components shall be such that their interactions with the structural components are controlled and the connecting forces do not cause degradation of the structural components or change in the plastic mechanism of the main structure.

> Note: Such degradations can occur as a result of changing the static scheme, e.g. by forming short pillars, or by introducing additional efforts in structural components, e.g. joint damage in masonry panels framed in frames.

(273) In the case of a non-structural component falling under the provision given on 10.5, (306), (oooooooooooooooo), condition (10.1) can be expressed through efforts that develop into its elements: axial force, shear force, bending moment or torque, for each direction of seismic action:

$$R_{d,CNS} \geq E_{d,CNS} \tag{10.4}$$

where:

$R_{d,CNS}$ the design value of the resistance capacity determined in accordance with the applicable technical regulation or Romanian standard;

$E_{d,CNS}$ the design value of the effort from the non-structural component in the seismic grouping;

(274) Non-structural walls, regardless of the material from which they are made, shall be made in such a way as to meet the seismic performance criteria given at 10.2, for:

(aaaaaaaaaaaaaaaa) seismic action perpendicular to the plane of the wall, where the mass of the non-structural component includes the mass of the wall and the mass of furniture or other non-structural components attached to the wall;

(bbbbbbbbbbbbbbbb) seismic action in the plane of the wall, depending on how it is attached to the main structure.

(275) When checking non-structural masonry components for seismic actions at the final limit state, the following partial safety coefficient values for masonry are used:

(cccccccccccccccc) for non-structural components attached to the envelope and outside walls, whether or not framed: $\gamma_M = 1{,}90$;

(dddddddddddddddd) for framed or unframed interior walls: $\gamma_M = 1{,}50$.

(276) When checking non-structural masonry components for seismic actions at the service limit state, the partial safety coefficient for masonry is $\gamma_M = 1{,}50$.

(277) When verifying the condition of stability under horizontal forces of a non-structural component, the following provisions shall be met:

(eeeeeeeeeeeeeeeee) the effects favourable to the stability of the component of the gravitational action acting on the component shall be reduced by multiplication by 0.90;

(fffffffffffffff) the effects favourable to the stability of the component of vertical seismic action shall not be taken into account;

(ggggggggggggggggg) the effects unfavourable to the stability of the component of vertical seismic action shall be taken into account.

(278) In the case of buildings with agglomerations of people, for the calculation of parapets and handrails on the escape routes, the seismic action perpendicular to the plane considers simultaneous with the load from the push exerted by people, established according to SR EN 1991 regulations, for the persistent design situation.

(279) When designing non-structural walls, framed or unframed, which are supported on main structural components in the cantilever or on wide-opening beams, the effect of vertical deformations produced by seismic movement, including deformations caused by the rotation of the joint in the bearing section, shall be taken into account.

10.4 Effects of seismic action

10.4.1 Seismic force

(280) The design value of the seismic force caused by the direct effect of the earthquake on a non-structural component shall be determined using one of the following methods:

(hhhhhhhhhhhhhhhh) equivalent static force method;

(iiiiiiiiiiiiiii) the storey spectrum method;

(281) The design seismic force determined in accordance with this chapter shall be used for the design or selection of the non-structural component, its connections and for the local verification of the supporting elements. When checking the supporting components, the effects of this force shall be combined with the effects of the seismic force acting on the building as a whole.

(282) The equivalent static force method shall be applied to all non-structural components that are seismically projected.

(283) In the case of non-structural components of high importance or which, due to damage, pose a particular risk, the method of floor spectra shall also be applied when determining the design value of the seismic force. The design shall consider the most covering value of the seismic force resulting from the application of the two methods.

(284) In determining the design value of the seismic force acting on a non-structural component, account shall be taken of the importance and earthquake exposure factor of the non-structural component, $\square_{CNS,}$ established as follows:

(jjjjjjjjjjjjjjjj) $\square_{CNS}$1,50, for the following categories of non-structural components :

- essential and supporting components for the continued operation of, or safe evacuation of, class I buildings;

- components located on escape routes and emergency escape lighting systems in major class I and II buildings with large numbers of persons;

- containers and reservoirs containing toxic or explosive substances in quantities considered dangerous to public safety;

The value shall be established by the designer and/or at the request of the investor/user, through the design theme.

Where these non-structural components are attached to other non-structural components, the factor of importance determined in accordance with the provisions of this paragraph shall also apply to the non-structural components on which they are supported and to the links with them.

(kkkkkkkkkkkkkkkk) $\square_{CNS}=\square_{I,e}$ for non-structural components that do not belong to the categories indicated in (jjjjjjjjjjjjjjjj), where $\square_{I,e}$ is the importance and earthquake exposure factor of the building corresponding to the limit condition considered.

(llllllllllllllll) according to the provisions of technical regulation GP 128, for steel shelves with the last level of storage located at a height greater than or equal to 3.00 m from the base.

(285) The effects of horizontal and vertical seismic action shall be combined in accordance with the provisions of this Chapter. 4.

(286) In determining the design seismic force for plant and equipment systems, account shall also be taken of the dynamic effects of the piping system, machinery and equipment and their connections.

10.4.1.1 Equivalent static force method

(287) For the design of non-structural components, the effect of the direct action of the earthquake on the non-structural component is equivalent to the effect of a horizontal force, applied statically -equivalent horizontal static seismic force.

(288) Equivalent horizontal static seismic force, F_{CNS}, which quantifies the effect of direct earthquake action on a non-structural component, is determined with the relation:

$$F_{CNS}=S_{ap,h}\frac{\gamma_{CNS}\beta_{CNS}K_z}{q_{CNS}}m_{CNS} \qquad (10.1)$$

where:

$S_{ap,h}$ the value of the absolute horizontal spectral acceleration corresponding to the plateau between the corner periods T_B and T_C, by constant accelerations of the elastic response spectrum calculated for a critical damping fraction ξ equal to 5 %, for checks at the limit state considered;

γ_{CNS} importance factor of the non-structural component, determined in accordance with 10.4.1, (284);

β_{CNS} dynamic amplification factor of the non-structural component, determined according to (289) and ;

q_{CNS} behaviour factor of the non-structural component, determined in accordance with (290) or (291);

m_{CNS} the maximum mass of the non-structural component in service;

K_z ground acceleration amplification factor at the height of the construction:

$$K_z = 0{,}40 + 0{,}80 \frac{z}{H} \tag{10.2}$$

z the elevation of the structural attachment point of the non-structural component measured in relation to the conventional f;

H the average height of the roof in relation to the conventional fixation-clamping section;

(289) Value of the non-structural component dynamic amplification factor for ultimate limit state or service limit state checks, $_{CNS},\beta$ shall be determined in accordance with the provisions of Table 10.1.

(290) The value of the behaviour factor of the non-structural component for checks at the ultimate limit state, $q^{ULS}_{CNS,}$ shall be determined in accordance with the provisions of Table 10.1.

Table 10.1 Factors β_{CNS} and q_{CNS} for non-structural components

Category and type of non-structural components	β_{CNS}	q^{SLU}_{CNS}
Elements attached to the envelope of the structure:		
- cantilevered or anchored to the main structure below the level of the centre of gravity, irrespective of the material Note: Such elements are, for example, chimneys or ventilation, parapets, attics.	2.50	1.50
- anchored above the level of the centre of gravity	1.00	2.50
- ornaments, firms, advertisements, television antennas and the like, regardless of how they are attached to the main structure	2.50	1.50
Envelope elements		
- external non-structural walls, regardless of material, resting in the cantilever Note: Such elements are, for example, the walls of turbot, leaning against the structure at the bottom.	2.50	1.50
- framed external non-structural walls, regardless of material, and framed masonry panels	1.00	1.50
- plywood and finishings with ductile elements and fixings	1.00	2.50
- plywood and finishes with fragile elements and grips	1.00	1.50
- fixings and bracing of the envelope elements	1.25	1.00
Fixed or non-movable partitioning elements, including finishings and embedded joinery		
- framed interior non-structural walls and framed panels of	1.00	2.50

simple masonry;		
- interior non-structural walls and panels of simple masonry not fixed to the structure at the top	2.50	2.50
- single masonry internal bulwarks leaning against the cantilever or fixed below the centre of gravity	2.50	2.50
- single masonry interior guardrails fixed above the level of the centre of gravity	1.00	2.50
- interior partitioning elements made of materials other than masonry	1.00	2.50
Suspended ceilings.	1.00	2.50
Raised floors		
- simple systems	1.00	1.50
- special systems	1.00	2.50
Surrounding fences	2.50	2.50
Sanitary facilities		
- pipe systems made of deformable materials with flexible connections	2.50	6.00
- pipe systems made of brittle materials (cast iron, glass, non-ductile plastic)	2.50	3.00
Electrical installations/lighting		
- main overhead cable systems	2.50	6.00
- rigidly mounted main cable systems	1.00	2.50
- electrical equipment	1.00	2.50
- lighting fixtures	1.00	1.50
Conditioning/heating & ventilation systems		
- outdoor equipment	2.50	6.00
- neoprene insulated vibration equipment	2.50	2.50
- equipment isolated with springs against vibration	2.50	2.00
- equipment that is not isolated against vibration	1.00	2.50
- equipment installed on pipes	1.00	2.50

- other equipment	1.00	2.50
Special installations with machines operating with steam or water at high temperatures		
- boilers, reservoirs	1.00	2.50
- pressure vessels supported on the mantle or positioned freely	1.00	2.50
Electromechanical equipment		
- lifts and escalators	1.00	2.50
Furniture		
- furniture in medical, research facilities, including computer systems; office furniture (shelves, binders, cabinets)	1.00	1.50
- furniture and exhibits in museums of national interest	1.00	1.00
- furniture and special furnishings in structures belonging to importance class IV: (control panels for dispatch offices of emergency services, fire units, police stations, telephone exchanges, equipment in radio and television broadcasting stations)	1.00	1.00
- steel racks with the last level of storage located at a height greater than or equal to 3.00 m from the base	According to technical regulation GP 128	

(291) Value of the non-structural component behaviour factor for service limit state checks, $q_{CNS,}^{SLS}$ is equal to 1.00.

(292) In the case of non-structural components attached at the level of two successive floors with elevations z_{inf} and $z_{¿}$, F_{CNS} is considered evenly distributed over the height of the level. For the calculation F_{CNS} in the expression of the factor K_z the mean value of the size z is used:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad z=\frac{z_{¿}+z_{inf}}{2} \tag{10.3}$$

Note: This is the particular case of non-structural masonry walls framed in concrete, steel or composite frames.

(293) Equivalent horizontal static seismic force, F_{CNS}, shall be limited according to the relations:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad \begin{array}{l} F_{CNS}\geq 0,30\,S_{ap,h}\,\gamma_{CNS}\,m_{CNS} \\ F_{CNS}\leq 1,60\,S_{ap,h}\,\gamma_{CNS}\,m_{CNS} \end{array} \tag{10.4}$$

(294) Equivalent static vertical seismic force $F_{CNS,V}$ shall be determined with the relation (10.1) using the acceleration value of the vertical component determined in accordance with the provisions of Chapter 3.

(295) For the calculation of the strength and stability of a non-structural component, the equivalent static seismic force, F_{CNS} is considered to be acting as:

- uniformly distributed load, perpendicular to the axis of the non-structural component, horizontally and vertically, in the case of linear elements that can oscillate simultaneously on the two directions - pipes, ducts, ventilation ducts and the like;
- uniformly distributed or concentrated load, perpendicular to the plane of the non-structural component, in the case of vertical or inclined plane elements - interior and exterior walls, curtain facades and the like;
- uniformly distributed or concentrated load in the CNS plane, in the case of horizontal plane elements - suspended ceilings, raised floors;
- concentrated force applied in the centre of gravity of the non-structural component, in the most unfavourable direction, in the case of elements having three comparable orthogonal dimensions - machinery, equipment, tanks, chimneys and ventilation and similar.

10.4.1.2 The storey spectrum method

(296) The horizontal acceleration of the non-structural component shall be determined from the elastic response spectrum expressed in accelerations of the floor corresponding to the attachment point of the non-structural component.

(297) The elastic response spectrum expressed in accelerations of the floor is determined by the spectral analysis of the floor movement in the horizontal plane determined by the non-linear dynamic calculation of the structure as a whole, according to the provisions of Chapter 4.

(298) Seismic action shall be modelled in the calculation in accordance with the provisions of Chapter 3.

(299) It is also recommended to use this model for non-structural components of categories A1, A2 and B4 of buildings with a height of 50 m or more.

10.4.2 Horizontal displacements

(300) A non-structural component that is clamped at two different elevations on the same structure or section shall be so constructed as to meet the seismic performance criteria given on 10.2 for a design value of the relative horizontal displacement between the attachment points, $d_{r,CNS}$, determined with the relation:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad d_{r,CNS}=d_{si}-d_{sj} \tag{10.1}$$

where:

d_{si} the design value of the horizontal movement, at the level elevation '*i*';

d_{sj} the design value of the horizontal movement, at the level elevation '*j*';

(301) A non-structural component that is clamped at two different elevations on two different structures or sections shall be so constructed as to meet the seismic performance criteria given on 10.2 for a value of the relative horizontal displacement between the attachment points, $d_{r,CNS}$, determined with the relation:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad d_{r,CNS}=|d_{siA}|+|d_{sjB}| \tag{10.2}$$

where:

d_{siA} the design value of the horizontal movement at level 'i' in building 'A' at the considered limit state;

d_{sjB} the design value of the horizontal movement at level 'j' in building 'B' at the limit state considered.

(302) The values of the horizontal displacements of the structure under design seismic action, corresponding to the ultimate limit state or service limit state, constitute theme data for the design or selection of non-structural components.

10.4.2.2 Ultimate limit state

(303) For checks at the ultimate limit state, the design value of the relative horizontal displacement between the attachment points shall be determined in accordance with 10.4.2, (300) or (301), considering the design values of the relative level displacements at the ultimate limit state.

10.4.2.3 Service limit state

(304) For checks at the service limit state, the design value of the relative horizontal displacement between attachment points shall be determined in accordance with 10.4.2, (300) or (301), considering the design values of the relative level displacements at the service limit state.

(305) In the case of:

(mmmmmmmmmmmmmmmmmm) elements attached to the envelope placed on facades adjacent to the public property or to other spaces where crowding of persons is possible;

(nnnnnnnnnnnnnnnnn) piping systems which are fixed to two adjacent sections, in the case of buildings in class I or II of importance and exposure to earthquakes;

the design values of the relative level displacements at the service limit state determined in accordance with the provisions of Chapter 4 shall be multiplied by 1.30.

10.5 Permitted values:

(306) The permissible values of the horizontal forces capable of loading a non-structural component and the permissible values of the relative displacements between its attachment points shall be determined:

(ooooooooooooooooo) in accordance with the provisions of the technical regulations or the Romanian design standards specific to the type of component, where such normative documents are in force,

or

(ppppppppppppppppp) according to the provisions of the technical approval.

(307) The technical approval of a non-structural component of any type for the certification of seismic performance shall contain explicit provisions on:

(qqqqqqqqqqqqqqqqq) suitability for use under conditions of seismic stress, under cyclic-alternating dynamic conditions;

(rrrrrrrrrrrrrrrr) the values allowed for horizontal forces or horizontal accelerations capable of acting on the component, to ensure the function of the component and to ensure strength and stability;

(sssssssssssssssss) the permissible values of the relative deformations between the attachment points, in the case of multi-point components, in order to ensure the function of the component and to ensure strength and stability;

(tttttttttttttttt) how the attachments are made and the connecting forces in the attachments associated with the level of horizontal forces permitted to ensure stability;

meeting the seismic performance criteria given on 10.2.

Technical approval refers to the non-structural component as a whole, including all the parts and embedded materials, the links between them and the attachments to the structure, to other non-structural components or the backing on the ground, as the case may be.

Note: The technical approvals of the constituent parts of a non-structural component shall not replace the technical approval of the non-structural component.

(308) The effect of friction due to the own weight of the non-structural component, the components supported by it or the applied loads shall not be taken into account to ensure the slip stability of the non-structural component.

10.6 Supplementary provisions

(309) This paragraph contains minimum provisions on the design or selection of non-structural components in certain categories. These provisions shall apply in addition to the provisions in specific technical regulations or specifications in technical approvals of products.

(310) The provisions on the composition of non-structural components given in this paragraph apply to all non-structural components, with the limitations set out herein, regardless of the type of dominant action over them.

10.6.1 Architectural components of masonry

(311) In buildings designed for ductility class DCH or DCM, masonry elements of groups 1 and 2 may be used for the execution of non-structural masonry components, as classified in SR EN 1996-1-1.

(312) Non-structural masonry walls and framed masonry panels shall not be made of masonry elements made of burnt clay with horizontal voids made when casting.

(313) Non-structural masonry components in buildings of class I or II of importance and exposure to earthquake shall be made of masonry elements of category I, defined according to technical regulation CR 6.

Non-structural masonry components in buildings of class III or IV of importance and exposure to earthquake shall be made of masonry elements of category I or II, defined according to technical regulation CR 6.

(314) Non-structural components in masonry shall be made with mortar with a characteristic value of compressive strength greater than or equal to 2.50 N/mm^2. Non-structural masonry components in buildings in classes of importance and exposure to earthquakes I or II shall not be made with mortar prepared at the site.

(315) The facade walls made up of two layers of masonry with internal void shall be provided with anchors of solidarity according to the provisions of SR EN 1996-1-1. Anchors shall be made in such a way as to comply with the provisions of SR EN 845-1.

The number and dimensions of anchors shall meet the following conditions:

- for establishments characterised by $S_{ap,h}^{SLU} \leq 2{,}50\, m/s^2$, minimum 2 anchors shall be provided for each m^2 of wall;
- for establishments characterised by $2{,}50\, m/s^2 < S_{ap,h}^{SLU} \leq 6{,}00\, m/s^2$, minimum 3 anchors shall be provided for each m^2 of wall;
- for establishments characterised by $S_{ap,h}^{SLU} > 6{,}00\, m/s^2$, minimum 4 anchors shall be provided for each m^2 of wall.

(316) In areas with moderate or high seismicity, masonry walls resting at the bottom on the slabs are not allowed if they are bounded at the top by other structural components, such as beams or slabs.

10.6.1.1 Non-structural masonry walls

(317) This paragraph contains additional minimum provisions for the fulfilment of seismic performance criteria for non-structural masonry walls.

(318) Non-structural masonry walls are made of masonry panels and, where appropriate, pillars and reinforced concrete belts.

(319) In buildings located in areas of moderate or high seismicity, the exterior walls of the masonry shall be framed on all four sides by the contribution of the main structural components, secondary ones, pillars and/or belts.

(320) At the edges of voids in walls having an area of 2.50 m or more^2, for establishments characterised by $2{,}50\, m/s^2 < S_{ap,h}^{SLU} \leq 6{,}00\, m/s^2$, and voids in walls of 1.50 m or more^2, for establishments characterised by $S_{ap,h}^{SLU} > 6{,}00\, m/s^2$, pillars and reinforced concrete belts with anchorage in the main structural components shall be provided.

(321) The framed masonry walls by the contribution of the main structural components shall be made in such a way that the unarmed masonry panels meet the conditions:

- panel area less than or equal to 12.0 m^2;
- the height of the panel is less than or equal to 3.50 m;
- the length of the panel is less than or equal to 5.00 m.

Note: If the strength of the panels of unreinforced masonry is insufficient, the following measures can be taken: the dimensions of the panel are reduced by the introduction of reinforced concrete pillars, in addition to those introduced for the plating of gaps, the masonry is plastered with reinforced plaster with steel mesh, polymer grilles or fibre-reinforced polymers (FRP), or another constructive solution or other materials is adopted for the walls concerned.

(322) Unframed masonry walls by the contribution of the main structural components shall be made in such a way that the unbarred masonry panels meet the conditions:

- panel area less than or equal to 9.0 m^2;
- the height of the panel is less than or equal to 2.50 m;
- the length of the panel is less than or equal to 3.50 m.

Note: Unframed walls of masonry are also walls resting on consoles or walls with large gaps for which a system of diagonals is not made by confining the wall by the main structural components with which it is in contact.

(323) The blind walls, fronton wall or lunettes made of masonry or other major masonry elements working in a vertical cantilever under horizontal loads perpendicular to the plane shall be secured against overturning by the provision of pillars and/or belts at the top. The pillars have their rotation and movement restricted to the base by connecting to the main structural components. For masonry panels higher than 2.0 m, horizontal intermediate belts shall be provided so that the maximum height of a masonry panel between two horizontal concrete elements does not exceed 2.0 m.

Note: Reinforced concrete pillars have a higher bending resistance capacity than bending moments caused by overturning masonry elements without the contribution of masonry.

(324) Lateral wall stability shall be ensured by connecting structural components and/or by weaving with the intersecting non-structural masonry walls.

(325) Interior walls with partial height are not fixed laterally by the intake of the suspended ceiling.

Note: If the normal unit bending efforts perpendicular to the wall plane are greater than the design values of the resistors, one or more of the following solutions shall be adopted:

- the wall is reinforced in the horizontal joints if, from the calculation, it results that the breakage occurs in a plane perpendicular to the horizontal joints in the wall field and at the supports;

- the dimensions of the masonry panel are reduced by the introduction of middle belts and pillars; the belts and pillars are anchored to the structure and are sized to take over their horizontal loads.

10.6.1.2 Attics, parapets, chimneys

(326) This paragraph contains additional minimum provisions for meeting seismic performance criteria for masonry attics, bulwarks and chimneys.

(327) The stability of attics and bulwarks under horizontal seismic action perpendicular to the plane shall be ensured by one or a combination of the following measures:

(uuuuuuuuuuuuuuuuu) the use of reinforced concrete intermediate pillars and reinforced concrete belts at the top, connected to the main structural components;

Note: Reinforced concrete pillars have a higher bending resistance capacity than bending moments caused by overturning masonry elements without the contribution of masonry.

(vvvvvvvvvvvvvvvvvv) the reinforcement continues in the horizontal joints, with the anchoring of the reinforcements in the main structural components.

(328) The stability of chimneys or ventilation made of masonry under horizontal seismic action shall be ensured by one or a combination of the following measures:

- covering the masonry with plaster reinforced with nets, at which the vertical bars are anchored in the main structural components;
- external plating with laminated profiles anchored in the main structural components, covered with plaster;
- anchoring baskets with cross-ties fixed in the main structure.

(329) Cornices and girdles, which exceed the masonry plane by no more than half of the wall thickness, shall be made with HD-type elements, by removing the bricks in increments of no more than 1/4 of the brick in each row. When designing the cornices, account shall be taken of the roll-over effect caused by the gravitational action on the part of the cornice that exceeds the plane of the masonry. The stability of the cornices shall be ensured by the provision of pillars and reinforced concrete belts.

(330) Cornices that exceed the face of the outer wall by more than half its thickness shall be made of reinforced concrete elements.

10.6.2 Prefabricated concrete exterior walls

(331) This paragraph contains additional minimal provisions for meeting the seismic performance criteria of precast concrete walls.

(332) External walls made of prefabricated concrete panels, mounted after the execution of the structure, are leaned directly on the elements of the main structure or are connected to it with anchors or other mechanical devices.

(333) The connections and joints between the panels shall allow for relative level displacements at least equal to the relative level displacement corresponding to the ultimate limit state, but not less than 15 mm.

10.6.3 Exterior glass walls

(334) This paragraph contains additional minimum provisions for meeting seismic performance criteria for glazed facades.

(335) In the case of facades placed towards public spaces or with agglomerations of people, regardless of the importance and exposure class of the building, the glass of the windows with an area of more than 2.00 m^2, and which are situated at a height ≥ 2.00 m above the pavement level, shall be of the 'secure' type.

10.6.4 Suspended ceilings

(336) Suspended ceilings of buildings in classes of importance and exposure to earthquakes I, II or III, located in areas with moderate or high seismicity, shall meet the following conditions:

- in each of the two horizontal orthogonal directions, one end of the ceiling support grid shall be fixed to the edge structural element and the other end shall be capable of free movement of at least 20 mm;

- ceilings with a surface area ≥ 100 m^2 are attached in a horizontal direction to the main structure;

- ceilings with a surface > 250 m^2 shall be divided into surface areas of ≤ 250m^2 by separating joints or walls developed over the entire height of the floor; this measure may be waived if it is demonstrated by calculation that the fixing system can fully accommodate the lateral movements of the ceiling;

- arrangements shall be made to permit free horizontal movement of the ceiling in the vicinity of sprinkler heads or other parts passing through the ceiling;

- in the case of ceilings that develop in areas at different elevations, the lateral stability of each area is ensured by its own system of limiting horizontal displacements (wind bracing);

- ducts, ventilation ducts, electrical wiring and other installation elements are fixed to the suspended ceiling.

10.6.5 Raised floors

(337) The floor shall meet the seismic performance conditions taking into account the loads established according to SR EN 1991-1.

(338) In order to determine the seismic requirement represented by the equivalent static seismic force, the mass of the raised floor is determined taking into account the unladen mass of the floor, the total mass of the fixed equipment and 1/4 of the mass of the mobile equipment resting on the floor.

(339) The floor shall meet the conditions of seismic performance with consideration also of the efforts caused by the rollover effect of rigidly fixed equipment to the floor.

(340) If heavy equipment is to be mounted on the floor (indicatively, weighing more than 5.0 kN), the panels shall be checked for a concentrated load corresponding to a small machine (indicatively, a concentrated load of 10 kN).

(341) The connections that transmit seismic forces to the floor shall be made by parts fixed to the floor.

10.6.6 Non-structural components located on escape routes

(342) For the safe evacuation, in case of design seismic action, corresponding to the ultimate limit state, of buildings located in areas with moderate or high seismicity, it is recommended to apply the following measures regarding the construction elements and finishes located on the evacuation routes:

- the doors of the garages of rescue stations, fire brigades and the like shall permit the transition of motor vehicles to design seismic action corresponding to the ultimate limit state, for relative displacements equal to the relative level displacements associated with the ultimate limit state multiplied by 1.50;

- Escape doors of buildings that can accommodate a large number of people (indicatively more than 250 people) shall be designed in such a way that they do not lock for relative displacements equal to $1.50\ d_{r,CNS}$ where $d_{r,\mathrm{CNS}}$ is the value calculated for the ultimate limit state;

- the doors of the main rooms of buildings of importance classes I and II (classrooms, for example) and the escape doors of buildings of importance classes I ÷ III shall be designed in such a way that they do not lock themselves for relative displacements equal to $1.25d_{r,CNS}$ where $d_{r,CNS}$ is the value calculated for the ultimate limit state;

- awnings over escape doors in the building shall meet the seismic performance criteria for a vertical seismic force higher by 50 % than that in the relation (10.1) for buildings in importance classes I and II and by 25 % for buildings in importance class III;

- floors, suspended ceilings and other finishes on escape routes shall be such that their dislocation, fall and/or damage do not hinder or restrict the movement of persons;

- in buildings belonging to importance Classes I and II, any furniture items placed along the evacuation routes shall be fixed onto the structure or its non-load-bearing walls in accordance with Article 10.4.1.

10.6.7 Facilities

10.6.7.1 Linear components

(343) Movements of pipes, ducts, ducts which are laid in a horizontal, vertical or inclined direction shall be restricted vertically and for two horizontal orthogonal directions by the installation of cross-ties and bracing , as appropriate.

(344) Pipes laid in a vertical direction, which are located at a distance of more than 1.0 m from a pillar or structural wall, shall allow different vertical movement of the floors, in compliance with the performance criteria given at 10.2.

(345) Vertically located ducts allow relative displacement between the floors they cross or between their attachment points, in compliance with the performance criteria given at 10.2.

(346) Water pipes are essential to ensure the functioning of the building.

(347) Water pipes passing from the ground to the building and voids passing through the building envelope or foundations shall be designed in such a way that the pipes maintain their tightness at the highest credible value of the horizontal movement of the building relative to the ground. A minimum joint of 25 mm between the pipe and the edge of the gap shall be provided at the passageways throughout the perimeter.

(348) Gas ducts passing from the ground into the building and gaps through the building envelope or foundations shall be designed in such a way that the ducts maintain their tightness at the highest credible value of the horizontal movement of the building relative to the ground.

(349) The pipes that cross the joints between two adjacent building blocks and the passageways shall be made in such a way that they allow the relative horizontal movement of the building blocks, in compliance with the performance criteria given at 10.2.

(350) Suspended components that are installed along piping paths and weigh more than 10 kg shall have horizontal movements restricted by the installation of bracings, independently of those used to fasten piping.

(351) The automatic fire-fighting systems with sprinklers shall be designed for seismic actions, in accordance with the performance criteria given in this technical regulation, according to the provisions of SR EN 12845-3.

10.6.8 Lifts

(352) High-speed lifts (indicatively above 45 m/minute) shall be fitted with decoupling devices calibrated for a ground acceleration value of 50 % of the design seismic acceleration for the SLS.

(353) The counterweights of the lifts shall be fitted with devices to avoid their egress from the guide rails and their impact with the cab.

(354) Locking devices shall be provided at the bottom and top of the cab and at the counterweight.

10.6.9 Escalators

(355) Escalators in crowded areas (shopping centres, exhibition halls, airports and the like) shall be constructed in such a way as to meet the seismic performance criteria for relative displacements between bearing points 25 % higher than those corresponding to the limit state considered.

10.6.10 Shelves of steel

(356) Steel shelves with the last level of storage located at a height greater than or equal to 3.00 m from the base shall be seismically designed in accordance with technical regulation GP 128, in compliance with the additional provisions given in this technical regulation.

(357) Shelves shall be constructed in such a way as to meet the seismic performance criteria given at 10.2 in P 100-1 for non-structural components.

(358) The values of the ordinates of the reduced spectrum of accelerations shall be determined according to the provisions of the chapters 3 and 4 from P 100-1. Maximum value of the behaviour factor, *q*, shall be considered:

(wwwwwwwwwwwwwwwww) according to the provisions of the technical regulation GP 128, for checks at the ultimate limit state;

(xxxxxxxxxxxxxxxxx) equal to 1.00 for checks at the service limit state.

(359) The partial safety coefficient for seismic action shall be considered as:

(yyyyyyyyyyyyyyyyy) according to the provisions of the technical regulation GP 128, for checks at the ultimate limit state;

(zzzzzzzzzzzzzzzzz) equal to 1.00, for checks at the service limit state.

(360) In order to meet the seismic performance criteria at the service limit state given at 10.2, (265), (vvvvvvvvvvvvvvvvv) or (wwwwwwwwwwwwwwwww), where appropriate, shelving components, joints between them and their structural attachments or foundation elements shall be made in such a way as to respond elastically having a resistance capacity greater than or equal to the effect of the design seismic action corresponding to the service limit state established in accordance with the provisions of 10.3.

(361) Shelves shall be constructed in such a way as to maintain local and general stability to the seismic design action corresponding to the ultimate limit state.

(362) The structural components to which the shelves for storage are attached or on which they rest shall meet the strength requirements laid down in technical regulation

P 100-1 for main structural components, under the action of the connecting forces established in accordance with the provisions of 10.3.

10.6.11 Resistance of framed masonry panels

(363) Design value of shearing strength in plane of masonry panels framed in frames F_{Rd} shall be determined with the relation:

$$z=\frac{z_{¿}+z_{inf}}{2} \quad F_{Rd}=minim\left(F_{Rd1},F_{Rd2},F_{Rd3}\right) \tag{10.1}$$

where

F_{Rd1} the design value of the tensile strength of the shear force in horizontal joints;

$$z=\frac{z_{¿}+z_{inf}}{2} \quad F_{Rd1}=f_{vd0}A_{pan}k_{1,pan} \tag{10.2}$$

F_{Rd2} the design value of the strength corresponding to the cracking along the compressed diagonal;

$$z=\frac{z_{¿}+z_{inf}}{2} \quad F_{Rd2}=f_{vd0}A_{pan}k_{2,pan} \tag{10.3}$$

F_{Rd3} the design value of the strength corresponding to the crushing of the compressed diagonal at the corner of the frame.

$$z=\frac{z_{¿}+z_{inf}}{2} \quad F_{Rd3}=minim\left(f_d b_{st,ech} t_p k_{3,pan} k_{5,pan}, f_{dh} A_{pan} k_{4,pan}\right) \tag{10.4}$$

A_{pan} the horizontal section area of the panel;

$$z=\frac{z_{¿}+z_{inf}}{2} \quad A_{pan}=t_p l_p \tag{10.5}$$

t_p the thickness of the masonry panel;

l_p the length of the masonry panel;

E_c, E_z modulus of elasticity of concrete in the frame and masonry (short-term values);

f_d the design value of the compressive strength of the masonry perpendicular to the joint;

f_{dh} the design value of the compressive strength of the masonry parallel to the joint;

f_{vd0} design value of zero compressive stress shearing strength of the

masonry;

$b_{st,ech}$ side of the pillar of the equivalent frame to be determined with the relation

$$z=\frac{z_{¿}+z_{inf}}{2} \quad b_{st,ech}=\sqrt[4]{6\left(I_1+I_2\right)}$$

I_1, I_2 moments of inertia in the frame plane of the pillars

Table 10.1 Factor values $k_{1,pan}$, $k_{2,pan}$, $k_{3,pan}$, $k_{4,pan}$

$\lambda_p = h_p/l_p$	0.50	0.75	1.00	1.50	2.00
$k_{1,pan}$	1.20	1.45	1.70	2.50	3.30
$k_{2,pan}$	1.90	2.15	2.40	3.05	3.70
$k_{3,pan}$	0.640	0.512	0.400	0.245	0.160
$k_{4,pan}$	0.111	0.125	0.141	0.180	0.224

Table 10.2 Factor values $k_{5,pan}$

E_c/E_z	h_p/t_p				
	6.0	8.0	10.0	12.0	14.0
4.0	1.20	1.28	1.35	1.41	1.47
6.0	1.32	1.41	1.50	1.57	1.63
8.0	1.41	1.52	1.61	1.68	1.75
10.0	1.50	1.61	1.70	1.78	1.85
12.0	1.60	1.72	1.81	1.90	1.97
14.0	1.70	1.83	1.92	2.02	2.09

λ_p the form factor of the panel;

$$z = \frac{z_{¿} + z_{inf}}{2} \qquad \lambda_p = \frac{h_p}{l_p} \tag{10.6}$$

h_p the height of the masonry panel;

θ angle formed by the diagonal of the framed masonry panel with the horizontal axis;

10.6.12 Other provisions

(364) In the case of non-structural components of buildings located in areas with moderate or high seismicity attached to the structure with post-installed anchors, the strength of the post-installed anchors is verified by in-situ tests based on a control program included in the project.

(365) Shot-mounted bolts shall not be used as tension anchors for non-structural components.

(366) The attachment of non-structural components with adhesives shall not be taken into account when verifying the seismic performance criteria laid down in 10.2. Exceptionally, lightweight surface elements fixed with adhesives throughout their surface by structural components or non-structural surface components.

11 Seismic devices

11.1 Definitions

(367) The terms used in this chapter have the following meanings:

Insulation system: all components used for seismic isolation;

Insulation interface: the continuous interface that completely separates the infrastructure from the superstructure, where the containment system is positioned;

Insulating devices: the components of the containment system that meet the conditions of 11.2.1, (371).

Infrastructure: the part of the structure located below the insulation interface, including the foundations. The lateral flexibility of the infrastructure is negligible compared to that of the containment system;

Superstructure: that part of the structure which is insulating and situated above the insulation interface;

Complete isolation: insulation which provides the superstructure with elastic behaviour to horizontal seismic actions corresponding to the ultimate limit state.

Partial isolation: insulation which does not provide the superstructure with elastic behaviour to horizontal seismic actions corresponding to the ultimate limit state.

Effective centre of rigidity: centre of rigidity to horizontal actions of the insulation interface.

Note: In buildings, the rigidity of the superstructure may be neglected when determining the position of the effective centre of rigidity.

Depreciation centre: the horizontal damping centre of the containment interface.

The design value of the horizontal displacement of the insulation system shall be the maximum horizontal displacement of the effective centre of rigidity, recorded under the design seismic action, corresponding to the ultimate limit state, between the upper face of the infrastructure and the lower part of the superstructure, in a given horizontal direction.

The design value of the total horizontal displacement of an insulating device is the maximum horizontal displacement of the device, in a given horizontal direction, also considering the overall rotation effect of the superstructure around the vertical axis.

The effective rigidity of the containment system is the ratio between the value of the total horizontal force transmitted through the containment interface and the absolute design value of the horizontal displacement.

The effective period is the vibration period of a system with a single degree of dynamic freedom having the mass and rigidity equal to the mass of the superstructure and the effective rigidity of the insulation system.

The effective damping of the insulation system is the equivalent viscous damping value corresponding to the energy dissipated by the insulation system for a cyclic response with an amplitude equal to the design value of the horizontal displacement.

Drive-dependent energy dissipation device: device connecting two structural elements without transmitting vertical loads. Their behaviour is primarily dependent on travel and secondary speed. Devices with linear or non-linear behaviour are included in this category;

Energy dissipating device, speed dependent: device connecting two structural elements without transmitting vertical loads. Their behaviour is primarily dependent on speed and secondary movements. Includes viscous fluid type shock absorbers and viscous-elastic shock absorbers;

Rigid devices: devices connecting two structural elements without transmitting bending moments or vertical loads. Included in this category are: permanent connection devices, fuse safety devices, temporary connection devices;

11.2 Isolation of the base

11.2.1 Purpose and scope

(368) The chapter contains provisions on the design of fully seismically insulated structures, where the insulation system is arranged below the main mass of the superstructure and aims to reduce the seismic response of the main structure to seismic actions.

(369) Reducing the seismic response of the main structure to seismic actions can be achieved by increasing the fundamental period of vibration of the structure, by increasing damping or by combining these effects. The insulation system can be made of insulators or insulators and linear or non-linear shock absorbers.

(370) The chapter deals with passive energy dissipation systems that are arranged at a single interface. Passive energy dissipation systems that are distributed at several levels of the structure are dealt with in Chapter 11.3

(371) Insulation devices which may be used in accordance with the provisions of this chapter are: laminated bases of elastomers, viscous or frictional damping elastoplastic devices, pendulums and other devices the behaviour of which conforms to the requirements of (369). Each device has one or more of the following properties:

(aaaaaaaaaaaaaaaaaa) high rigidity and vertical strength, with high flexibility in the horizontal direction;

(bbbbbbbbbbbbbbbbbb) high energy dissipation capacity through viscous or hysteretic cushioning;

(cccccccccccccccccc) ability to return to the initial horizontal position after the end of the seismic action;

(dddddddddddddddddd) sufficient rigidity at non-seismic horizontal loads to meet the requirements at service limit conditions.

11.2.2 Basic requirements

(372) When designing seismic insulated structures, the basic requirements of seismic design given in chapter 2 shall be complied with.

(373) Insulation devices and their structural attachments shall be designed for a higher degree of safety than that used in the design of the structure by increasing the seismic action applied to each device with a safety coefficient $\gamma_x = 1,50$.

(374) Non-structural components crossing the containment interface shall be designed to meet the seismic performance criteria given in Chapter 10, taking into

account the relative movements between the infrastructure/land and the superstructure.

11.2.3 Criteria for compliance with the requirements

(375) In order to satisfy the basic requirements of seismic design, the building shall be checked against the requirements of the limit states defined in 2.3.

(376) Utility networks connecting the superstructure must maintain their function according to the importance and earthquake exposure class of the building, according to the provisions of the chapter 10.2, (265).

> Note: For this purpose, it is recommended that the connecting parts of the superstructure networks are designed in such a way as to allow for large relative displacements through elastic response to seismic action.

(377) Relative level movements of the superstructure and infrastructure shall be limited in accordance with the provisions of Chapter 4.

(378) The resistance and deflection capacity of the insulation devices is higher than the seismic requirement corresponding to the ultimate limit state, amplified considering the safety coefficient established according to the provision in 11.2.2, (373).

(379) The infrastructure shall be designed to respond elastically to the design seismic action corresponding to the ultimate limit state.

(380) The superstructure shall be designed for the ductility class DCL as specified in Chapters 5-9, specific to structures made of different materials, considering the connecting forces with the isolation interface.

(381) The infrastructure shall be designed for the ductility class DCL as specified in Chapters 5-9, specific to structures made of different materials, considering the connecting forces with the isolation interface and the inertial seismic forces acting directly on the infrastructure mass.

> Note: When designing infrastructure and superstructure it is not necessary to apply the design-to-capacity method.

(382) At the ultimate limit state, the gas and other networks which may cause disastrous effects, passing through the separation surfaces of the superstructure from the surrounding terrain or other constructions, shall be designed to withstand the relative displacements between the isolated superstructure and the surrounding terrain or constructions, considering a safety coefficient. γ_x, as defined in 11.2.2, (373).

(383) Insulation devices shall be selected or designed in compliance with the provisions of SR EN 15129 and in accordance with the provisions of the European Technical Certificate (EC).

11.2.4 General design provisions

11.2.4.1 General provisions regarding isolation devices

(384) Installation, inspection, maintenance and replacement of insulation devices shall be carried out in accordance with the provisions of SR EN 15129. Sufficient space between the superstructure and the infrastructure shall be provided for this purpose.

(385) In the case of devices protected from the potential effects of hazard sources such as fire, chemical or biological attack, the protection shall not affect the functioning of the devices during an earthquake.

(386) The fire protection of the insulation devices shall be designed in accordance with the fire protection requirements of the building. This protection shall not affect the operation of the containment devices during the earthquake.

Note: The design of the devices shall be carried out in accordance with environmental factors, including wind, aging effects, ambient temperature, operating temperature, exposure to moisture or other harmful substances.

(387) Insulation devices sensitive to fatigue failure at low amplitude stress cycles will have a linear-elastic response to wind action.

(388) Vertical load transmitting isolating devices shall comply with a minimum overlap length to ensure the operation of the support in seismic displacements amplified by the safety coefficient, γ, determined in accordance with the provision of $_{x}$11.2.2, (373).

(389) Insulation devices transmitting vertical loads shall be sufficiently rigid in the vertical direction.

11.2.4.2 Control of undesirable movements

(390) The effective centre of rigidity and the damping centre of the isolation interface shall be as close as possible to the projection of the centre of masses on the isolation interface.

Note: This reduces the torsion effects around the vertical axis at the insulation interface.

(391) Compressive effort resulting from permanent action shall be as evenly distributed as possible between the containment devices.

(392) The containment system shall be designed so as not to cause shocks or, if they do occur, shall be controlled by appropriate mitigation devices.

(393) The rotation of the ends of the insulation devices around any horizontal axis shall be upper limited to 0.005 rad.

11.2.4.3 Control of differential ground movements

(394) Immediately above and below the isolation interface, rigid and durable horizontal diaphragms shall be provided.

(395) By way of exception from (394), insulation devices may be placed on the height of main vertical structural components, pillars or walls, if the relative horizontal displacements of the vertical elements under design seismic action, corresponding to the ultimate limit state, are less than 1/20 of the relative displacement of the insulation devices, determined without taking into account the safety coefficient γ.$_{x}$

11.2.4.4 Control of the relative displacements in respect with the ground and the surrounding buildings

(396) The distance between the superstructure and the surrounding terrain or buildings is greater than the maximum displacement caused by the design seismic action, corresponding to the ultimate limit state, amplified by the safety coefficient γ_x, as defined in 11.2.2, (373).

11.2.4.5 Insulation system retrieval capability

(397) The containment system has the ability to retrieve in both main horizontal directions. This requirement is met when the system has small residual displacements in relation to its displacement capacity.

(398) The containment system shall fulfil the condition:

$$E_s \geq 0{,}25\, E_D \qquad (11.1)$$

where

E_s is the accumulated reversible energy (elastic deformation energy and potential energy) of the structure, including the containment system;

E_D is the energy dissipated by seismic devices

for displacements between 0 and the maximum displacement caused by design seismic action corresponding to the ultimate limit state, d_{Ed}^{SLU}.

(399) By way of exception from (398), systems with bilinear behaviour in the horizontal direction and complying with the (400) may be used, if the relation is fulfilled:

$$\frac{d_{Ed}^{SLU}\, K_P}{F_0} \geq 0{,}5 \qquad (11.2)$$

where:

d_{Ed}^{SLU} seismic displacement of the containment system in the direction considered;

K_P post-elastic (tangent) rigidity;

F_0 force corresponding to zero displacement of the containment system to cyclic actions, not including the contribution of speed-dependent devices (Figure 11.1);

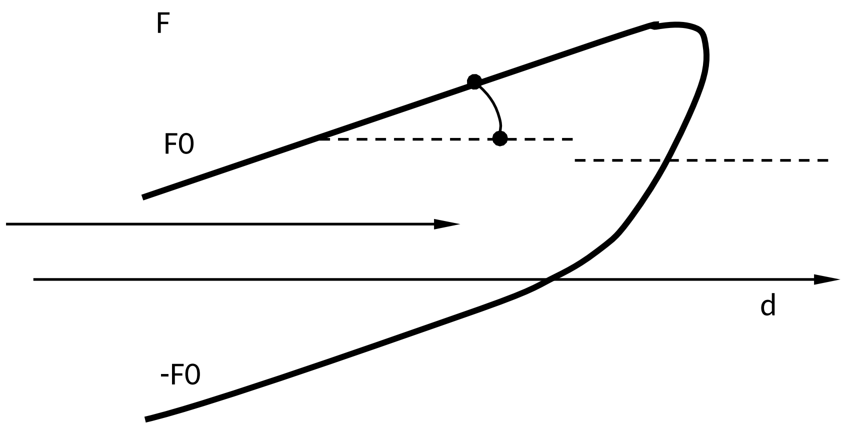


Figure 11.2 Definition of the equivalent bilinear model for the assessment of the retrieval capability

(400) The deviation of the force-movement relation of the isolation system from the nearest bilinear form shall not exceed $\pm 15\%$ for no movement in the interval $0{,}3\, d_{Ed}^{SLU}$ - $1{,}0\, d_{Ed}^{SLU}$ on the loading segments. When applying this criterion to friction systems, the

effect of vertical load variation on the force-movement relation of the insulation system may be neglected.

(401) The alternative method defined in (399) can be applied ignoring the favourable elastic contribution of the infrastructure.

(402) It may be assumed that systems that do not satisfy (398) or (399) comply with the provision of (397) if the containment system is designed considering the displacements amplified by the safety coefficient, γ_x, determined in accordance with the provision of 11.2.2, (373), and the factor ρ_d established with the relation:

$$\rho_d = 1 + 1{,}50\left(1 - 2\frac{d_{Ed}^{SLU} K_P}{F_0}\right) \geq 1{,}0 \qquad (11.1)$$

where d_{Ed}, K_P, F_0 are defined in (399).

Residual displacement increased by factor $2(\rho_d - 1) d_{Ed}^{SLU}/3$ must be compatible with the function of the building.

11.2.4.6 Limitation of movement of the containment system

(403) The containment system shall have sufficient lateral constraint to meet all the relevant requirements of the applicable technical regulations relating to the fulfilment of the service limit state requirements.

Note: For example, this is the case with wind loads on buildings.

(404) Where restraint devices of the fuse type are provided, their flow limit shall be limited upwards to 40 % of the design value of the horizontal force corresponding to the elastic response of the device to the ultimate limit state.

(405) Where temporary connection devices are used, for application (1), they shall be included in the model.

11.2.4.7 Seismic action

(406) When seismic design, the simultaneous action of the three components of seismic action is considered, as defined in chapter 3.

(407) The combination of the effects of seismic action shall be carried out in accordance with the provisions of Chapter 3.

(408) In the case of the application of dynamic calculation, at least seven sets of accelerograms meeting the requirements of 3.2.

(409) At completely insulated structures, the provisions of 4.5 shall apply with T_{eff} instead of T_1, according to 11.2.5.2.

11.2.4.8 Modelling

(410) The modelling of the structure shall reflect with sufficient accuracy the spatial distribution of the seismic devices so as to adequately consider the translation in the two horizontal directions, the corresponding turning moments and the rotation around the vertical axis. The model will also adequately reflect the properties of different types of seismic devices.

(411) The properties of seismic devices shall be worst-case scenario during the lifetime of the construction, reflecting, where relevant, the influence of a) to e):

(eeeeeeeeeeeeeeeee) the loading step;

(fffffffffffffffff) the size of the vertical loads;

(gggggggggggggggggg) the size of the simultaneous horizontal loads;

(hhhhhhhhhhhhhhhhhh) the temperature;

(iiiiiiiiiiiiiiii) changes in properties during the lifetime of the construction.

(412) In order to capture the variation in properties of seismic systems, multiple analyses shall be carried out. Upper and lower limits of the properties of each seismic device shall be determined for each modelling parameter.

(413) the horizontal flexibility of the infrastructure shall be taken into account, including, where relevant, the land-structure interaction;

(414) at least two analyses shall be run regardless of the calculation method chosen:

(jjjjjjjjjjjjjjjjj) The acceleration and inertial forces induced by an earthquake shall be determined by taking into consideration the rigidity and minimum values of the damping and friction coefficients.

(kkkkkkkkkkkkkkkkkk) displacements shall be determined on the basis of minimum values of rigidity and damping and friction coefficients.

(415) Where non-linear dynamic calculation is applied, the following conditions shall apply:

(lllllllllllllllll) the model shall include the non-linear force-speed-movement characteristics of the energy dissipation devices to explicitly take into account the dependence of the device on the amplitude, frequency and duration of the seismic movement;

(mmmmmmmmmmmmmmmmmm) the design of displacement-dependent devices shall include their hysteretic behaviour, in accordance with the experimental results and taking into account any significant changes in the strength, rigidity and shape of the hysteretic curve;

(nnnnnnnnnnnnnnnnnn) the design of speed-dependent energy-dissipating devices shall include a speed coefficient in accordance with the experimental results; damping characteristics that change over time and/or due to temperature shall be explicitly modelled;

(oooooooooooooooooo) if the properties of the energy dissipating devices change in the domain of seismic action effects determined by analysis, the dynamic response may be captured based on the minimum and maximum design characteristics of the devices, for the analysed domain, determined in accordance with SR EN 15129;

(pppppppppppppppppp) the maximum situation provides, depending on the relevance, at the same time maximum values of speed coefficient, rigidity, energy dissipation and resistance. The minimum situation for analysis and design provides, depending on relevance, at the same time minimum values of speed coefficient, rigidity, energy dissipation and resistance.

Note: Minimum values of speed coefficients and energy dissipation typically produce maximum design forces.

(416) Maximum value of the behaviour factor, *q*, is equal to 1.50.

11.2.5 Calculation of the structure

11.2.5.1 General information

(417) The dynamic response of the structure is characterized in terms of accelerations, inertia forces and displacements.

(418) The effects of seismic action shall be assessed by structural calculation using at least the following methods:

(qqqqqqqqqqqqqqqqqq) linear dynamic calculation, according to the provisions of 11.2.5.4;

and

(rrrrrrrrrrrrrrrrr) simplified linear calculation, in line with the limitations referred to in 11.2.5.3.

(419) When designing the building, the most unfavourable values of the effects of seismic action determined by the two structural calculation methods shall be used. The effects of seismic action determined by linear dynamic calculation shall be limited below the values established by simplified linear calculation, equivalent to the fundamental vibration mode.

(420) Insulated superstructures shall be designed for an elastic response to design seismic action corresponding to the ultimate limit state. If, when designing insulated structures, there are suspicions about the behaviour in the elastic domain of the superstructure, the demonstration of the elastic response of the superstructure is made by nonlinear dynamic calculation.

(421) In determining the effects of seismic action, account shall be taken of the overall torsion effects of the building, including those due to accidental eccentricity, and of second-order effects.

(422) In isolated structures equipped with seismic insulators and/or displacement-dependent devices and speed-dependent devices, the phase difference between maximum displacement and maximum speed may be considered when determining the effects of seismic action. Thus, the total horizontal force of the containment system in the direction considered corresponds to simultaneous values of increased speeds and displacements to take account of the effect of the combined use of the devices.

(423) In order to determine the effects of seismic action, the values of the physical and mechanical characteristics of the insulators that determine the worst effects shall be used. For this purpose, several scenarios may be used to combine the values of these characteristics. The values of the physical and mechanical characteristics shall be established according to the technical specifications of the seismic devices.

(424) By way of exception from (423), for buildings classified in class III or IV of importance and exposure to earthquakes, the average values of the physical and mechanical characteristics of insulators may be used, where the minimum and maximum values of these characteristics do not differ by more than 15 % from the average values.

11.2.5.2 Modelling of the dynamic behaviour of the insulating system

(425) The provisions of this paragraph shall be used for the simplified modelling of the dynamic behaviour of the insulating system by a model with linear viscous-elastic

behaviour, when it is made of laminated elastomeric supports, or a hysteretic bilinear model, when the system is made of elasto-plastic devices.

(426) Simplified modelling of dynamic behaviour shall be used if the effective rigidity of the insulating system is greater than or equal to 1/3 of the secant rigidity of the insulating system corresponding to a displacement $0.2d_{Ed}$.

(427) In case of using the equivalent linear model the effective rigidity K_{eff} of the insulating system is the sum of the effective rigidities of the insulators. The effective rigidity of each insulator is the value of the dry rigidity at the total design displacement, d_{Ed}.

(428) In the case of the use of the hysterical bilinear model, the rigidity corresponding to the elastic response of the insulation system shall be used, which shall be determined as the sum of the rigidities of the insulators.

(429) If the equivalent linear model is used, the energy dissipation of the insulating system may be expressed by the effective damping, ξ_{eff}, which expresses the equivalent viscous depreciation:

$$\xi_{eff} = \frac{1}{2\pi}\left[\frac{\sum E_{D,i}}{K_{eff}\, d_{Ed}^2}\right] \quad (11.1)$$

where

d_{Ed} the design value of the horizontal displacement of the system equivalent to a single degree of freedom under design seismic action corresponding to the ultimate limit state;

$E_{D,i}$ energy dissipated by seismic devices.

(430) The energy dissipated by seismic devices shall be set for stress cycles in the frequency range of the relevant vibration modes.

(431) For higher vibration modes with frequencies outside this range, the modal damping factor of the structure as a whole shall be that of the superstructure considered as recessed at the base.

(432) If the values of the actual rigidity or effective damping of the buffers depend on the displacement, d_{Ed}, their determination shall be made by iterative calculation, until the difference between the selected and the calculated value does not exceed 5 % of the selected value.

(433) If the equivalent linear model is used, the design seismic action shall be determined on the basis of a damping value corresponding to the actual vibration period, T_{eff}, established with the relation:

$$T_{eff} = 2\pi\sqrt{\frac{m_d}{K_{eff}}} \quad (11.2)$$

where

m_d design value of the mass of the superstructure;

K_{eff} the effective rigidity of the containment system, defined in (428).

11.2.5.3 Simplified linear calculation

(434) Simplified linear calculation is equivalent to the fundamental vibration mode and applies to completely insulated structures, which respond predominantly as systems with a single degree of freedom in each horizontal direction.

(435) In this calculation method, two horizontal dynamic translations are considered and the effects of the overall torsion are statically overlapping. The superstructure shall be modelled as a rigid solid that translates above the insulating system, with the (437) and (5).

(436) When calculating structures using the simplified linear calculation method, a maximum value of horizontal displacement shall be taken into account, d_{db}, established with the relation:

$$d_{db}=\gamma_x d_{Ed} \qquad (11.1)$$

where

d_{Ed} the design value of the horizontal displacement of the system equivalent to a single degree of freedom under design seismic action corresponding to the ultimate limit state;

γ_x safety coefficient, established in accordance with 11.2.2, (373).

(437) The behaviour of an isolated structure can be modelled as a system with a single degree of dynamic freedom for the application of the simplified linear calculation method if the cumulative conditions are met:

(sssssssssssssssss) the containment system can be considered linearly equivalent, according to 11.2.5.2, (426);

(ttttttttttttttttt) the contribution of the higher vibration modes of the superstructure and infrastructure can be neglected; this condition shall be deemed to be met if the effective modal mass driven in the fundamental vibration mode is at least 90 % of the total mass of the building;

(uuuuuuuuuuuuuuuuu) the effective period of the fully insulated structure shall be at least three times longer than the period of the superstructure recessed at the base, T_f;

(vvvvvvvvvvvvvvvvv) the equivalent viscous damping defined in 11.2.5.2, (429), is less than or equal to 40 % for sliding curved surface insulators and 30 % for all other types of seismic devices;

(wwwwwwwwwwwwwwwww) increasing the retracting force for the containment system for displacements between $0.5d_{db}$ and d_{db} is greater than 2.50 % of the total gravity load above the containment system, where d_{db} is established in accordance with (436).

(438) The simplified linear calculation method may be applied to insulation systems with equivalent damped linear behaviour, provided that all of the following conditions are met:

(xxxxxxxxxxxxxxxxx) the largest horizontal dimension of the superstructure shall be 50 m;

(yyyyyyyyyyyyyyyyyy) insulating devices shall ensure the direct transmission of vertical connecting forces from the superstructure to the infrastructure;

(zzzzzzzzzzzzzzzzzz) the actual period satisfies the relation:

$$3T_f \leq T_{eff} \leq 3s \tag{11.2}$$

where

T_{eff} actual period, defined in accordance with 11.2.5.2, (433);

T_f the period of the superstructure considered recessed at the base immediately above the containment system;

(aaaaaaaaaaaaaaaaaa) the ratio between the vertical and horizontal rigidities of the insulating system satisfies the condition:

$$\frac{K_v}{K_{eff}} \geq 150 \tag{11.3}$$

where

K_{eff} the effective rigidity of the containment system, defined in 11.2.5.2, (427);

K_v the vertical rigidity of the insulation system which is calculated as the sum of the vertical rigidities of the insulators, determined according to the technical specifications;

(bbbbbbbbbbbbbbbbbb) the fundamental period in the vertical direction fulfils the condition:

$$T_v = 2\pi\sqrt{\frac{m_d}{K_v}} \leq 0{,}10\,s \tag{11.4}$$

where

m_d design value of the mass of the superstructure;

K_v vertical rigidity of the insulation system.

(439) In determining the effects of seismic action, the influence of eccentricity between the effective centre of rigidity of the containment system and the centre of masses of the superstructure shall be considered.

11.2.5.4 Dynamic calculation

(440) This paragraph contains specific provisions for isolated structures for dynamic, linear or non-linear calculation.

(441) Dynamic calculation shall be carried out on three-dimensional models.

(442) The behaviour of the insulators shall be modelled by force-shift response laws that reproduce the behaviour of the system in the field of deformations and speeds anticipated for the seismic grouping.

(443) The inherent damping fraction, corresponding to the damping associated with the deformations of the main structural components and the deformation of the non-structural components, at the initiation of the flow into the main structural components, shall be taken to be less than or equal to 3 % of the critical value.

(444) For a set of accelerograms, the maximum value of a seismic effect shall be determined as the maximum value of this effect determined by calculation at each time step of the analysis.

11.2.6 Checking seismic devices at limit states

11.2.6.1 General information

(445) For non-seismic stress situations, the supports and seismic devices shall be checked in accordance with the requirements of the relevant Romanian standards of the EN series and, where relevant, with SR EN 1337-1.

11.2.6.2 Verifications at the ultimate limit state

(446) For checks at the final limit state, the displacement and/or rotation of seismic devices shall be determined by multiplying the displacement and/or rotation corresponding to the design seismic action, corresponding to the final limit state, by the safety coefficient, established according to 11.2.2, (373).

(447) For checks at the final limit state, the displacements caused by long-lasting deformations, temperature variations and vertical loads shall also be taken into account when calculating the displacement.

(448) The horizontal displacement of seismic devices caused by design seismic action is less than the displacement capacity of the seismic device, determined according to the product specifications.

(449) The rotation around the horizontal axis of the insulators caused by the design seismic action is less than the rotation capacity of the insulators, established according to the product specifications. When assessing the expected rotation, the influence of the construction deviations according to the product specifications shall also be taken into account. In the absence of this information, the rotation requirement due to design deviations shall be considered equal to 0.005 rad.

(450) The verification of the infrastructure shall be made considering the inertial seismic forces acting directly on it and the forces connecting it to the containment system.

(451) Depending on the type of seismic device, the resistance of the insulators and their grips on the infrastructure and superstructure must be checked at the ultimate limit state as follows:

(ccccccccccccccccccc) in terms of forces, taking into account the maximum and minimum vertical forces caused by non-seismic actions and the maximum vertical and horizontal forces caused by seismic design action, including the effects of the rollover moment;

(ddddddddddddddddddd) in terms of total relative horizontal displacements between the lower and upper faces of the insulator. Total horizontal displacement includes deformation caused by design seismic action and deformation caused by contraction, slow flow, temperature variations and/or post-tensioning.

Note: Safety coefficient, established in accordance with 11.2.2, (373), shall apply to forces and movements.

(452) Non-structural components of buildings shall be designed in accordance with the provisions of Chapter 10, taking into account the dynamic effects due to isolation.

11.3 Buildings equipped with passive seismic devices

11.3.1 Purpose and scope

(453) This chapter contains provisions on the design of structures equipped with passive energy control devices, which may be displacement dependent or speed dependent or rigid.

(454) The response of displacement-dependent energy dissipation devices is independent of the speed or frequency of excitation.

11.3.2 Basic requirements

(455) When designing structures equipped with seismic devices, the basic requirements of seismic design given in chapter 2 shall be complied with.

(456) Seismic devices and their structural attachments shall be designed for a higher degree of safety than that used in the design of the structure by increasing the seismic action applied to each device with a safety coefficient $\gamma_x = 1{,}50$.

(457) Structures equipped with energy dissipation devices meet the requirements of 4.3.

(458) In order to satisfy the basic requirements, the limit states defined in 2.3 shall be checked.

(459) Seismic devices shall be selected in accordance with the provisions of SR EN 15129 and the provisions of the European Technical Certificate (EC).

(460) By way of exception from (459), in the case of buildings designed for the ductility class DCL, rigid connecting devices and supports shall be selected in accordance with SR EN 1337 and in accordance with the provisions of the European Technical Certificate (EC).

(461) Structures of buildings equipped with passive seismic devices fall within the structural types defined in Chapters 5-9.

11.3.3 General design provisions

11.3.3.1 General provisions relating to seismic devices

(462) Installation, inspection, maintenance and replacement of seismic devices shall be carried out in accordance with the provisions of SR EN 15129.

(463) The fire protection of the devices shall be carried out in accordance with the fire protection requirements for the structure laid down in accordance with specific technical regulations. If a fire protection device is used, it shall be so constructed as not to impair the operation of the seismic device during the earthquake.

(464) Seismic devices shall be protected against the potential effects of hazard sources such as chemical or biological attack, in accordance with the provisions of specific technical regulations. The protective device shall not affect the operation of the seismic device during the earthquake.

(465) The selection of devices shall be carried out in accordance with environmental factors, including wind, aging effects, ambient temperature, operating temperature, exposure to moisture or other harmful substances.

(466) The service life of devices, for which the effects of fatigue are insignificant, shall be determined on the basis of the maximum service life of the devices, established in accordance with the product specifications.

(467) Seismic devices sensitive to fatigue failure at low-amplitude stress cycles shall be selected for linear-elastic response to wind action.

(468) In the case of displacement-dependent energy dissipation devices, at each level where they are installed, the flow-in of the seismic devices shall occur at a value of horizontal displacement of the structure equal to or less than 0.40 of the value corresponding to the flow-in of the main structural components at that level.

(469) In the case of displacement-dependent energy dissipation devices, at each level where they are installed, the resistance to horizontal action provided by seismic devices shall be lower than the resistance to horizontal action of the main structural components other than seismic devices at that level.

(470) All components of the energy dissipation system, excluding elements common to the main structural system and energy dissipation devices, shall be designed to remain within the elastic range of behaviour for seismic action effects multiplied by 1.50.

11.3.3.2 Torsion control

(471) The plan distribution of the energy dissipation devices shall provide the building with torsion strength and rigidity.

11.3.3.3 Seismic action

(472) The provisions of 11.2.4.7 apply.

(473) Flexible components of energy dissipation devices, which connect them with the main structure, are included in the model.

11.3.4 Calculation of the structure

11.3.4.1 General information

(474) The provisions of 11.2.5.1 paragraph (417) and (421) apply.

(475) Modelling of the structural system shall be carried out in accordance with the provisions 4.5.1.2.

11.3.4.2 Dynamic calculation

(476) The provisions of Chapter 4 and specific provisions of 11.2.5.4 (441), (443) and (444) apply.

11.3.5 Verification at limit states

11.3.5.1 General information

(477) Structural elements common to the main structural system and the energy dissipation system shall be considered as main structural components.

(478) 11.2.6.1, (445). applies

11.3.5.2 Verifications at the ultimate limit state

(479) The main structural components shall be checked in accordance with the provisions of Chapter 4.

(480) On structures equipped with speed-dependent energy dissipation devices, checks shall be made for maximum travel level, maximum speed and maximum acceleration.

(481) On structures equipped with displacement-dependent energy dissipation devices, checks shall be carried out for the maximum displacement level.

(482) All components of the energy dissipation system shall be checked in accordance with the specific provisions of this technical regulation. Checks shall be carried out in terms of:

(eeeeeeeeeeeeeeeeeee) forces, for speed-dependent energy dissipation devices;

(fffffffffffffffffff) displacements, for displacement-dependent energy dissipation devices;

(ggggggggggggggggggg) forces, for all other components of the energy dissipation systems.

11.3.5.3 Verification at the serviceability limit state

(483) At the limit state of service, the provisions of 4.3.2 apply.